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*Engineering optimization and adaptation
of vertical breakwaters in deep water*

Titolo della tesi

Matteo Centorami

Nome e Cognome del dottorando

firma

Alessandro Romano

Claudia Cecioni

Docente Guida/Tutor: Prof.

firma

Fabrizio Paolacci

Coordinatore: Prof.

firma

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*"The only real mistake is the one
from which we learn nothing."*

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Sommario

L'innalzamento del livello medio del mare e il cambiamento delle caratteristiche degli stati di mare estremi stanno mettendo sotto pressione le opere di difesa marittima, aumentando la probabilità che carichi d'onda e tracimazione superino le soglie considerate in fase di progetto. In questo contesto, l'ottimizzazione delle strutture costiere assume un ruolo centrale non solo come strategia di adattamento di opere esistenti, ma anche come esigenza progettuale per nuove infrastrutture chiamate a garantire vita utile più lunga, margini di sicurezza più elevati e sostenibilità economica in scenari climatici incerti.

Tra le diverse tipologie di opere per la protezione dei bacini portuali, le dighe frangiflutti a parete verticale (cassoni cellulari) riverstano un'importanza particolare. Tali strutture sono spesso impiegate in condizioni di acqua relativamente profonda, dove le soluzioni a scogliera possono diventare poco competitive per ingombri e volumi di materiale. Tuttavia, tali strutture sono caratterizzate da elevata riflettività e possono essere soggette a notevoli pressioni e a fenomeni di tracimazione capaci di governare sia la stabilità sia la funzionalità dell'opera. Ne deriva la necessità di investigare come specifiche ottimizzazioni influenzino la risposta idraulica della struttura sotto l'azione delle onde.

All'interno di questo quadro, una soluzione progettuale di crescente interesse riguarda l'ottimizzazione della sovrastruttura di coronamento e, in particolare, della geometria del muro paraonde. Una soluzione promettente, perché potenzialmente efficace senza richiedere modifiche estese al tronco del cassone, è l'arretramento del muro paraonde rispetto al fronte del cassone, con la creazione di una piattaforma (orizzontale o inclinata) che altera i meccanismi di interazione onda-struttura. Tale configurazione è già presente in opere reali, come la nuova diga foranea del Porto di Genova (in costruzione) e la diga verticale del Porto di Civitavecchia, oltre che in alcune

applicazioni in Giappone, talvolta con obiettivi specifici (ad es. protezione da tsunami). Nonostante l'applicazione di queste soluzioni in opere reali di grande rilevanza, l'ottimizzazione delle dighe verticali a cassone mediante arretramento del muro paraonde rimane un tema ancora poco esplorato dal punto di vista scientifico. In particolare, risultano carenti linee guida progettuali capaci di fornire criteri affidabili per la stima delle azioni idrodinamiche agenti su tali strutture, soprattutto in condizioni di acqua relativamente profonda e in presenza di processi di post-tracimazione. In tali condizioni, i flussi che si instaurano sul coronamento dopo l'innescio della tracimazione (*post-overtopping flow*) possono generare carichi impulsivi difficili da rappresentare con gli strumenti di progetto tradizionali; diventa quindi essenziale chiarire, in modo sistematico, come l'arretramento influenzi azioni globali e locali, riflessione ondosa e tassi di tracimazione, nonché la dinamica dei flussi responsabili dei picchi di pressione sul muro paraonde.

Su queste basi, la ricerca si basa su un articolato programma sperimentale sviluppato attraverso diverse campagne di modellazione fisica in scala ridotta, condotte sia con onde regolari sia con onde irregolari, e integrate, per un caso mirato, da simulazioni numeriche CFD. L'attività è strutturata attorno a quattro quesiti di ricerca principali (RQ) di seguito elencati: (RQ1) l'influenza dell'arretramento del muro paraonde sulle azioni globali e locali, sulla riflessione e sulla tracimazione; (RQ2) il ruolo dei diversi regimi di flusso post-tracimazione nella generazione di carichi impulsivi sul muro paraonde; (RQ3) l'effetto della variabilità stocastica legata alla generazione delle onde irregolari sulla stima delle grandezze misurate; e (RQ4) la natura e l'entità dei carichi indotti sulle celle interne dei cassoni durante la fase di costruzione.

I risultati hanno mostrato che l'arretramento del muro paraonde modifica in maniera significativa i meccanismi di interazione onda-struttura. In media, le configurazioni arretrate tendono ad aumentare le azioni impulsive sul muro, mentre possono ridurre le forze agenti sul tronco del cassone e, in alcuni casi, il coefficiente di riflessione. Al contempo, si osserva un incremento della tracimazione, particolarmente rilevante in condizioni prossime alla soglia di innescio, evidenziando come l'arretramento rappresenti una leva progettuale non lineare che richiede valutazioni caso per caso.

L'analisi dei carichi post-tracimazione consente di ricondurre i fenomeni osservati a tre regimi principali, adattati dalla classificazione dei fenomeni di "green water" in ambito navale: *Dam – Break* (DB), *Plunging – Dam – Break* (PDB) e *Hammer – Fist* (HF). I casi più gravosi risultano asso-

ciati a eventi di tipo Plunging-Dam-Break e Hammer-Fist combinati con piccoli arretramenti, nei quali le azioni impulsive sul muro e le azioni sul cassone possono risultare fortemente sincronizzate. L'integrazione tra misure di forza e analisi delle immagini permette una lettura orientata al processo e conduce alla definizione di parametri sintetici e di una mappa parametrica (ispirata a PROVERBS) utile per la classificazione degli eventi e per una prima stima dei carichi sul muro paraonde.

La tesi affronta inoltre lo studio della variabilità legata alla generazione delle onde irregolari, mostrando come il *seeding* influenzi in modo significativo la stima della tracimazione in condizioni caratterizzate da eventi rari (poche onde tracimanti), mentre le forze globali risultano generalmente meno sensibili. Infine, viene analizzata una condizione critica non operativa relativa alla fase di costruzione dei cassoni, evidenziando come i carichi sulle celle interne, ancora esposte e non protette dal coronamento, dipendano fortemente dal regime di riempimento e mostrando una chiara attenuazione spaziale delle pressioni procedendo dal lato mare verso terra.

Nel complesso, la presente tesi fornisce risultati e analisi di studio alla progettazione e all'adattamento delle dighe verticali a cassone in acque relativamente profonde, chiarendo i limiti e le potenzialità dell'arretramento del muro paraonde, proponendo un quadro interpretativo dei carichi post-tracimazione basato sui regimi di flusso, quantificando l'incertezza sperimentale legata alla natura stocastica delle onde irregolari e mettendo in luce una vulnerabilità rilevante durante la fase di costruzione.

Abstract

Sea level rise and changes in the characteristics of extreme sea states are placing increasing pressure on maritime defence structures, increasing the probability that wave loads and overtopping exceed the thresholds considered at the design stage. In this context, the optimization of coastal structures assumes a central role not only as an adaptation strategy for existing structures, but also as a design requirement for new infrastructures, which are required to guarantee longer service life, higher safety margins, and economic sustainability under uncertain climate scenarios.

Among the different types of maritime structures, vertical breakwaters (cellular caissons) play a particularly important role in coastal protection and are often employed in relatively deep water conditions, where rubble mound solutions may become uncompetitive in terms of footprint and material volumes. However, such structures are characterized by high reflectivity and may be subject to significant pressures and overtopping phenomena capable of governing both the stability and the functionality of the structure. This leads to the need to investigate how specific optimizations influence the hydraulic response of the structure under wave action.

Within this framework, a design approach of growing interest concerns the optimization of the crest superstructure and, in particular, the geometry of the wave wall. A promising solution, potentially effective without requiring extensive modifications to the caisson trunk, is to retreat the wave wall with respect to the seaside edge of the caisson trunk, thereby creating a platform (horizontal or sloping) that alters the wave-structure interaction mechanisms. This configuration has already been implemented in real-world structures, such as the new vertical breakwater of the Port of Genoa (currently under construction) and the vertical breakwater of the Port of Civitavecchia, as well as in some applications in Japan, sometimes with specific objectives (e.g., tsunami protection). Despite the adoption of these solu-

tions in important real-world projects, the optimization of vertical caisson breakwaters through wave-wall retreat remains a topic that has received limited scientific attention. In particular, design guidelines providing reliable criteria for estimating the hydrodynamic actions acting on such structures are still lacking, especially under relatively deep-water conditions and in the presence of post-overtopping processes. Under these conditions, the flows that develop over the crest after the onset of overtopping (*post-overtopping flow*) can generate impulsive loads that are difficult to represent with traditional design tools; it therefore becomes essential to systematically clarify how the retreat affects global and local actions, wave reflection and wave overtopping rates, as well as the flow dynamics responsible for pressure peaks on the wave wall.

On this basis, the research is founded on a comprehensive experimental program developed through several scaled physical modelling campaigns, conducted with both regular and irregular waves and complemented, for a targeted case, by CFD numerical simulations. The activity is structured around four main research questions (RQ), listed below: (RQ1) how the wave wall retreat influences forces, reflection and overtopping; (RQ2) how distinct post-overtopping regimes control impulsive loads on the wave wall; (RQ3) how random phase seeding in irregular-wave tests affects the estimation of measured quantities; (RQ4) how post-overtopping loads act on internal caisson cells temporarily exposed during construction.

First results (RQ1) show that crown-wall retreat substantially changes the interaction mechanisms. On average, retreated configurations tend to increase impulsive loads on the wall, while they can reduce loads on the trunk and thus certain components of the overall horizontal action; wave reflection may also decrease for sufficiently large retreats. At the same time, overtopping is found to be higher than in the flush arrangement, particularly in near-threshold conditions where the flush wall would yield negligible or no overtopping. Overall, the retreats emerges as a non-linear design lever that must be assessed as a trade-off between reductions in loads/reflection and increases in overtopping and impulsive impacts, requiring case-specific evaluation and, where feasible, dedicated modelling.

To study and interpret post-overtopping loads (RQ2), the observed processes on vertical caisson breakwaters are shown to be consistently described by three regimes, adapted from ship-deck green water classifications: Dam-Break (DB), Plunging-Dam-Break (PDB), and Hammer-Fist (HF). Exper-

iments confirm the occurrence of these regimes on caisson superstructures and support criteria for their onset as a function of wave and geometric parameters. The most critical scenarios are coherently associated with PDB and HF events combined with small retreats, where impulsive actions on the wall and on the trunk can become more “in phase”, leading to severe global loading conditions. Coupling force measurements with image-based analysis enables a process-oriented interpretation and supports the definition of synthetic indicators (e.g., air content and impact related descriptors), culminating in a parametric map inspired by the PROVERBS framework for event classification and preliminary loads estimation.

The thesis also addresses an aspect of irregular-wave testing (RQ3): the dependence of results on the phase seed used to assign random phases to spectral components. Through systematic repetitions (25 realizations per spectral condition), random seeding is shown to significantly affect overtopping estimates, especially when only a few realizations are available and overtopping is controlled by a small number of waves (high dimensionless freeboard). In such conditions, scatter increases markedly: the coefficient of variation remains low when overtopping is frequent, but can reach about 40% for mean discharge and overtopping probability when events become rare; extreme metrics (maximum overtopped volume) display similarly high variability, in some cases exceeding 40-50%. Conversely, global horizontal forces and synchronous pressures on the front face are generally less seed-sensitive, whereas downfall pressures and post-overtopping-related vertical forces exhibit larger scatter (with locally higher values for impulsive pressures), indicating that stochastic uncertainty should be explicitly accounted.

Finally, a non-operational critical condition is investigated (RQ4): the construction phase of caissons, when some internal cells may be temporarily open and not protected by the superstructure. The experiments show that internal loads are governed by the filling regime: during an initial transient (partially filled cells) more impulsive responses may occur; as filling progresses up to the crest level, a more stationary and repeatable (quasi-impulsive) regime is approached. A clear spatial attenuation of pressures along the caisson is observed, with peak indicators decreasing systematically from the seaward to the landward cell, consistent with progressive dissipation of the overtopping-jet energy. In addition CFD simulations reproduce the stationary regime and the spatial pattern of pressures peaks reasonably well.

Overall, the thesis contributes to the design and adaptation of vertical breakwaters by (i) clarifying mechanisms and limits of wave wall retreat; (ii) developing a framework based on the regime and on the process for post-overtopping impacts, and introducing a parametric map to classify event types and support preliminary estimation of loads, useful as designing; (iii) quantifying the impact of phase-seed stochasticity on experimental repeatability and on the estimation of measured quantities; and (iv) highlighting and characterizing a construction phase vulnerability, supporting future developments towards more robust guidance.

Chapter 1

Introduction and state of the art

1.1 Problem statement

Coastal regions around the world face a complex series of natural and anthropogenic challenges that threaten their long-term stability and viability. Traditionally, the design of coastal and offshore structures, including harbor breakwaters, seawalls, dikes, and offshore platforms, has relied on wave climate datasets and empirical design rules derived from historical observations. These criteria were largely formulated under assumptions of relatively milder wave conditions and lower sea levels. Although such approaches have proven effective for many decades, they are being tested by rapidly changing environmental forcing, such as accelerated sea level rise, evolving storm patterns, and an expanded range of extreme wave events (Mori et al., 2010; Vousdoukas et al., 2016, 2017, 2018b).

According to major assessments by the Intergovernmental Panel on Climate Change (IPCC), notably the “Special Report on the Ocean and Cryosphere in a Changing Climate” (IPCC, 2019) and the “Sixth Assessment Report” (IPCC, 2021, 2022, 2023), sea levels are rising faster than earlier projections indicated, while extreme wave events and storm surges are also increasing in both frequency and magnitude. Figure 1.1 shows the plausible range of global mean sea level rise projections by 2150 under various emission scenarios, as assessed in the IPCC AR6 report (IPCC, 2021). These evolving hazards pose a significant risk to aging or under designed infrastructure, heightening the probability of structural failure in coastal defence systems (Camus et al., 2017, 2019).

Projected global mean sea level rise under different SSP scenarios

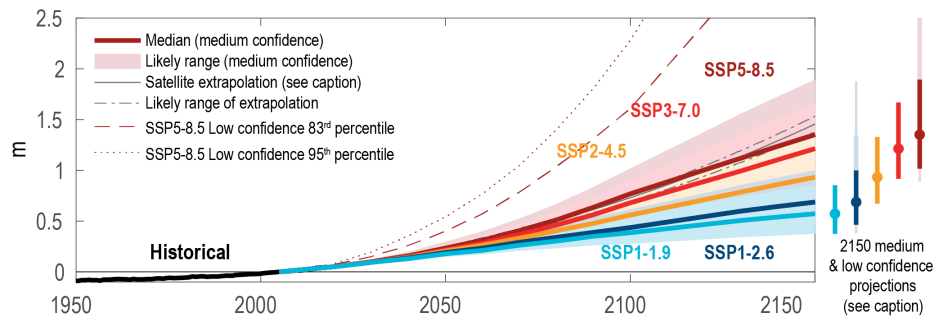


Figure 1.1: Projected global mean sea level rise under different Shared Socio-economic Pathway (SSP) scenarios. Likely global mean sea level (GMSL) change for SSP scenarios resulting from processes in whose projection there is medium confidence. Projections and likely ranges at 2150 are shown on right. Lightly shaded ranges and thinner lightly shaded ranges on the right show the 17th-83rd and 5th-95th percentile ranges for projections including low confidence processes for SSP1-2.6 and SSP5-8.5 only, derived from a p-box including structured expert judgement and marine ice-cliff instability projections. Black lines show historical GMSL change, and thick solid and dash-dotted black lines show the mean and likely range extrapolating the 1993-2018 satellite altimeter trend and acceleration. *Source: Figure 9.27 in IPCC (2021): Chapter 9. In: Climate Change 2021: The Physical Science Basis. Contribution of Working Group I to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change. Used in accordance with IPCC policy.*

Given these scenarios, one of the most pressing concerns for coastal engineering are wave loads and overtopping, the phenomenon in which waves exceed a structure's crest and spill landward. Overtopping volumes and impact forces can surpass original design thresholds, threatening public safety, damaging infrastructure, and imposing unexpected maintenance costs (Esteban et al., 2013; Harrison et al., 2015; Sweet et al., 2018; McAlpine and Porter, 2018). Moreover, ongoing and future climate shifts demand extended service life and greater safety margins than those considered in earlier designs. For example, EurOtop (2018) provides updated guidance on overtopping discharge limits and highlights more stringent requirements for structural stability under extreme wave conditions, reflecting the urgency to account for uncertain future climates and longer design lifespans.

Consequently, many existing coastal defences, often built decades ago, no longer align with modern design demands or anticipated wave climates

(Vousdoukas et al., 2018b). Retrofitting or replacing these structures to withstand high hydrodynamic loads requires robust analyses of wave forces and pressures, wave run-up, wave reflection, turbulent processes, and energy dissipation. Engineers and researchers thus face a twofold challenge:

1. Diagnosing the limitations of current defences, which may suffer from inadequate crest height, insufficient armouring, or outdated design assumptions.
2. Developing cost-effective, resilient solutions, whether through incremental upgrades (e.g. raising crest elevations) or comprehensive re-design (e.g. designing entirely new structural concepts).

Balancing economic viability with long-term resilience is essential. Advanced physical modelling, numerical modelling (e.g. computational fluid dynamics simulations), and analytical approaches facilitate deeper insights into wave-structure interactions. Meanwhile, ongoing innovations in structural optimization target a wide spectrum of designs, from rubble mound breakwaters and vertical caissons to hybrid “green-gray” solutions, aiming to reduce wave forces, limit overtopping discharges, and enhance overall stability under more severe hydraulic conditions (Camus et al., 2019). This growing body of research underscores the critical role of coastal engineering in safeguarding populations, infrastructure, and economic interests against escalating climate pressures.

1.2 Climate change and economic impact

1.2.1 Rising hazards and increasing vulnerability

Climate change is having pronounced effects on coastal zones, which are home to dense populations, key economic activities, and ecologically rich environments (Mehvar et al., 2018). In Europe alone, more than 200 million people reside close to the sea, a figure expected to rise as urban development and tourism continue to flourish in coastal areas (Neumann et al., 2015; Jones and O’Neill, 2016). These demographic and socioeconomic trends make coastal regions especially vulnerable to threats such as sea level rise, storm surge, and intensified wave climates (Barnard et al., 2015; Vitousek et al., 2017).

Scientific evidence indicates that these extreme events and climate stress factors will persist, and potentially intensify, given ongoing greenhouse gas emissions (Mori et al., 2010; Woodworth et al., 2011; Muis et al., 2016; Wahl et al., 2017; Lee et al., 2017; Vousdoukas et al., 2016, 2017, 2018b). As sea levels rise, even modest storms can produce coastal flooding events of disproportionate severity. For example, recent estimates suggest that average annual flood losses in Europe already amount to 1.4 billion euros, affecting around 100,000 people each year (Vousdoukas et al., 2018a). By 2100, in the absence of adaptation measures, these figures could escalate dramatically surpassing 200 billion euros under moderate emission scenarios (i.e. RCP4.5 from IPCC (2014)) and exceeding 1,200 billion euros under high emission pathways (i.e. RCP8.5 from IPCC (2014)). A spatial representation of these projected adaptation costs across European regions is shown in Figure 1.2, highlighting the pronounced regional disparities in expected economic impacts under two different emission scenarios, these data are reported in Vousdoukas et al. (2020a) and in the EU JRC technical report Vousdoukas et al. (2020b). Beyond direct flood damages, knock-on effects such as erosion, saltwater intrusion, and habitat loss pose broader challenges for coastal landscapes, economies, and livelihoods (Nicholls, 2007; Hinkel et al., 2014).

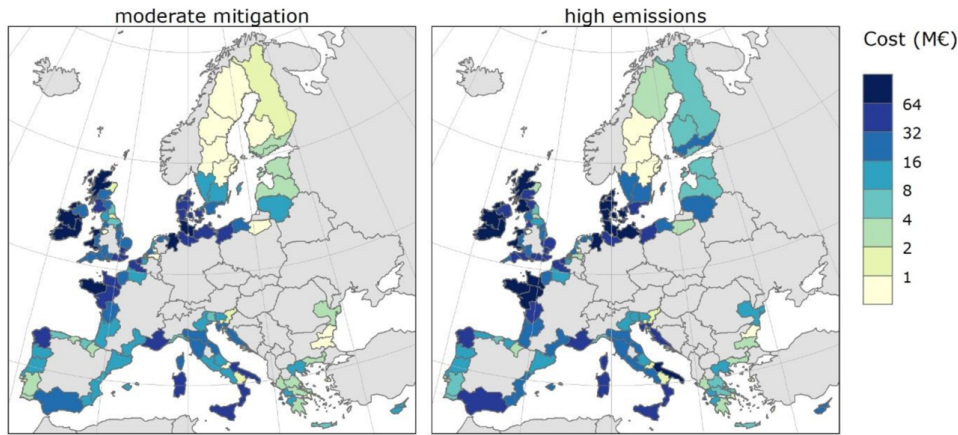


Figure 1.2: Average annual costs of adaptation (in €million/year; not discounted and averaged over period 2020-2100) per NUTS2 (Nomenclature of Territorial Units for Statistics) region under a moderate mitigation and high emissions scenario. *Source: Figure 4 in Voudoukas et al. (2020b) JRC PESETA IV project - Task 6. Used in accordance with the license: Creative Commons Attribution 4.0 International (CC BY 4.0)*

1.2.2 Infrastructure pressures and adaptation measures

Many existing maritime works and structures (i.e. dikes, seawalls, breakwaters, and harbor facilities) were not originally designed to accommodate the new or expected hydrodynamic loads associated with climate change (Camus et al., 2017, 2019). Even a slight rise in mean sea level can magnify wave run-up and overtopping rates, exerting continuous stress on aging structures. Costly retrofits, such as raising crest heights or reinforcing armour layers, are thus often necessary to maintain adequate levels of protection (Sweet et al., 2018; McAlpine and Porter, 2018; Esteban et al., 2013; Harrison et al., 2015). However, large-scale overhauls can be financially prohibitive, especially in regions with limited economic resources.

In the engineering and policy literature, adaptation strategies are commonly categorized into “hard” measures (e.g., seawalls, surge barriers, dikes) and “soft” measures (e.g., dune restoration, beach nourishment, wetland conservation). These terms, “hard” and “soft”, have been widely used in climate adaptation discourses to distinguish between built, largely infrastructural solutions and approaches that leverage natural ecosystems (Barbier et al., 2011; IPCC, 2014; Sayers et al., 2015; IPCC, 2019). A growing subset of “green-gray” or hybrid infrastructure combines engineered elements with

ecological components-such as mangroves or coral reefs, to dampen wave energy while providing co-benefits for biodiversity and carbon sequestration (Spalding et al., 2014; Van der Meer and Sigurdarson, 2016).

Regardless of the chosen adaptation path, cost-benefit analyses are necessary to weigh immediate expenditures against long-term resilience. Numerous studies highlight the importance of integrating climate scenarios into port and harbor design and coastal management plans (Townend and Burgess, 2005; Okayasu and Sakai, 2007; Olabarrieta et al., 2009; Chini and Stansby, 2012; McEvoy and Mullett, 2013; Suh et al., 2013; Sánchez-Arcilla et al., 2016; Koftis et al., 2015; Galiatsatou et al., 2018). While progress has been made, uncertainties related to storm intensity, wave transformation, shoreline morphodynamics, and sediment transport demand continued research to refine existing standards and guidelines and ensure that future coastal defences are both robust and cost effective.

1.2.3 Regional variations in coastal adaptation

Although awareness of these risks is increasing, the level of preparation still varies considerably across different regions. Wealthier nations often have the financial resources and technological capacity to retrofit or replace outdated infrastructure, while lower, and middle, income countries may struggle to secure funding for even basic adaptation measures (IPCC, 2014, 2019). This discrepancy can deepen existing social inequalities: communities with limited economic means tend to be more exposed to climate hazards and less able to invest in protective structures or relocate to safer locations.

The situation is further complicated by the loss or degradation of coastal ecosystems (such as coral reefs, mangroves, and wetlands), which provide natural buffering against storm waves and contribute to local economies through fisheries and tourism (Barbier et al., 2011; Ferrario et al., 2014; Spalding et al., 2014). When these habitats are compromised, wave attenuation decreases, and the overall risk profile for coastal communities increases. In such contexts, strategic investment in eco-engineering and nature-based solutions can simultaneously address hazard mitigation and ecosystem preservation, although high implementation costs and limited governance capacity often pose significant challenges.

Nevertheless, there are some indications for cautious optimism. Inter-

national partnerships and the exchange of technical expertise can encourage more integrated and effective solutions, even in regions with limited economic resources (IPCC, 2014, 2019). Advances in numerical modelling, scaled physical experiments, and improved sea level rise datasets make it possible to refine risk assessments and support the development of scientifically grounded policies and engineering designs. By leveraging these approaches in a coordinated manner, coastal adaptation strategies can better address the complex challenges posed by a changing climate.

1.3 State of the art: optimization of coastal structures

As mentioned in the previous paragraphs, the growing need to safeguard coastal communities and infrastructures from changing wave and storm conditions, including region-dependent variations in magnitude and frequency associated with climate change, has led to extensive research on optimizing maritime and coastal structures (IPCC, 2019, 2021, 2022, 2023; Camus et al., 2017, 2019). Traditional designs often rely on substantial concrete volumes, massive armour units, or elevated crests to provide protection against wave action. While these methods can be effective, they are not always cost efficient or environmentally sustainable, especially given the uncertainties associated with future climate forcing. As a result, a range of innovative design improvements and retrofits has emerged to enhance hydraulic performance (e.g., reduce: wave overtopping, wave forces, and wave reflection), improve structural stability, and minimize environmental impacts.

This section presents a non-exhaustive overview of key optimization strategies and then surveys the current state of the art for each major type of coastal structure: rubble mound breakwaters, vertical breakwaters, dikes and seawalls, floating breakwaters, eco-engineering adaptations and the integration of wave energy converter. To provide a quick overview of these coastal structures, Figure 1.3 presents representative photographs illustrating different types of maritime defences commonly used.

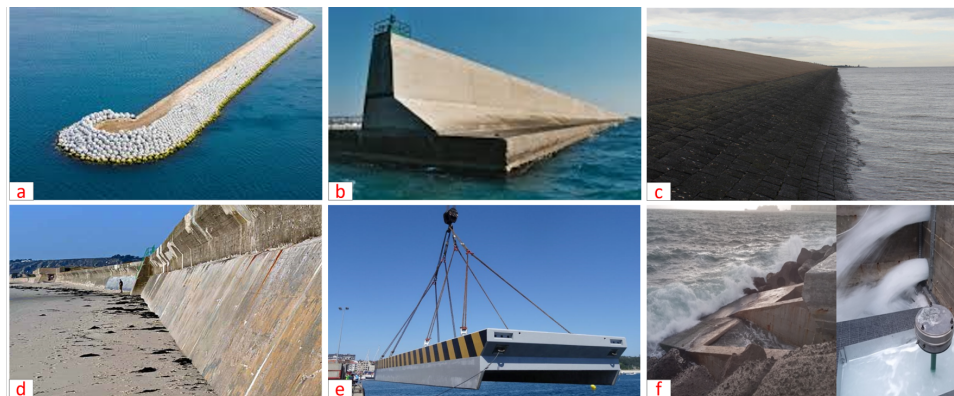


Figure 1.3: Example of common coastal structures: rubble mound breakwater (panel *a*), vertical breakwater (panel *b*), sea dike (panel *c*), seawall (panel *d*), floating breakwater (panel *e*), wave energy converter inside a rubble mound breakwater (panel *f*).

Since a significant part of this thesis focuses on the **optimization of vertical breakwaters with retreated wave walls** in relatively deep water conditions, after the following general overview, Section 1.4 provides a description of this particular configuration.

1.3.1 Overview of optimization strategies

Coastal infrastructure must fulfill multiple objectives under increasingly adverse hydraulic conditions, including:

1. **Reducing Wave Forces and Pressures:** Wave impact forces can lead to structural instability, causing cracking, sliding, or overturning. Approaches to mitigate forces include perforated caissons, submerged breakwaters, or modified crown walls designed to dissipate energy by redirecting or breaking incoming waves.
2. **Minimizing Wave Overtopping:** Overtopping volumes can exceed acceptable safety thresholds when crest elevations are not sufficiently high or when structural geometries are suboptimal for dissipating wave energy. Optimizations such as bullnoses, recurved parapets, overhangs, or stepped profiles have demonstrated the capacity to divert or reduce incoming wave energy and thereby lower overtopping rates.
3. **Reducing Wave Reflection:** Excessive wave reflection can amplify standing wave patterns seaward of the structure, adversely affecting navigation, adjacent coastal sectors, and sometimes generating scouring at the toe. Perforations or porous materials (e.g., rock layers, geotextile sand containers) enable partial transmission of wave energy, leading to enhanced dissipation.
4. **Improving Stability:** Suboptimal slope angles, inadequate toe protection, or insufficient foundation layers may render structures susceptible to sliding or toppling under large waves. Structural modifications, such as berms, reinforced toes, and carefully weighted armour layers, can enhance overall stability.
5. **Mitigating Environmental Impacts:** Conventional hard structures can disrupt local sediment transport and ecological connectivity. In response, low-impact or hybrid solutions aim to preserve or restore natural features (e.g., dunes, wetlands) while still providing sufficient protection.

To achieve these optimizations, engineers rely on various methods such as physical, numerical, and analytical modelling. Physical modelling involves constructing scaled-down versions of structures in wave flumes or tanks to simulate and test performance under controlled wave conditions. Numerical modelling, using sophisticated computational tools, allows to simulate wave-structure interactions digitally, providing useful insights on the hydraulic performances. Analytical models offer predictive capabilities, helping engineers optimize designs based on theoretical calculations. Additionally, real-time field monitoring of existing structures provides valuable feedback that can inform and enhance future design efforts.

1.3.2 Rubble mound breakwaters

Rubble mound breakwaters have a longstanding history, dating back millennia to early civilizations that recognized the need to safeguard coastal regions and harbor basins against wave action. One of the earliest documented examples, the Phoenician breakwater at Atlit in Israel, is believed to have been constructed in the ninth or early eighth century BC (Haggi, 2010). Over the centuries, the Egyptians, Minoans, Phoenicians, Carthaginians, Greeks, Etruscans, and Romans developed and refined such structures, relying on permeable, rough slopes composed of carefully placed stones to absorb and dissipate wave energy (Franco, 1996). This traditional approach, where waves lose energy through percolation, friction, and partial breaking across multiple layers of armour stones, remains a cornerstone of modern coastal engineering.

The fundamental design principle of rubble mound breakwaters relies on the careful selection of stone size in relation to the severity of the wave climate (Hudson, 1959; Van der Meer, 1988). In open waters exposed to strong storms and large waves, larger stones or specially designed artificial armour units are required to withstand significant hydrodynamic forces, whereas milder conditions demand smaller units. Modern research and design methodologies focus on ensuring both hydraulic performance and structural integrity, as evidenced by extensive guidelines and manuals such as those by Burcharth (1994); Van der Meer (1995); USACE (2002); CIRIA et al. (2007); Goda (2010); EurOtop (2018). Several large-scale research initiatives, including the Environmental Design of Low Crested Coastal Defence Structures (DELOS) and the Crest Level Assessment of Coastal Structures (CLASH), have further advanced the theoretical understanding and

practical design of rubble mound breakwaters, providing insight into their performance under varied wave and storm conditions (De Rouck et al., 2004; Airolidi et al., 2005; De Rouck et al., 2009; van der Meer et al., 2009; Hawkins et al., 2010).

Contemporary innovation in rubble mound breakwater design centers on the development of advanced armour units and the optimization of the cross section to improve wave dissipation, reduce overtopping, and minimize reflection. A variety of artificial units (e.g. Tetrapods, Antifers, Accropode, Dolosse, X-blocs, etc.) offer enhanced interlocking characteristics, which optimize hydraulic stability while reducing the overall volume of stone required. Recent iterations such as the single layer Cubipod, Accropode II, and Core-Loc have refined these principles by further improving the balance between material economy and energy dissipation (Burcharth et al., 2014). In particular, in Burcharth et al. (2014) several methodological approaches are presented for upgrading coastal defence structures to withstand the increased loads associated with climate change. A first category includes interventions that allow an increase in crest elevation. In these cases, reinforcement of the armour layer and reduction of overtopping are achieved by adding an additional layer of armour units on the seaward slope and on the crest. The resulting increase in pore volume enhances the stability of the armour and helps reduce run-up and overtopping. Examples are also provided in which a new concrete superstructure is added, or an existing one is raised and strengthened. The second category includes solutions that do not involve modifications to the crest level. In these configurations, armour units are added exclusively to the seaward face in order to flatten the slope or to create a berm. Both strategies contribute to reducing overtopping and improving armour stability. Additional alternatives include the installation of a detached breakwater, a forebay in front of the structure, or an artificial submerged reef, all designed to lessen wave energy before it reaches the existing defence. Figure 1.4, adapted from Burcharth et al. (2014), provides a conceptual overview of structural adaptation options that preserve the crest level, achieved through front-slope reinforcement, berm formation, and the implementation of external wave-dissipating systems designed to decrease wave loading on the original armour layer. Another key area of progress involves crest configuration, where alterations such as wave-return parapets or bullnoses help control overtopping volumes without compromising structural robustness (Martin et al., 1999; Oh et al., 2017; Molines et al., 2019). Further enhancements to existing rubble mound breakwaters often include the addition of extra armour layers, particularly to adapt to more energetic

future wave climates or to correct for natural settlement over time, which has proven highly efficient in reducing overtopping risks (Burcharth et al., 2014; Cecioni et al., 2019; Pepi et al., 2022).

A further effective solution is the use of a berm breakwater, that is typically a rubble mound structure featuring one or more near-horizontal ledges (the berm), designed to dynamically adjust under wave action and enhance overall stability. Such structures have gained significant attention for their effectiveness in reducing run-up and reflection by dispersing wave energy under variable hydrodynamic conditions, making them a particularly effective and low-cost option for shoreline protection (Lykke Andersen, 2006; Sigurdarson and Van der Meer, 2012; Burcharth et al., 2014; Van der Meer and Sigurdarson, 2016).

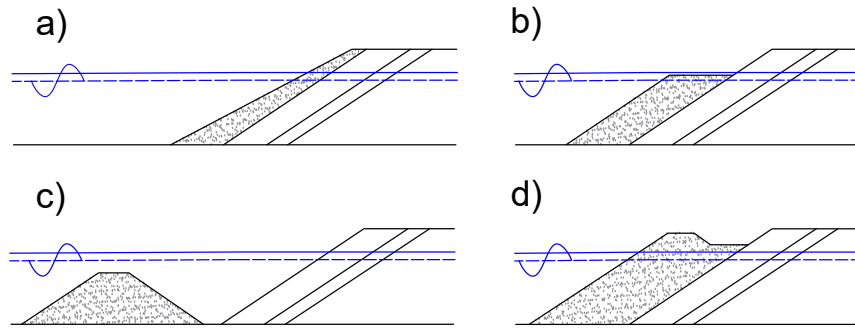


Figure 1.4: Conceptual overview of structural adaptation options that preserve the crest level (adapted from Burcharth et al., 2014).

Additionally, a particularly effective strategy for mitigating wave action, preventing shoreline erosion, and reducing flood risk is the implementation of stilling wave basins and overspill basins atop breakwaters. These basins temporarily collect wave run-up and regulate the return flow, significantly reducing wave overtopping and dissipating excess energy. By moderating hydrodynamic forces, they help to reduce the impact on the protected side of the structure. Additionally, they are often integrated with promenade features, enhancing both hydraulic performance and providing recreational benefits (Geeraerts et al., 2007; Cappietti and Aminti, 2012; Grossi et al., 2017; Kisacik et al., 2019). Figure 1.5 shows a conceptual sketch of a stilling wave basin.

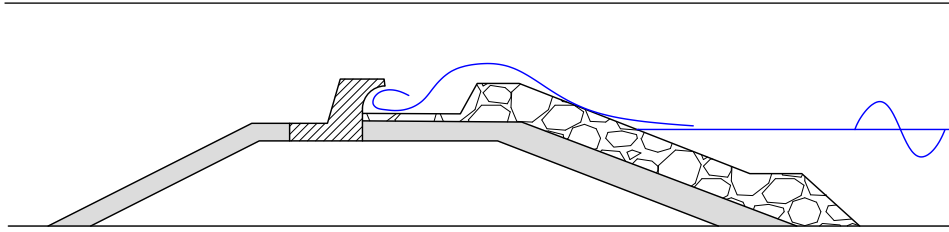


Figure 1.5: Conceptual sketch of stilling wave basin on a sloping structure (adapted from Grossi et al., 2017).

1.3.3 Vertical breakwaters

Vertical breakwaters represent a longstanding and continuously evolving solution in coastal engineering, tracing their origins back to the Roman era, when the mastery of hydraulic cement made with *pozzolana* (volcanic ash binder for masonry) enabled the construction of highly durable underwater walls. These early achievements, manifesting as *opus pilarum*, sometimes replaced the traditional Phoenician/Greek rubble mound breakwaters due to their ability to withstand harsh marine environments (Franco, 1996). Modern advancements build upon these historical precedents and are characterized by the use of prefabricated monolithic cellular reinforced concrete caissons. These large caissons, first used by the Allies to build the temporary “Mulberry” ports during the 1944 Normandy landings in World War II (Harris, 1989), are typically floated to the project site, then ballasted with seawater and filled with sand or concrete once in position (Franco, 1994).

Compared to rubble mound solutions, vertical or caisson-type breakwaters are often preferable in deeper waters, where the volume of rock required for conventional rubble mounds can become prohibitively expensive. While rubble mound breakwaters rely largely on the mass of stone for stability, vertical breakwaters depend on the structural integrity of the caisson and a prepared foundation, which may be composed of a rubble bedding layer or a dredged seabed. This design can significantly reduce the spatial footprint of the structure, offering a potentially more economical and compact choice for deep water applications (Oumeraci et al., 2001). However, the success of vertical breakwaters hinges on managing wave reflection, overtopping, and slamming forces, which can be extremely high when waves break directly on the vertical face.

The quest to refine the design approaches for vertical breakwaters has been notably propelled by Japanese and European experiences, with an extensive body of field data and applied research contributing to more precise

predictive models. Among these, a central aspect involves calculating the wave-induced forces, often using the well-established Goda formulas, originally developed in the 1970s and later refined to accommodate different wave conditions (i.e., non-breaking, breaking, or broken) and structural configurations (Goda, 1974; Takahashi, 1996; Goda, 2010).

A major consideration in vertical breakwater design is the significant reflectivity of their upright walls, which can send a substantial portion of incident wave energy back to the sea. This reflexivity, while beneficial for reducing wave transmission into the harbor area, can result in pronounced wave run-up and intense localized pressures on the structure itself, especially under storm conditions (Calabrese and Vicinanza, 1999). Building on these theoretical advancements, efforts to further quantify wave-structure interactions led to the development of probabilistic design frameworks such as the PROVERBS (Probabilistic Design Tools for Vertical Breakwaters) research project (Oumeraci et al., 2001). This initiative introduced a parameter decision map to guide engineers in identifying critical loading conditions based on a combination of dimensionless parameters related to the breakwater geometry, water depth, and wave climate. A key outcome was a clearer methodology for discerning whether conditions aligned with non-breaking, breaking, or broken wave loads on the structure's front wall. These guidelines also include methods to estimate wave impact magnitude and duration on vertical breakwaters, providing a foundation for improved stability assessments. Further studies, such as those by Martinelli and Lamberti (2011), have refined these assessments by analyzing the dynamic response of caisson breakwaters and proposing equivalent static analysis methods to evaluate their stability under wave impact.

Ongoing optimization strategies for vertical breakwaters have focused on mitigating high reflection and impulsive slamming pressures. Early work by (Jarlan, 1961) and subsequent studies by (Kondo, 1979; Huang, 2006) demonstrated that perforated or cellular caissons can attenuate wave energy through controlled flow into internal chambers, thereby reducing peak dynamic loads and limiting the severity of reflection. Multiple or stepped chamber designs have further evolved this concept by diffusing wave energy across several compartments, a development confirmed by experimental campaigns showing decreased dynamic pressures on the rear walls under both regular and irregular wave conditions (Belorgey et al., 2000; Bergmann and Oumeraci, 2001). Recent studies, such as those on the ARPEC breakwater, have shown that staggered perforations on both external and internal

walls can further improve wave energy dissipation and water circulation, reducing reflection while enhancing harbor conditions (Sammarco et al., 2021). The role of multi-chamber configurations in enhancing dissipation has also been analyzed in previous research (Gonzalez-Escriba et al., 2018; Franco et al., 2019), showing that openings at transverse and internal walls can lengthen the wave dissipation path and improve hydraulic efficiency. Additionally, experimental and numerical studies have confirmed that varying the porosity and chamber layout of perforated caissons significantly influences their reflection and transmission coefficients, providing a basis for further optimization in deep water applications (Bergmann and Oumeraci, 2001; Suh, 2010).

Another approach to mitigating wave forces on vertical breakwaters involves geometric modifications to the crown walls and parapets, such as retreated or recurved configurations, which can influence wave overtopping and impact loads. Some studies have analyzed the hydrodynamic effects of different configurations, providing experimental and numerical insights into their effectiveness. Kortenhaus et al. (2002, 2003) investigated the influence of recurves and parapets on vertical walls, focusing on their effectiveness in reducing overtopping rates without excessively increasing crest height. They proposed design guidance and reduction factors for overtopping as a function of parapet geometry and discussed the associated increases in wave loading under different structural configurations. Pearson et al. (2005) further examined the interaction between breaking waves and parapets, emphasizing that the curvature and elevation of the parapet significantly affect the load distribution along the wall. Kisacik et al. (2012, 2014) provided detailed experimental evidence on the role of aeration and wave impact dynamics, showing that overhanging horizontal cantilever slab influence the air entrapment within the wave crest, which in turn affects the pressure distribution. Stagonas et al. (2014) investigated the pressures and loads generated by breaking waves in different recurve parapet geometries, demonstrating how the seaward extension of the superstructure and the incoming wave conditions influence the spatial pressure distribution and overall loading, with substantial variability observed even between similar wave events. Martinelli et al. (2018) studied wave forces and overtopping at the recurved parapets on vertical breakwater in deep water, under nonbreaking conditions by incorporating numerical simulations into physical model tests. Also Castellino et al. (2018, 2021) focused on recurved parapets on vertical breakwater in deep water, quantifying the extent to which different curvature profiles influence overtopping rates and wave-induced forces. These studies provided criti-

cal design recommendations regarding parapet height and curvature radius. de Almeida and Hofland (2020) investigated the applicability of pressure-impulse theory for predicting wave impact magnitudes on vertical hydraulic structures with overhangs, validating their semi-analytical model against laboratory measurements and demonstrating its accuracy in capturing the overall distribution of pressure-impulses and force-impulses under standing wave conditions. Ravindar et al. (2021) investigated the impact of breaking waves on vertical walls with different recurved parapets at two different scales, proposing a method to better predict impact forces under both pulsating and impulsive conditions. More recently, Romano and Bellotti (2023) investigated wave-induced forces on vertical caissons with a retreated crown wave wall through physical model tests, showing that shifting the wave wall backward can significantly reduce global wave loads on the whole structure, especially under high-energy sea states, although it can also generate impulsive loads on the wall itself depending on the retreat distance. Figure 1.6 show a sketch of some different configurations of vertical caisson breakwater.

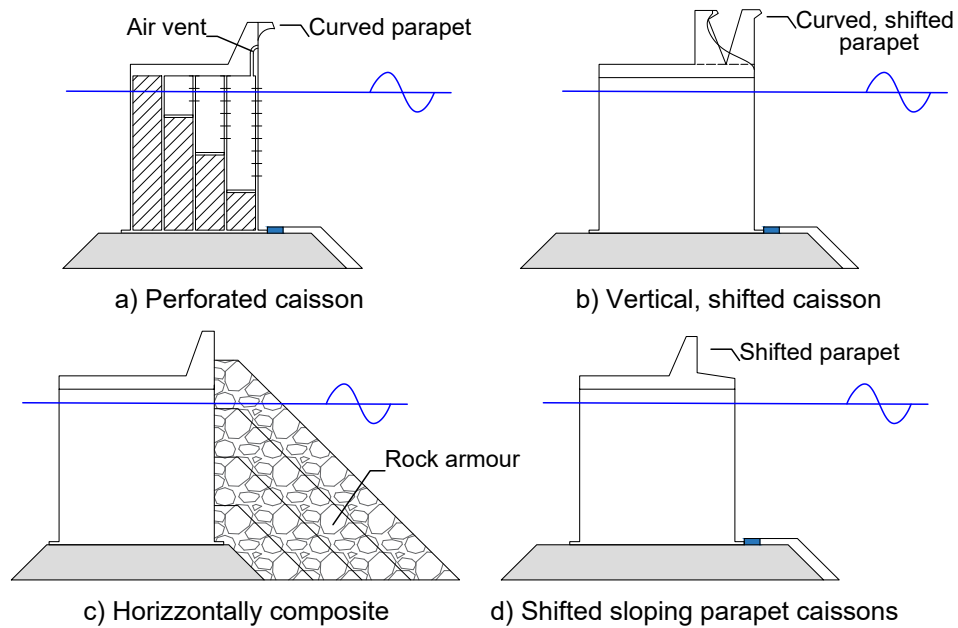


Figure 1.6: Sketch of different configurations of vertical caisson breakwater (adapted from Franco, 1994).

1.3.4 Dikes and seawalls

Unlike most Mediterranean countries, which typically have narrow foreshore widths, Northern European countries experience large tidal ranges and often feature broad, shallow foreshores where sea dikes and seawalls are constructed at the landward end (Veale et al., 2012). In these environments, dikes and seawalls have long been among the most prominent coastal defence structures, developed over centuries to protect low-lying coastal areas from wave action, tidal forces, and storm surge-induced overtopping. Some of the earliest examples date back to medieval embankments built from earthen materials and rudimentary revetments, underscoring the enduring need for effective coastal flood protection (van de Ven, 1993). In modern engineering practice, these structures are typically designed as clay-core embankments covered with concrete slabs or armoured revetments on the seaward side. Alternatively, reinforced concrete seawalls are employed where spatial constraints or high hydrodynamic loads require a compact and rigid solution. While conceptually simpler than rubble mound or vertical breakwaters, dikes and seawalls have undergone significant optimization to meet the increasing demands of contemporary coastal engineering, particularly in response to rising sea levels, more extreme storm conditions, and increasingly stringent overtopping and stability requirements. A fundamental design criterion for dikes and seawalls is the control of overtopping discharge to ensure the safety of hinterland infrastructure and human activities. Key hydraulic processes, including wave run-up, wave breaking on the seaward slope, and flow along the crest, must be rigorously analyzed in both conceptual and detailed design phases (USACE, 2002; CIRIA et al., 2007; EurOtop, 2018).

Recent optimization strategies aimed at reducing overtopping have drawn from design principles applied in rubble mounds and vertical breakwaters, adapting them to the specific characteristics of dikes and seawalls. Given that these structures are often built along broad foreshores, parapets and wave walls are commonly set back from the shoreline, which modifies the incident wave conditions, promotes early wave breaking, and reduces overtopping rates (Altomare et al., 2016). In particular, extensive research has investigated overtopping and the associated hydrodynamic forces and pressure distributions on retreated wave walls applied to smooth-sloped dikes, demonstrating that the effectiveness of this approach depends strongly on the wave climate and structural geometry. Early experimental work by van Gent (2003) on wave forces acting on smooth slopes identified key parameters influencing wave dissipation and load reduction on the crest. Building

on this, large-scale physical tests and numerical simulations by De Rouck et al. (2012) and Chen et al. (2015) refined these findings, showing that retreated wave walls can effectively dissipate wave energy and mitigate loads. Subsequent research has focused on optimizing the placement of retreated wave walls relative to the dike crest to achieve optimal overtopping control under different hydrodynamic conditions. Van Doorslaer et al. (2017) analyzed the role of set back distance, demonstrating that properly configured walls can significantly reduce overtopping discharge without excessive increases in crest elevation. De Finis et al. (2020) examined turbulent flow fields associated with post-overtopping waves on retreated walls, showing that strategic positioning can reduce dynamic impact pressures. More recent studies (Cao et al., 2021; Chen et al., 2021) have extended these findings by examining wave overtopping on sloped seawalls and its effects on pedestrians. They derived empirical formulas for overtopping depth and forces, showing that both decrease gradually with distance from the seawall edge, which underscores the need for effective overtopping control to ensure safety.

Another widely investigated optimization strategy is the integration of stepped (i.e. terraced) or roughened slopes, which enhance turbulence and promote wave energy dissipation along the seaward face of the structure, thereby reducing wave run-up. In particular, stepped revetments have proven effective at limiting wave overtopping by increasing surface roughness, disrupting flow patterns, and dissipating energy before the wave reaches the crest. Both laboratory experiments and field observations confirm that these adaptations can significantly decrease wave velocities, run-up heights, and overtopping volumes compared to smooth-sloped structures (McCabe et al., 2013; Van Steeg et al., 2018; Kerpen et al., 2019; Schoonees et al., 2021; Herbst et al., 2025). Figure 1.7 show a sketch of the integration of a stepped revetments on a dike.

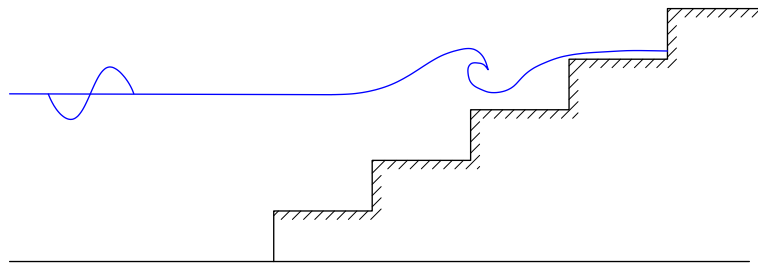


Figure 1.7: Sketch of the integration of a stepped revetments on a dike (adapted from Herbst et al., 2025).

Another structural modification adopted to reduce overtopping on sea dikes, similarly to rubble mound breakwaters, is the use of stilling wave basins. These recessed basins dissipate wave energy lowering run-up and overtopping discharge (Geeraerts et al., 2007; Altomare et al., 2014; Kisacik et al., 2019). Similar to crown walls in vertical breakwaters, crest parapets and bullnoses are often added to the top of dikes and seawalls to deflect wave crests and jet flows, reducing the volume of water overtopping the structure. Despite their relatively small size, these structural features have been shown to significantly decrease overtopping rates while introducing minimal additional height to the primary embankment. However, their geometric design must be carefully optimized, as certain configurations can amplify localized impact pressures (Kortenhaus et al., 2003; Pearson et al., 2005; Kisacik et al., 2012, 2014; Stagonas et al., 2014; de Almeida and Hofland, 2020; Ravindar et al., 2021). Figure 1.8 shows a conceptual sketch of a dike, in Panel a) with different geometric modifications (e.g. berm, wave wall, bullnose, etc.), and in Panel b) shows the application of a stilling wave basin.

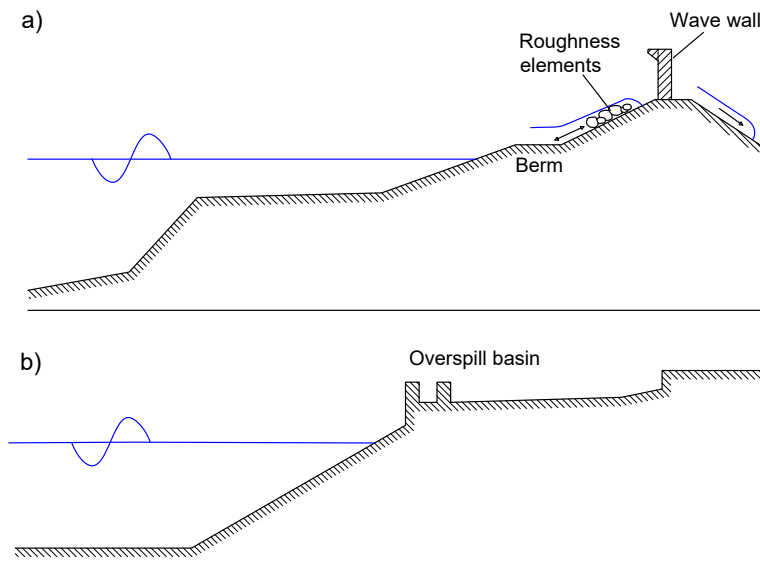


Figure 1.8: Conceptual sketch of a dike, in Panel a) with different geometric modifications (e.g. berm, wave wall, bullnose...), and in Panel b) the application of a stilling wave basin (adapted from EurOtop, 2018).

1.3.5 Floating breakwaters

Floating breakwaters have emerged as a viable and sustainable solution for coastal and harbor protection, particularly in regions characterized by deep waters or unfavorable seabed conditions where conventional rubble-mound

or caisson breakwaters become economically or technically unfeasible. According to Dai et al. (2018), floating breakwaters can be broadly categorized into seven principal types: box-type, pontoon-type, frame-type, mat-type, tethered float type, horizontal plate type, and several hybrid or innovative configurations. Their effectiveness is typically evaluated through the wave transmission coefficient, defined as the ratio between transmitted and incident wave heights, which serves as a key indicator of performance. The box-type and pontoon-type breakwaters are the most extensively investigated and practically implemented configurations due to their simple geometry, high structural stability, and efficiency in reflecting incoming wave energy (Carr, 1950; Macagno, 1953; Bottin and Turner, 1980; McCartney, 1985; Williams et al., 2000; Dong et al., 2008; Diamantoulaki and Angelides, 2010). While box-type structures primarily rely on wave reflection, pontoon-type systems improve hydrodynamic stability and energy dissipation through interference and turbulence in the inter-pontoon spacing (Dai et al., 2018). Frame-type breakwaters, combining pontoons and truss elements, further enhance dissipation by promoting flow turbulence and partial wave breaking (Bottin and Turner, 1980; Mani, 1991; Wu et al., 2001; Uzaki et al., 2011).

Mat-type breakwaters, often constructed using recycled materials such as scrap tires, bamboo, or PVC tubes, attenuate waves mainly through frictional resistance along the wetted surfaces and by disturbing the orbital motion of water particles (Hegde et al., 2007). These systems are cost-effective and environmentally advantageous but typically require a large plan area, with widths comparable to the incident wavelength, to achieve significant attenuation. The tethered float type, originally proposed by Jones (1978); Hales (1981), employs arrays of spherical or cylindrical floaters moored to the seabed and dissipates wave energy through viscous drag induced by the oscillatory motion of the floaters (Vethamony, 1995). Horizontal plate-type breakwaters, composed of single or multiple rigid or flexible plates, operate by suppressing the vertical motion of water particles and by inducing wave fission and phase cancellation; their performance strongly depends on plate stiffness, submergence depth, and relative width (Cheong and Patarapanich, 1992; Koley and Sahoo, 2017).

In addition to these conventional configurations, several innovative and hybrid systems exist, such as the resonance-type breakwaters allowing internal water motion to dissipate energy efficiently under long waves (Ikegami et al., 1994), porous and modular HDPE floating structures (Wang and Sun,

2010), and designs incorporating pneumatic chambers or oscillating water columns (He et al., 2019; Hong et al., 2006) that enable simultaneous wave attenuation and potential energy harvesting.

Despite their numerous advantages, including modularity, ease of installation, adaptability to varying sea levels, and reduced environmental impact, floating breakwaters remain less effective in attenuating long-period waves, and their mooring systems are susceptible to dynamic loads under severe sea states (Macagno, 1953; McCartney, 1985; Martinelli et al., 2008; Ruol and Martinelli, 2009). Mooring design is therefore paramount to ensuring both wave attenuation efficiency and overall stability. Catenary moorings generally allow greater freedom of motion, which can be beneficial in dissipating energy but may lead to large excursions under extreme conditions (Liu, 1973; Huang and Vassalos, 1993). Taut or semi-taut moorings offer improved station-keeping but can introduce high structural loads, necessitating robust materials and careful stress analyses. Hybrid systems that combine elements of both approaches have been proposed to strike a balance between energy dissipation and reduced peak stresses (Diamantoulaki and Angelides, 2010). Advances in hydro-elastic modelling, computational fluid dynamics, and real-time mooring simulations now enable more precise assessment of floating breakwater behavior under diverse wave climates, thereby facilitating optimization of design parameters (Maeda et al., 2000; Hong et al., 2006; Patil et al., 2011; Loukogeorgaki et al., 2012; Ruol et al., 2013; Ji et al., 2016). According to Dai et al. (2018), future research directions include the enhancement of hydrodynamic performance under long-wave conditions, the improvement of mooring system reliability, and the development of multifunctional and sustainable configurations capable of integrating coastal protection with renewable energy harvesting.

1.3.6 Eco-engineering adaptations

Beyond structural optimization, a growing body of research has emphasized the integration of ecological functions into the design of coastal defence structures. This eco-engineering perspective, also referred to as “building with nature” or “green-grey infrastructure”, aims to reconcile hydraulic performance with habitat creation and biodiversity enhancement (Airolidi et al., 2005; Bulleri and Chapman, 2010; Borsje et al., 2011; Firth et al., 2014; Pontee et al., 2016). Traditionally conceived as inert barriers, breakwaters, seawalls and dikes are increasingly viewed as multifunctional systems that

can sustain both engineering and ecological roles within coastal ecosystems (Naylor et al., 2012; Schoonees et al., 2019).

Recent developments have demonstrated that simple design interventions can increase the ecological value of hard and hybrid structures without compromising their stability or functionality. Increasing surface roughness or introducing structural heterogeneity-through grooves, cavities, crevices, or interstitial voids-enhances microhabitat availability for benthic and intertidal organisms (Chapman and Underwood, 2011; Coombes et al., 2015; Firth et al., 2014). Similarly, water-retaining depressions or shallow rock-pool features can mitigate thermal and desiccation stress during low tide, facilitating colonization by algae, molluscs and crustaceans (Chapman and Blockley, 2009; Strain et al., 2018). At the material level, bio-active or low-pH concretes, mineral additives, and textured moulds promote the attachment and early growth of biofilms and sessile assemblages (Airolidi et al., 2015; Coombes et al., 2015).

Green-grey hybrid strategies integrate biological components into engineered systems. Vegetated berms, salt-marsh fringes, and forelands can be combined with conventional armouring to dissipate wave energy, stabilize sediments, and provide habitat continuity (Temmerman et al., 2013; Sutton-Grier et al., 2015; Pontee et al., 2016). On earthen dikes and gentle slopes, halophytic vegetation increases surface roughness, reduces run-up, and contributes to long-term resilience through self-repair and carbon sequestration (Borsje et al., 2011; Schoonees et al., 2019).

Overall, eco-engineering promotes a shift from purely protective infrastructure toward multifunctional coastal systems delivering both safety and ecosystem services. By coupling engineering robustness with habitat creation, coastal defences act as resilient interfaces between land and sea, balancing protection needs with biodiversity enhancement in an era of accelerating climate change (Morris et al., 2018; Naylor et al., 2012). Figure 1.9 shows photos of “ecological engineering” applications on coastal structures; the image comes from Chapman and Underwood (2011).

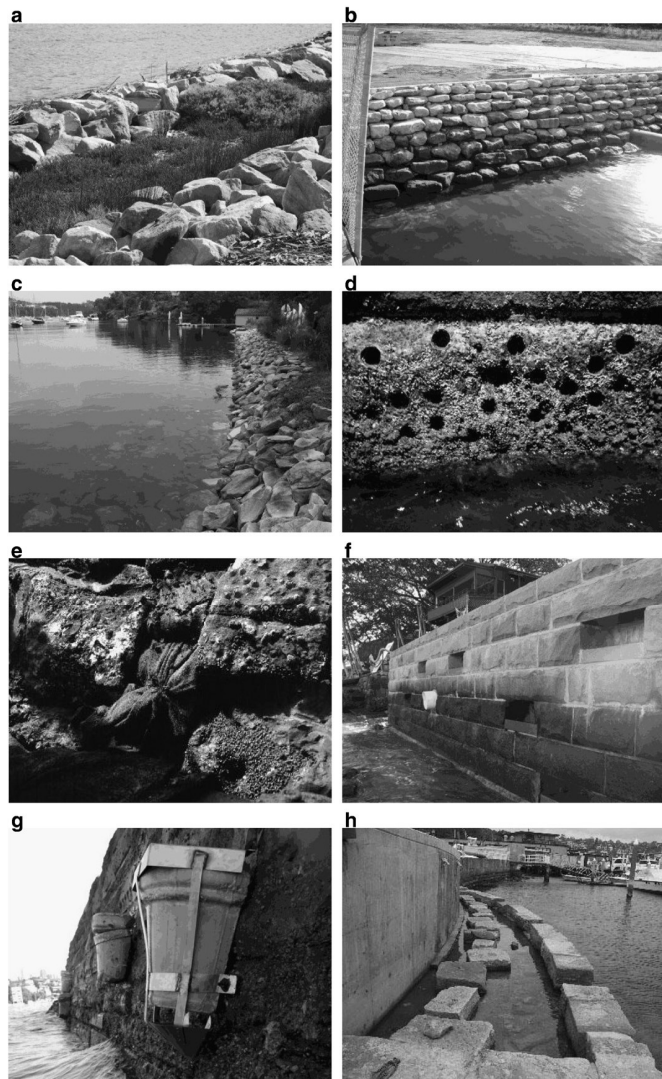


Figure 1.9: Various types of “ecological engineering” interventions applied to seawalls in Sydney Harbour: (a) a saltmarsh area planted in a “garden bed” created within a new sloping boulder wall at Kogarah Bay; (b) a stepped stone wall built at White Bay to replace a sheet-metal wall; (c) a structure of loose boulders with an extended intertidal platform constructed to replace a concrete wall at Quakers Hat Bay; (d) cavities drilled into sandstone blocks at Farm Cove to provide microhabitats for small gastropods; (e) sandbags used to create cavity features similar to rock pools in a wall at Rose Bay; (f) experimental cavities designed to mimic rock pools in a wall at McMahon’s Point; (g) “flower pots” attached to the seawall in North Sydney as analogues of rock pools; and (h) an intertidal pool created at the top of a new seawall at The Spit. Taken from Chapman and Underwood (2011).

1.3.7 Integration of wave energy converters (WECs)

Wave Energy Converters (WECs) integrated into coastal and harbor structures represent a significant innovation in modern coastal engineering, combining traditional protection functions with renewable energy generation (Falcão, 2010). As highlighted by Vicinanza et al. (2019), the concept of hybrid breakwaters with embedded WECs addresses two pressing challenges: the need for resilient coastal defences under sea level rise and extreme events, and the global transition toward sustainable energy systems. Two main typologies have proven suitable for structural integration, the Oscillating Water Column (OWC) and the Overtopping Device (OTD). OWC systems consist of an air chamber connected to the atmosphere through a bidirectional turbine, where wave-induced pressure oscillations generate air-flow for electricity production. They are typically integrated into vertical caisson breakwaters, maintaining high structural stability while reducing reflected wave energy and overall loading (Evans, 1982; Takahashi, 1988; Falcão, 2010; Viviano et al., 2016). Large-scale prototypes, including the Sakata Port facility in Japan, the Mutriku plant in Spain, and the ReWEC3 system in Civitavecchia, Italy, have demonstrated the technical feasibility of such devices, although efficiency remains relatively low compared to off-shore systems (Torre-Enciso et al., 2009, 2010; Arena et al., 2017; Vicinanza et al., 2019). OTDs, by contrast, rely on the controlled overtopping of waves into elevated reservoirs, where potential energy is converted through low-head turbines. Designs such as the Seawave Slot-Cone Generator (SSG) and the Overtopping Breakwater for Energy Conversion (OBREC) have shown promising hydraulic efficiency and adaptability to existing rubble mound or caisson geometries (Vicinanza and Frigaard, 2008; Contestabile et al., 2017). The OBREC concept, in particular, has been extensively investigated through small-scale physical modelling, numerical assessments, and full-scale monitoring campaigns (Vicinanza et al., 2014; Contestabile et al., 2020), highlighting its structural reliability and suitability for mild to moderate wave climates. Overall, breakwater-integrated WECs embody the evolution of traditional maritime infrastructure toward multifunctional systems capable of simultaneously ensuring coastal protection, energy harvesting, and reduced environmental impact, representing a crucial step toward the development of “green ports” and climate-resilient coastal zones (Vicinanza et al., 2019).

1.4 Background and motivation of the present study

Following the general overview of optimization strategies for coastal structures presented in the previous section, particular attention within this thesis is given to vertical caisson breakwaters. The optimization of vertical breakwaters in relatively deep water conditions has gained increasing attention due to the necessity of mitigating extreme wave loads while maintaining economic feasibility. A promising strategy for improving hydraulic performances without significant geometric modifications is the adoption of retreated wave walls, where the wave wall is positioned landward from the front face of the caisson trunk. This configuration has already been implemented in several real-world breakwaters, as for example in the Mediterranean Sea and Japan, motivating further research into its hydraulic and structural performance. Notably, the new offshore breakwater at the Port of Genoa (the second largest in the world), currently under construction, features a high-depth caisson with a retreated wave wall, designed to enhance wave energy dissipation while preserving stability. Similarly, the vertical breakwater at the Port of Civitavecchia includes a retreated wave wall, demonstrating its applicability in Mediterranean engineering practice. In Japan, this design has been adopted in more specialized configurations, such as in the Wakayama Shimotsu Breakwater, where a retreated wave wall is integrated into a circular caisson, and in the Kashima Breakwater, where it is used for tsunami protection.

Within this context, particular attention within this thesis is given to vertical caisson breakwaters with retreated wave walls, especially in relatively deep water conditions. This configuration has recently attracted growing research and engineering interest, motivating the investigations presented in this work. The conceptual difference between the traditional and the optimized layouts is illustrated in Figure 1.10: in the conventional flushed wall configuration the wave wall is aligned with the caisson trunk, while in the retreated wall configuration it is shifted landward, creating a horizontal, or eventually inclined, platform that modifies the hydrodynamic of wave-structure interaction.

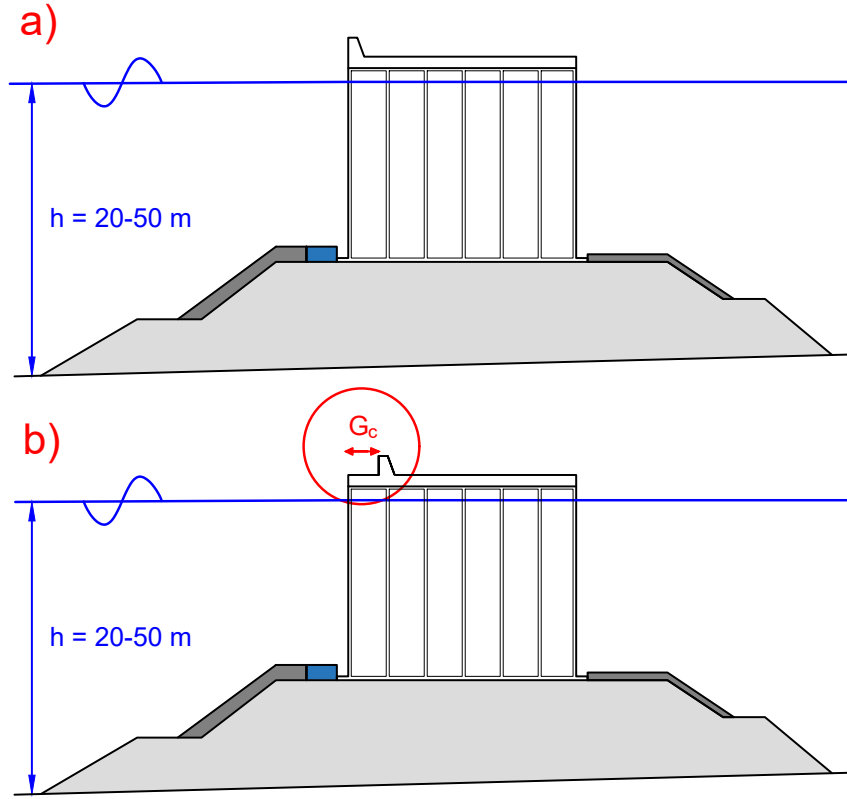


Figure 1.10: Conceptual sketch of composite vertical breakwaters. Panel a) conventional configuration with a flushed wave wall. Panel b) configuration with a retreated wave wall (G_c indicates the distance of retreat).

Despite these practical implementations, design guidelines for retreated wave walls are lacking. In fact, most of the available literature and classical design guidelines (Goda, 2010; Takahashi, 1996; Oumeraci et al., 2001; Martinelli and Lamberti, 2011) concentrates on flushed wave wall structures, while several works addressed the study of wave loads induced by breaking waves on vertical walls placed in shallow water conditions (Cooker and Peregrine, 1990; Bullock et al., 2007; Bredmose et al., 2009; Cuomo et al., 2010a,b; Bredmose et al., 2015). Indeed, studies that investigate forces and pressures on retreated walls placed on tops of smooth dikes and rubble mound breakwaters are available (Martin et al., 1999; van Gent, 2003; De Rouck et al., 2012; Chen et al., 2015; Van Doorslaer et al., 2017; Molines et al., 2019; De Finis et al., 2020; Cao et al., 2021; Chen et al., 2021), but the physical processes that take place at this type of structures appear to be significantly different from those for vertical walls.

In order to advance the understanding of the physical processes and governing factors that influence the reduction or increase of wave loads on this type of structure, Romano and Bellotti (2023) recently conducted a series of experimental investigations. The study reports the results of a wave flume campaign, illustrating the main phenomena involved and the physical and geometrical drivers expected to affect the force variation. The geometry of the caisson and rubble mound foundation was kept constant, while four wave wall configurations were tested: one aligned with the trunk and three in retreated positions. The wall was purposely designed high enough to prevent overtopping. A total of 76 tests were performed, corresponding to 19 regular wave conditions for each geometry.

Despite the limited number of geometries and wave conditions, the experimental campaign by Romano and Bellotti (2023) has led to a better understanding of the effects of the retreated wall and to identify the physical/geometrical drivers and preliminary conclusions and considerations, as follows:

- in general, the wave forces on the whole structure and on the caisson trunk tend to decrease as the wave wall retreat increases;
- the wave forces on the wall in general increase for retreated wall configurations;
- the retreated wall is subject to impulsive loads, due to the development of a bore on the flat superstructure between the seaward edge of the trunk and the wave wall;
- the time shifting between wave loads on the trunk and impacts on the wall is significant only if the retreat of the wall is relevant. While small retreats induce negligible shifting and strong impacts on the wall, resulting in equal or even larger global forces on the structure as a whole;
- a major source of wave loads reduction is given by the fact that reflection is reduced by the wave overtopping occurring at the seaside edge of the trunk, resulting in smaller wave pressures acting on the vertical front of the caisson;
- the physical processes mentioned above (impacts, time shifting, reduction of pressures at the seaside edge of the trunk) can have both a concordant and antithetical effect among them.

These results highlight the complexity of the physical processes governing wave-structure interaction and the need for further investigation to achieve a comprehensive understanding. While the main drivers responsible for the reduction of global forces on the caisson were clearly identified, only a limited range of geometric and wave parameter combinations has been explored so far. In particular, a significant part of the observed response is controlled by post-overtopping flow phenomena, potentially involving highly transient air–water interactions and impulsive loading mechanisms that remain comparatively underexplored in the literature.

1.5 Concluding remarks on the problem context

The growing vulnerability of coastal infrastructure to climate change-driven hazards, such as intensified wave conditions, rising sea levels, and more frequent extreme events, underscores the need for resilient and adaptive design strategies (Mori et al., 2010; Vousdoukas et al., 2018b, 2020a; IPCC, 2019). Traditional coastal and maritime structures, many of which were designed under outdated assumptions, now face increasing hydrodynamic pressures, requiring systematic evaluation and targeted optimization to ensure long-term functionality (Camus et al., 2017, 2019; EurOtop, 2018). Paradoxically, the potential for optimizing coastal structures is virtually limitless, as it is shaped by site-specific conditions and the ingenuity of the designer. Indeed, each coastal environment and structures present different challenges, offering opportunities for a diverse range of engineering solutions tailored to local needs and environmental variables, as shown in the previous Section 1.3. Following this introduction, the next chapter outlines the objectives and methodology of the research and provides an overview of the structure of the thesis with the content of each chapter.

Chapter 2

Objectives and Methodology

2.1 Research questions

The present thesis aims to study the hydraulic performance of composite vertical breakwaters under specific and critical conditions, better detailed below. Building on the preliminary findings of Romano and Bellotti (2023), this research investigates the effects of wave-structure interactions on composite vertical breakwaters with both flushed and retreated wave walls, focusing on the resulting forces and pressures, wave reflection, overtopping behavior and the role of post-overtopping flows (i.e., bores) that occur on the superstructure (Chapters 3, 4, 5). Additionally, it explores the dynamics of post-overtopping flows and their impact on internal caisson cells that, during temporary construction phases, might remain uncovered by the superstructure, thus being directly exposed to wave actions (Chapter 6).

The research questions guiding this work were progressively formulated as new evidence highlighted emerging challenges and knowledge gaps. Accordingly, the chapters of this thesis are organized chronologically, reflecting the evolution of the research path from initial small-scale exploratory tests to the comprehensive large-scale campaigns:

- **RQ1:** How does the retreat distance of the crown wave wall influence wave-induced forces, reflection, and overtopping performance of composite vertical breakwaters under varying geometrical and wave conditions?
- **RQ2:** How do different types of post-overtopping flows interact with retreated crown wave walls, and which hydrodynamic features govern the magnitude, timing, and impulsive nature of the resulting loads on

the wall and on the superstructure?

- **RQ3:** How much does random phase seeding in irregular-wave generation affect the estimation of mean and extreme overtopping indicators and wave-induced loads on a vertical breakwater, and what are the implications for the robustness of experimental/design metrics?
- **RQ4:** What hydrodynamic loads are induced by post-overtopping flows on internal caisson cells left temporarily uncovered during construction phase, and how do these loads evolve across different filling regimes and along the caisson?

2.2 Methodological framework

This section introduces the general methodological framework adopted in this thesis, outlining the experimental and numerical approaches used to investigate the hydraulic behaviour of vertical breakwaters.

2.2.1 Physical models

The foundations of physical model testing for coastal structures were laid by researchers like Jensen (1984); Owen and Allsop (1984); Hughes (1993). More recently, Wolters et al. (2009) introduced updated guidelines that have become a benchmark for unified approaches to physical modelling in coastal engineering. These guidelines, which focus on standardizing methodologies for coastal structure testing, were followed rigorously in the experiments conducted for this thesis.

Both physical and numerical modelling are extensively used to evaluate the hydraulic and structural performance of coastal defences. Coastal structures, by nature, are highly site-specific and are affected by unique local boundary conditions such as bathymetry, wind patterns, wave climates, and sediment transport dynamics. For relatively simple structures, or designs that have been previously tested, predictive tools and accumulated design experience are often sufficient to ensure reliability. However, in cases involving complex geometries, novel designs, or where safety margins are uncertain, physical models play a critical role in refining and optimizing the design.

Physical modelling is commonly used to investigate wave-structure interactions, such as wave overtopping, run-up, reflection, and transmission, as well as armour stability and hydrodynamic forces. These models allow for the evaluation of specific upgrading or repairing measures on existing structures, providing a comparative analysis of their performance. One of the significant advantages of physical models lies in their ability to capture complex, real-world phenomena that are challenging to replicate using theoretical or numerical approaches. Furthermore, they enable direct observation of physical processes, which is invaluable in understanding and interpreting the data collected.

However, physical modelling has limitations, including scale effects, laboratory influences, and the constraints of testing within controlled environments. It is essential to ensure that physical models are scaled correctly to replicate the prototype's conditions accurately. To achieve reliable results, three types of similarity must be maintained: geometric, kinematic, and dynamic. Geometric similarity ensures that all linear dimensions in the model are scaled proportionally to the prototype. Kinematic similarity ensures that the flow patterns in the model mimic those of the prototype. Dynamic similarity requires that the ratios of forces in the model are consistent with those in the full-scale structure. Maintaining these similarities is vital to minimizing errors in experimental outcomes.

Several dimensionless numbers are used to characterize and validate the scaling process. The Froude number, Reynolds number, and Weber number account for gravity, viscosity, and surface tension forces, respectively. For coastal engineering applications, the Froude number is typically prioritized, as gravity forces dominate in wave-structure interactions. The scaling criteria are particularly crucial when dealing with turbulent flows, and specific thresholds for Reynolds and Weber numbers must be met to ensure the validity of the model results.

The relationship between the model and prototype is defined using dimensionless numbers, which capture the governing forces of the system. The key dimensionless numbers in physical modelling are:

- Froude Number:

$$Fr = \frac{u}{\sqrt{gL}} \quad (2.1)$$

-
- Reynolds Number:

$$Re = \frac{uL}{\nu} \quad (2.2)$$

- Weber Number:

$$We = \frac{\rho_w u^2 L}{\sigma} \quad (2.3)$$

- Reynolds Number for the armour:

$$Re_D = \frac{\sqrt{gH_s} D_n}{\nu} \quad (2.4)$$

Where: u is the particle velocity [$\frac{m}{s}$], g is the gravitational acceleration [$\frac{m}{s^2}$], ν is the kinematic viscosity of water and it is approximately $10^{-6} \frac{m^2}{s}$, ρ_w is the density of water [$\frac{kg}{m^3}$], σ is the surface tension [$\frac{N}{m}$], H_s is the significant wave height [m] and D_n is the nominal diameter of the armour units [m].

The Froude number ensures that gravity forces are correctly scaled between the model and prototype. For wave-structure interactions, gravity is the dominant force, making the Froude scaling law essential. When the Froude law is applied, surface tension (Weber number) and viscosity (Reynolds number) can generally be neglected if the model is appropriately scaled. Specific thresholds must be met to ensure accurate scaling: wave heights must exceed 0.03 m, water depths must exceed 0.05 m, and wave periods must be greater than 0.35 s Le Méhauté (1976); Hughes (1993). In addition, the dimensions of the armour unit should be scaled so that their nominal diameters are above 0.025 m.

For physical models of breakwaters, the scale factor typically ranges from 1:30 to 1:60 for two-dimensional experiments and from 1:30 to 1:80 for three-dimensional setups Hudson et al. (1979); Oumeraci (1984); Hughes (1993). The following criteria must be met to ensure reliable results:

- The largest feasible scale should be used to minimize scale effects.
- Turbulent flow conditions should be maintained throughout the armour layer ($Re_D \geq 30000$).
- Structural elements must be geometrically scaled to ensure dynamic similarity.
- The Froude criterion should govern the flow hydrodynamics.

A summary of the Froude scaling relationships is provided in Table 2.1. These principles were rigorously applied in the experiments described in this research. The following subsection briefly describes the wave flumes available at the Roma Tre Laboratory, while the specific details of each experimental setup and model are provided in their respective sections.

Parameter	Units	Scale factor
Acceleration	m/s^2	λ^0
Density	kg/m^3	λ^0
Discharge	$l/s/m$	$\lambda^{\frac{3}{2}}$
Force	N	λ^3
Frequency	s^{-1}	$\lambda^{-0.5}$
Length	m	λ
Mass	kg	λ^3
Moment	$N \cdot m$	λ^4
Pressure	N/m^2	λ
Speed	m/s	$\lambda^{0.5}$
Time	s	$\lambda^{0.5}$

Table 2.1: Summary of Froude scaling conversion factors

2.2.2 Numerical models

Experimental investigations in wave flumes or basins have long served as a significant design resource in coastal engineering, even though they usually allow only a limited range of configurations and wave conditions. To overcome these restrictions, numerical simulations of wave-structure interactions have grown increasingly popular over the last two decades and have become valuable tools in engineering design. Numerical models can broadly be divided into two main categories based on the governing equations: depth-integrated models and Computational Fluid Dynamics (CFD) approaches.

Early examples of depth-integrated models were derived from the Non-linear Shallow Water Equations (NSWE), as discussed by Kobayashi and Wurjanto (1989a,b); Hu and Causon (2000); Hubbard and Dodd (2002). These models, obtained through vertical integration of the Navier-Stokes equations under a hydrostatic pressure assumption, are computationally efficient. However, they present certain limitations: the boundaries must be positioned close to the structure to fulfill the shallow-water condition, wave breaking and porous media flow are treated in a semi-empirical manner, and

they cannot account for frequency dispersion. Moreover, due to nonlinear effects, wave steepening can continue throughout propagation. A widely used depth-integrated model today is SWASH (Simulating WAVes till SHore), which incorporates non-hydrostatic terms in the governing equations (Zijlema and Smit, 2011).

A development of NSWE-based methods is represented by Boussinesq-Type Equations (BTE), which are recognized for their accuracy in the nearshore region. The original work by Madsen and Sørensen (1992); Nwogu (1993), which focused on depth-averaged velocity formulations, paved the way for improved versions capable of providing reliable predictions even in intermediate water depths (Kennedy and Dalrymple, 2001; Liu, 1995; Lynett and Liu, 2002; Wei and Subramanya, 1995). Unlike NSWE models, BTE models include dispersion effects through nonlinear partial differential equations, although they may produce less reliable results in scenarios involving highly nonlinear wave behavior.

Among CFD-based approaches, several well-established methods solve the Navier-Stokes equations (Kawasaki, 1999; Li and De Rouck, 2004), incorporating turbulence modelling and the Reynolds-Averaged Navier-Stokes (RANS) equations to handle wave breaking. By introducing the Volume Averaged Reynolds Averaged Navier-Stokes (VARANS) equations, Hsu and Liu (2002) extended a conventional RANS framework to account for flow within porous structures. These methods usually adopt free-surface tracking techniques (e.g., Volume of Fluid or Level Set) to capture the evolution of the air-water interface.

Another subset of CFD includes models based on the Moving Particle Semi-implicit (MPS) technique (Koshizuka and Oka, 1996) or the Smoothed Particle Hydrodynamics (SPH) method (Dalrymple and Knio, 2001; Gotoh and Memita, 2004; Shao and Gotoh, 2004). In contrast to the Eulerian viewpoint of the previously mentioned approaches, MPS and SPH adopt a Lagrangian perspective, where the motion of each fluid particle is tracked individually. While this formulation is well-suited for handling large free-surface deformations, it typically entails high computational costs. A recent, widely adopted SPH-based software is DualSPHysics (Crespo et al., 2015).

The numerical model employed in this thesis adopts an Eulerian approach and is based on the open-source CFD framework OpenFOAM® (Jasak, 1996), using the IHFOAM solver for the simulation of wave-structure

interaction and free-surface flows (Higuera et al., 2013a,b). These tools have been widely adopted in coastal engineering applications and are documented in the relevant literature (Higuera et al., 2014a,b; Castellino et al., 2018; Romano et al., 2020). Although this model will be examined in detail in the Chapter 6, some general and common features are outlined here. As noted above, the Eulerian-type CFD models treat the fluid as a continuum, with flow velocity expressed as a function of space and time. Numerical solutions to the Navier-Stokes equations are obtained at discrete locations in the computational domain. Consequently, the first step in using these models is the generation of a computational mesh, whose resolution (fineness) directly influences both the accuracy and the computational demands of the simulation. A mesh convergence study is generally performed to identify an optimal grid size, and it is common practice to calibrate the initial turbulence parameters. Following these preparatory steps and validation, the model can be applied to more extensive simulations.

The next section describes how these approaches were combined within the different experimental and numerical campaigns developed throughout the thesis.

2.3 Thesis outline

Based on the methodological approaches and to address the research questions introduced in the previous paragraphs, this thesis adopts an experimental and phenomenological approach, mainly based on physical modelling and, in one specific case, complemented by numerical simulations (see Chapter 6). The methodological framework was conceived as a sequence of independent but complementary laboratory campaigns, each designed to investigate a specific aspect of the hydraulic behaviour of vertical breakwaters.

All experiments were carried out at the Hydraulic and Maritime Engineering Laboratory of Roma Tre University, following the principles of physical modelling and standardized measurement procedures, as described in the next section. The overall methodological approach was organized as a progressive investigation, in which each experimental phase addresses one of the main research questions introduced before:

- **Chapter 3 - Small-scale experimental investigation.** Building on the previous research by Romano and Bellotti (2023), this chapter

extends the investigation to a wider and more systematic exploration of wall retreat effects. It addresses **RQ1** through an extensive campaign of small-scale regular-wave physical model tests, varying both structural geometries and wave conditions across a broad parametric range, and quantifying how retreat distance influences forces, reflection and overtopping in comparative/statistical terms.

- **Chapter 4 - Post-overtopping flows on retreated wave wall.** This chapter addresses **RQ2** by analyzing large-scale regular-wave experiments focused on the dynamics of post-overtopping flows. It classifies the main types of post-overtopping events (Dam-Break, Plunging Dam-Break, and Hammer-Fist), already known in literature (Greco et al., 2005, 2007), and investigates the hydrodynamic parameters that govern their interaction with retreated walls. A novel image-based methodology (Formentin et al., 2024) is introduced to map the spatial and temporal evolution of these flows, offering a framework to interpret impulsive load generation, the air content and the influence of wall retreat on such processes.
- **Chapter 5 - Influence of random seeding under irregular waves.** This chapter addresses **RQ3** by investigating, in line with previous studies (Romano et al., 2015; Williams et al., 2019), the variability introduced by random wave generation (random seed) in physical tests on a standard vertical caisson with a flushed wave wall. Through systematic repetitions under identical spectral conditions, it quantifies seed-to-seed variability in overtopping indicators and wave-induced loads, discussing implications for repeatability and robustness of experimental/design metrics under irregular waves.
- **Chapter 6 - Loads on internal caisson cells during construction.** This chapter addresses **RQ4**, exploring wave-induced pressures on the internal diaphragms (cells) of caisson breakwaters in a temporary configuration with open compartments. Laboratory experiments are complemented by CFD simulations to interpret filling-regime effects, validate pressure evolution, and improve understanding of this construction-phase vulnerability.
- **Chapter 6.5 - Conclusions.** This chapter synthesizes the main findings, provides explicit answers to the research questions, and outlines practical implications and recommendations for future research.

After this synthesis on how each chapter is structured and how it con-

tributes to answering the research questions discussed in Section 2.1, the following subsection describes the experimental facilities at Roma Tre University where the laboratory campaigns were carried out.

2.3.1 The Roma Tre laboratory

The experimental activities described in this thesis were conducted at the Hydraulic and Maritime Engineering Laboratory of Roma Tre University (Department of Civil, Computer Science and Aeronautical Technologies Engineering, Rome, Italy). The facility is equipped with two wave flumes of different sizes, both featuring piston-type wave makers controlled by the software AWASYS 7 (Aalborg University, 2018), which incorporates an active wave absorption system (Andersen et al., 2016, 2018), and second-order wave generation using Schäffer and Steenberg (2003) with modification by Eldrup and Andersen (2019).

The smaller flume, used for the experiments presented in Chapter 3, is made of Perspex and measures 9.0 m in length, 0.27 m in width, and 0.50 m in height. It allows the generation of both regular and irregular waves with a maximum paddle stroke of 1.0 m. This facility was particularly suited for the large number of small-scale tests (approximately 2000) carried out to systematically analyze the influence of the wave wall retreat on wave loads, reflection, and overtopping. A photo and a sketch of the small wave flume is found in Chapter 3 in Figure 3.1.

The larger flume, used in the experimental campaigns described in Chapters 4, 5, 6, is primarily made of steel panels, with a 4 m long glass section that allows for visual observations and optical measurements of wave-structure interactions, which enabled the execution of image-based analyses of post-overtopping events, as described in detail in Chapter 4. The wave flume measures 20.0 m in length, 0.605 m in width, and 1.0 m in height, with a maximum water depth of about 0.70 m and a maximum wave height of approximately 0.25 m. A photo and a sketch of the larger wave flume is found in Chapter 4 in Figure 4.1.

Both facilities are equipped with state-of-the-art measurement systems, including resistive-type wave gauges (VTI Resistive Wave Gauge Modules installed in a WG-16CH rack) that measure the instantaneous water surface elevation through pairs of stainless-steel wires. The signal, proportional to

the immersion depth, is converted into voltage and recorded via a dedicated data acquisition system. The probes are calibrated least twice daily in still water using fixed reference points to account for temperature and salinity related variations. Additional wave gauges are placed on the paddle to monitor the incident wave field and to ensure effective active absorption. Both wave flumes can also be equipped with high-frequency pressure transducers (TRAFAG Submersible Pressure Transmitter NAL 8838, range 0-200 mbar), allowing the measurement of wave-induced pressures on structural models with high spatial and temporal resolution. The details of the configuration of the pressure transducers and wave gauges and their application in the different experimental campaigns are provided in the corresponding sections of each chapter.

Chapter 3

Small-scale experimental investigation

Parts of the work presented in this chapter have been included in the journal paper: Romano, A., Centorami, M., Cecioni, C., Bellotti, G. (2024). A physical model study on the hydraulic performances of vertical breakwaters with retreated wave walls. Coastal Engineering, 191, 104539.

3.1 Introduction

Building on the previous research by Romano and Bellotti (2023) this chapter describes the experimental investigation conducted to evaluate the hydraulic performance of composite vertical breakwaters equipped with retreated wave walls. The chapter aims to systematically assess how varying the distance of the wave wall from the seaward face of the caisson affects key hydraulic parameters such as wave-induced forces, reflection coefficients, and wave overtopping rates, under a broad range of regular wave conditions.

To achieve statistically robust insights, an extensive series of approximately 2000 two-dimensional physical model tests was carried out in the wave flume of Roma Tre University. In order to make feasible the execution of such a large number of tests, the small-scale wave flume was used and regular wave conditions were reproduced. Although these choices might represent a limitation of the present results, they appear to be a reasonable approach to investigate the complex physical processes involved (Ravindar et al., 2021). The test program explored multiple retreated wall configurations and wave conditions, significantly expanding the parameter space investigated in previous studies. Measurements included time histories of

forces acting separately on the caisson trunk and on the wave wall, as well as the overall structure, enabling a comprehensive evaluation of the force redistribution mechanisms induced by wall retreat.

Wave reflection and overtopping were also systematically measured, as overtopping represents a fundamental performance parameter for breakwater design, yet had not been previously considered in comparable experimental campaigns. The complexity and variability observed in the experimental data motivated the adoption of a statistical approach to derive reduction or amplification factors of forces and overtopping compared to the standard flushed wall configuration, rather than attempting to define empirical predictive formulas.

The chapter is organized as follows. After this introduction, a description of the experimental setup, together with the model structure and the wave conditions, is provided. Then, a detailed description and discussion of the experimental results, together with the limitations of the present study, is given. Finally, key insights of this section close the chapter.

3.2 Experimental setup

3.2.1 The physical model

The two-dimensional experiments described in this chapter were carried out in the small wave flume of the Hydraulic and Maritime Engineering Laboratory at Roma Tre University (see Figure 3.1, panel a)). Since the general characteristics of the facility and its wave generation and absorption systems have already been presented in Section 2.3.1, only the elements that are directly relevant to the present experimental campaign are described here.

A total of seven resistive wave gauges are positioned along the flume to measure the free surface elevation. As can be noted in the sketch of Figure 3.1 (panel d)), two wave gauges are fixed on the wave maker in order to provide the feedback to the active absorption system (WG1 and WG2). Four wave gauges have been aligned in the central longitudinal axis of the flume, to measure the wave field in front of the structure (WG3-WG6). The last wave gauge (WG7) is used to measure wave overtopping: it is placed in a fixed tank located behind the structure, in which the overtopping water volumes are collected through a chute. The acquisition frequency of the

wave gauges is set at 50 Hz, and a calibration procedure is carried out at least twice daily to account for environmental variations.

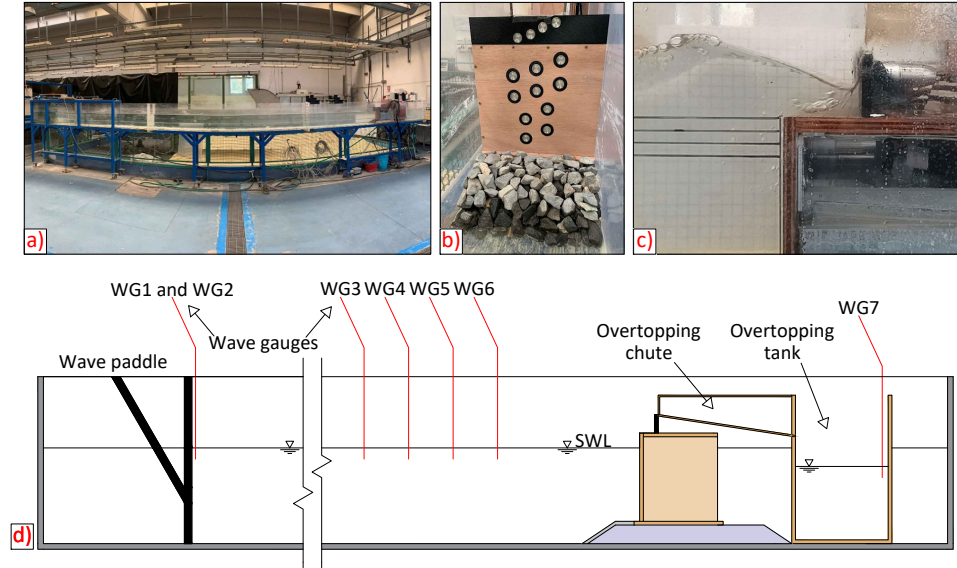


Figure 3.1: Panel a): picture (lateral view) of the wave flume. Panel b): picture (front view) of the caisson model (trunk in marine plywood and 3D printed wave wall) with pressure sensors. Panel c): picture (lateral view) of the wave wall during an impact. Panel d): sketch of the wave flume, model structure and overtopping chute and tank.

The model structure replicates a vertical breakwater placed on top of a rubble mound foundation (see Figure 3.1, panel b)). The foundation is made of two parts: an inner core of uniform-size rock material and an external rubble mound layer made of rocks (weight range 10.0-20.0 g). The vertical breakwater is composed by the trunk, made of marine plywood (thickness 0.01 m) and the wave wall, made of plastic and able to be fixed at different positions in order to test different wave wall retreats. In panel c) of Figure 3.1 the picture captures a wave impact on the wall during a test with retreated wall position. The caisson trunk height is 0.248 m and the wave wall height is $h_w = 0.050$ m; their width is 0.270 m to perfectly fit inside the wave flume. The dimensions of both the caisson trunk and the wall are kept constant during all the tests. The physical model is designed to be representative, in Froude similarity, of a quite broad range of vertical breakwater configurations in relatively deep-water, rather than replicating a specific existing structure. Nevertheless, a scale factor of approximately 1:100 can be assumed, to get at prototype scales a composite vertical breakwater based

on water depths in the range of 27.0-35.0 m, with freeboard values between 5.0-8.0 m. With this scale factor, considering also the other geometrical parameters, the tested structures refer to a typical Italian vertical breakwater (Franco, 1994).

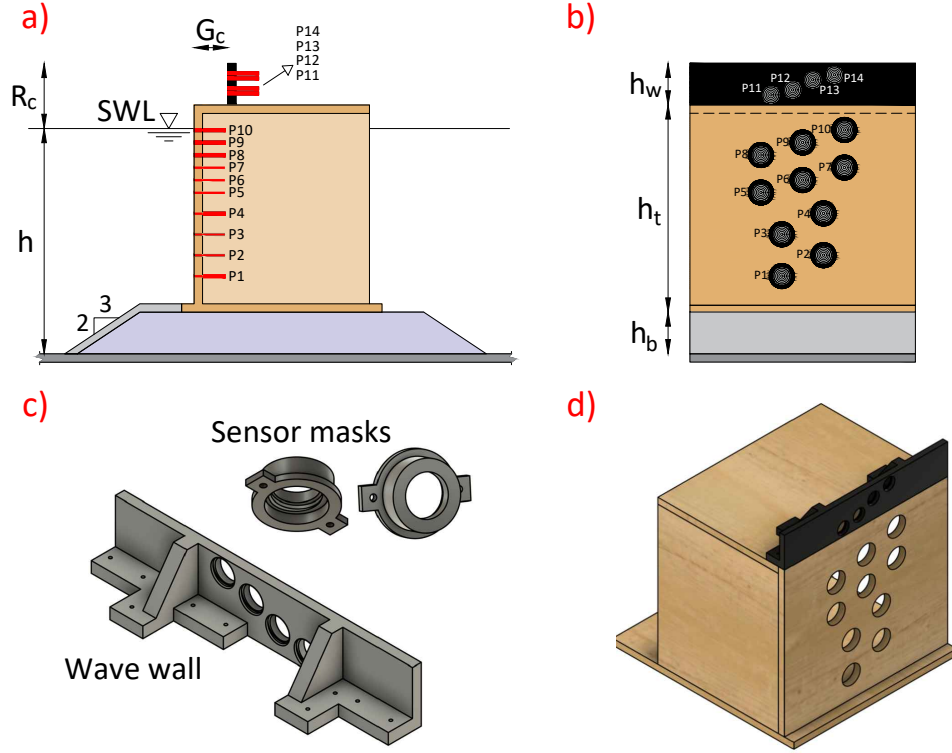


Figure 3.2: Panel a) and b): sketch of the pressure sensor positions and geometrical parameters. Panel c): detail of the 3D printed wave wall and pressure masks with thread. Panel d): 3D render of the model structure with wave wall and pressure sensors housing.

To accurately measure the wave-induced actions on the vertical structure, 14 pressure transducers (P_i , $i = 1, 2, \dots, 14$) are fixed at the seaside face of the structure. The acquisition frequency of these pressure sensors is 7,000 Hz to effectively measure impulsive load peaks. As illustrated in Figure 3.2 (panels a) and b)) and as can be noted in the model picture (Figure 3.1, panel b)), 10 sensors (P1-P10) are placed on the caisson trunk and 4 (P11-P14) on the wave wall. These transducers are strategically positioned in order to provide a very high spatial resolution. Moreover, to maximize the number of sensors in the areas of most interest, i.e. on the wave wall and around the mean sea level, the mutual distance between the pressure

sensors is not constant spanning in the range 0.006 m-0.025 m. In Table 6.1 pressure sensor names and positions are reported.

Due to the reduced scale of the experiments, a particular care has been adopted in minimizing potential laboratory effects. To this end, some special pieces have been ad-hoc designed and 3D-printed for housing the pressure sensors with the highest possible precision and minimizing possible sources of flow disturbance. In particular, the wave wall has been 3D-printed with plastic material, guaranteeing a structure as rigid as possible and equipped with threaded holes to house the pressure sensors (Figure 3.2, panel c)). On the other hand, to equip the caisson trunk with a large number of pressure sensors, and to keep ensure that its seaside face were as rigid and flat as possible, some special threaded masks for screwing the pressure sensors have been designed and 3D-printed (Figure 3.2, panel c)). These elements are fixed at the seaside face of the trunk, ensuring the perfect alignment between themselves and the vertical caisson. Panel d) of Figure 3.2 shows a render of the model structure with the wave wall in the flushed configuration and the trunk ready for housing the sensor masks.

Pressure Sensor	Elevation (m)	Influence Area (m)
<i>P1</i>	0.043	0.0455
<i>P2</i>	0.068	0.0250
<i>P3</i>	0.093	0.0250
<i>P4</i>	0.118	0.0250
<i>P5</i>	0.143	0.0250
<i>P6</i>	0.158	0.0150
<i>P7</i>	0.173	0.0150
<i>P8</i>	0.188	0.0150
<i>P9</i>	0.203	0.0150
<i>P10</i>	0.218	0.0375
<i>P11</i>	0.260	0.0150
<i>P12</i>	0.266	0.0090
<i>P13</i>	0.278	0.0090
<i>P14</i>	0.284	0.0170

Table 3.1: Pressure sensor names (first column), vertical positions measured from the toe of the caisson (second column) and areas of influence (third column). Note: areas are expressed in m as it is a 2D experiment.

3.2.2 Test program

The parameters space explored in the present experimental campaign is shown in Figure 3.3. The figure reports the value of the main geometrical and wave parameters tested in the present experimental campaign (gray markers), together with those explored by Romano and Bellotti (2023) (red markers). All the panels of Figure 3.3 use non-dimensional parameters axes, considering that h is the water depth at the toe of the rubble mound foundation; L_0 is the wave length in deep water conditions calculated as $L_0 = \frac{gT^2}{2\pi}$; H is the incident regular wave height; R_c is the crest wall freeboard; G_c is the horizontal wall retreat with respect to the seaward caisson face; h_b is the height of the rubble-mound foundation and h_w is the wall height (for geometrical parameters refer to the Figure 3.2 and List of Symbols).

The program for the new test campaign was thought in order to widen the range of the parameters values explored in the previous campaign (red markers in the Figure 3.3). To this end, both wave and geometrical parameters have been systematically varied. Specifically, regarding the geometrical parameters, the experiments considered two different rubble mound foundation heights, $h_b = 0.05$ and 0.1 m; six different wave wall retreated positions: G_{c0} for the flushed wall configuration ($G_c = 0$) and G_{c1} - G_{c5} , respectively for $G_c = 0.00, 0.02, 0.04, 0.06, 0.10$ and 0.15 m. The wave wall height h_w is fixed, however the R_c value varies due to different water depth tested. The wave height and the wave period of the regular waves and the water depth are systematically varied within the following ranges: $0.027 \leq H \leq 0.115$ m, $0.7 \leq T \leq 1.63$ s, and $0.27 \leq h \leq 0.35$ m. Table 3.2 reports the range of tested parameters values.

Therefore, 322 wave and geometrical parameters combinations have been tested for each wall configuration, resulting in a total number of 1932 experiments, and allowing the comparison between the performances of the structure under the same wave conditions for the flushed wall configuration (G_{c0}) and for each wall retreat (from G_{c1} to G_{c5}). The duration of each test is approximately 75.0 s in order to obtain a wave time series long enough to get stationary conditions acting on the structure.

Parameters	Min. Value	Max. Value
H (m)	0.027	0.115
T (s)	0.700	1.630
h (m)	0.270	0.350
H/h (-)	0.090	0.373
h/L_0 (-)	0.077	0.393
H/L_0 (-)	0.008	0.077
R_c/H (-)	0.547	2.805
G_c/L_0 (-)	0.000	0.196
h_b/h (-)	0.167	0.313
h_w/R_c (-)	0.641	1.042

Table 3.2: Ranges of explored parameters tested during the new experimental campaign.

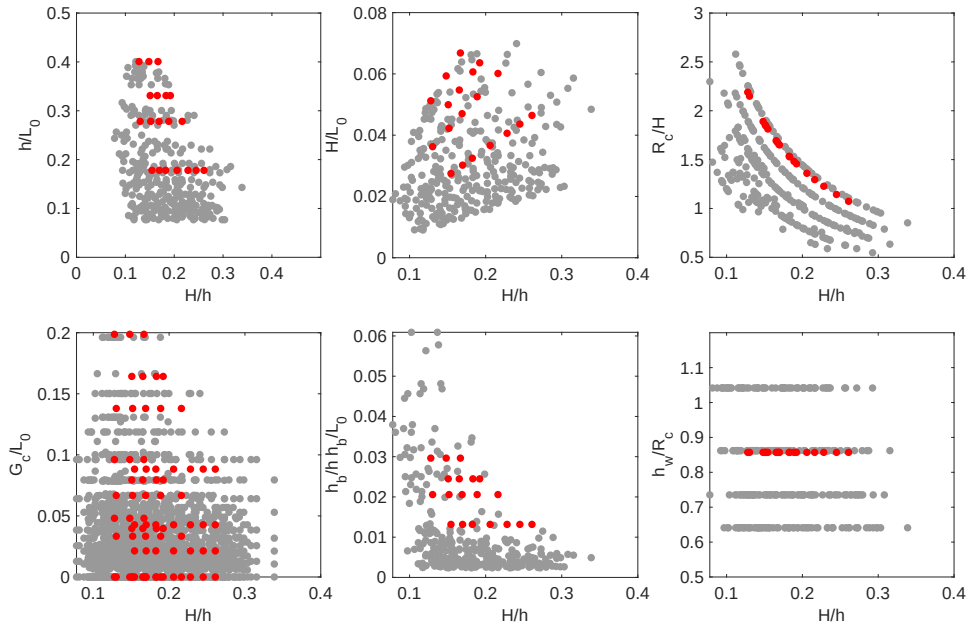


Figure 3.3: Ranges of explored parameters tested during the new experimental campaign (filled gray dots). Note that filled red dots refer to the previous data of Romano and Bellotti (2023).

3.3 Results and discussion

In this section the experimental results are presented and discussed. The analysis is divided into three parts: 1) the analysis of forces acting on the structure; 2) the analysis of reflection coefficients; 3) the analysis of wave overtopping. The last subsection discusses the limitations of the study.

3.3.1 Wave forces

For each test, the fourteen pressure signals were examined within a 20 s time window selected to include only the stationary part of the record, thereby avoiding the initial transient phase. The pressure records were integrated by assigning to each sensor its specific area of influence (values are listed in Table 6.1). This procedure yields three force time series: the global force acting on the entire structure, $F_G(t)$; the force on the caisson trunk, $F_T(t)$; and the force acting on the wave wall, $F_W(t)$. No filtering was applied to the signals. A peaks-over-threshold (POT) analysis was then performed on each record to identify the force maxima within the selected time interval. Since regular waves were used, for every experiment an average value of these force peaks was computed. Throughout the following discussion, F_G , F_T , and F_W denote such average peak global, trunk and wall forces, respectively.

To identify the average peak forces related to the 6 positions of the wave wall, the following symbols are used throughout the chapter: F_G^V , F_T^V , F_W^V , $F_G^{R_i}$, $F_T^{R_i}$, $F_W^{R_i}$. Note that the apex $(\cdot)^V$, standing for “vertical”, is used from now on throughout the whole manuscript to identify quantities related to the flushed wall configuration (G_{c0}). The apex $(\cdot)^{R_i}$ ($i = 1, \dots, 5$) standing for “retreated”, is used to identify quantities related to the five retreated wall configurations ($G_{c1} - G_{c5}$).

The analysis of the global forces is illustrated in Figure 3.4, which is organized into six panels (two rows and three columns). In each panel, the experimental data are plotted as solid black markers, while the dashed line represents the bisector. Panels b) - f) show $F_G^{R_i}$ plotted against F_G^V , whereas panel a) reports F_G^V on both axes, resulting in a perfect alignment of the data along the bisector. This figure enables a direct comparison of force amplification or reduction associated with the various wall positions. To guide the interpretation, a best-fit line (in blue) is added to each panel. These trend lines are not intended as predictive tools; correlation coefficients are

not reported, and the lines only highlight the general tendency of the data. A statistical quantification of force increase/reduction, suitable for preliminary design purposes, is presented later in this section.

For small wall retreats (panels b) and c)), the data points mainly cluster around the bisector, with moderate scatter, consistent with the inherently complex nature of the phenomenon. Under these conditions, global forces are comparable to, and in some cases exceed, those measured for the flushed wall configuration. For larger retreats (panels d), e) and f)), the data shift progressively to the right of the bisector, indicating that global forces tend to decrease as G_c increases. The reduction rate also becomes more pronounced for larger retreats. These observations align with the findings of Romano and Bellotti (2023). However, due to the wide parameter range explored, some cases still exhibit global forces exceeding those of the flushed configuration, even for large retreats. This further confirms the strong variability of the process and the interplay of multiple competing mechanisms. For this reason, a statistical approach is adopted in the later part of the section.

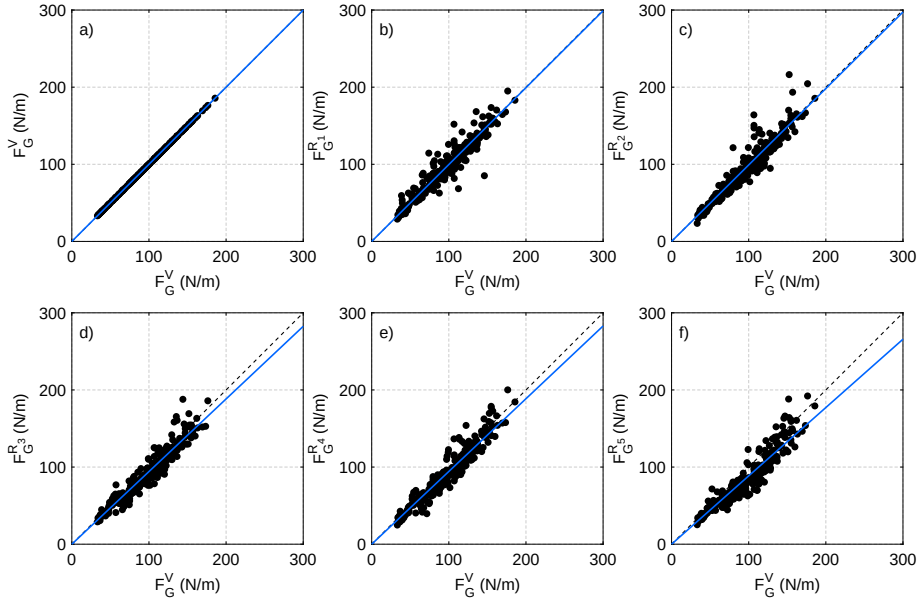


Figure 3.4: Scatter plots of the global forces (filled black markers). In the panels from b) (referring to G_{c1}) to f) (referring to G_{c5}), $F_G^{R_i}$ are plotted against F_G^V , while in the panel a) the same quantities (F_G^V , referring to G_{c0}) are reported in both the x- and y-axes. Note: the dashed black and continuous blue lines identify the bisector and trend lines, respectively.

The forces acting solely on the caisson trunk are presented in Figure 3.5. The layout mirrors that of the global force figure. Trunk forces, F_T^V and $F_T^{R_i}$, exhibit a behaviour comparable to the global forces. For small retreats, the data approximately follow the bisector, with smaller scatter than observed for global forces, which is consistent with the fact that global forces include impulsive contributions on the wall. As the wall retreat increases, the data migrate to the right of the bisector, indicating a reduction of trunk forces. The rate of reduction slightly increases with larger retreats, again consistent with previous findings. These results are also further examined through statistical indicators later on.

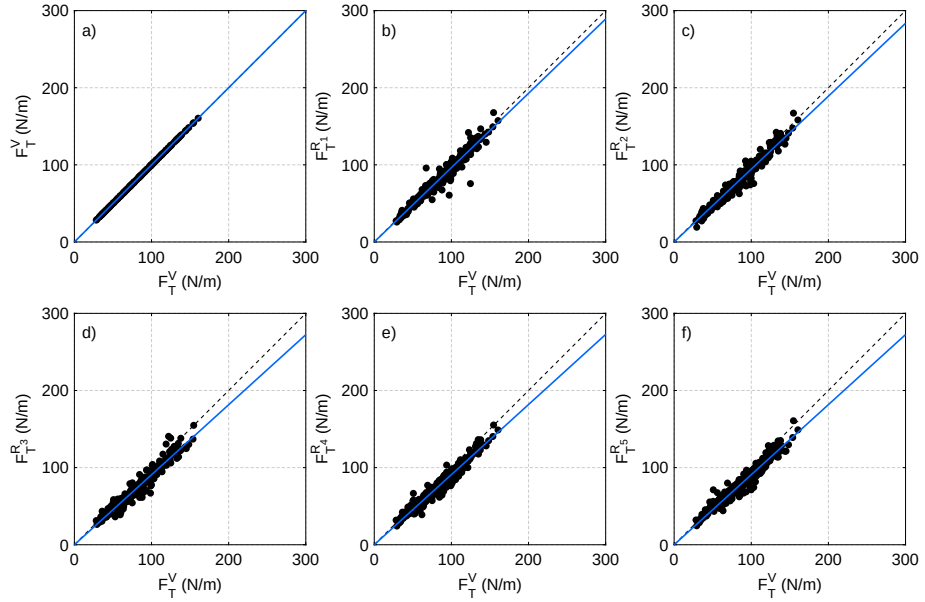


Figure 3.5: Scatter plots of the trunk forces (filled black markers). In the panels from b) (referring to G_{c1}) to f) (referring to G_{c5}), $F_T^{R_i}$ are plotted against F_T^V , while in the panel a) the same quantities (F_T^V , referring to G_{c0}) are reported in both the x- and y-axes. Note: the dashed black and continuous blue lines identify the bisector and trend lines, respectively.

Figure 3.6 displays the forces acting on the wave wall, F_W^V and $F_W^{R_i}$. No best-fit lines are shown here. The scatter patterns appear similar for all retreat distances tested. Even for small retreats (panels b) and c)), forces on the wall exhibit a significant increase compared to the flushed configuration. Moreover, the scatter is much larger than for global or trunk forces, reflecting the impulsive nature of the loads. As reported in Romano and Bellotti (2023), the wall retreat alters the flow regime over the superstructure, giving

rise to short duration, highly variable “church-roof-like” impacts on the wall. These impacts naturally lead to large variability in the measured forces. As discussed later, the average increase in wall forces does not grow substantially beyond a certain retreat distance.

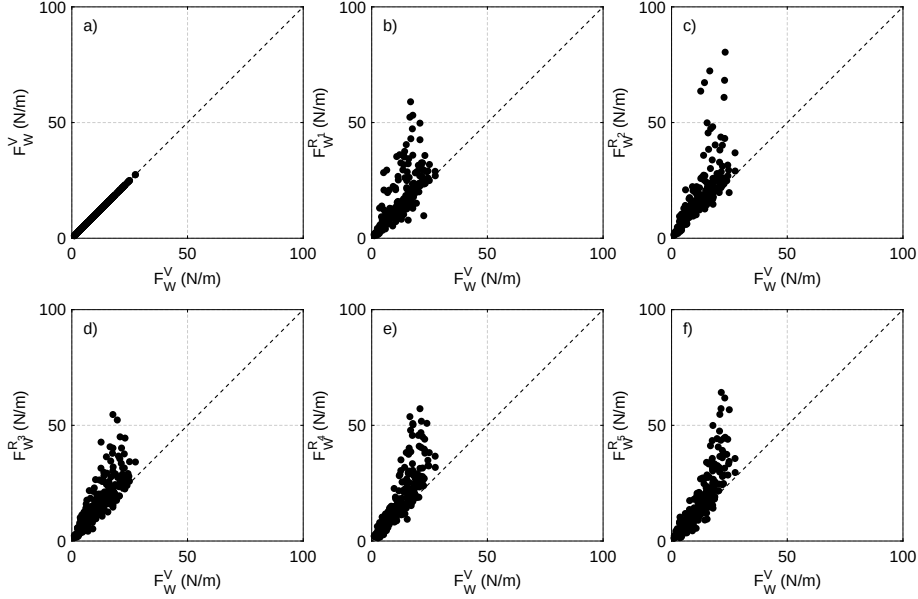


Figure 3.6: Scatter plots of the wall forces (filled black markers). In the panels from b) (referring to G_{c1}) to f) (referring to G_{c5}), $F_W^{R_i}$ are plotted against F_W^V , while in the panel a) the same quantities (F_W^V , referring to G_{c0}) are reported in both the x- and y-axes. Note: the dashed black line identifies the bisector line.

All the data represented in the Figures 3.4, 3.5, and 3.6 have been further processed as follows: for each force component (global, trunk and wall) the ratios between the considered quantity measured for all the retreated wall configurations ($F_G^{R_i}$, $F_T^{R_i}$ and $F_W^{R_i}$) and those for the flushed wall configuration (F_G^V , F_T^V and F_W^V), obtained for the same wave conditions and structural parameters, have been calculated. Then, some statistical descriptors and indicators (i.e., mean, median, 25th, and 75th percentiles) have been calculated for each set of force components ratios and are presented and discussed below.

Figure 3.7 shows the mean values of the force ratios (black empty markers) as a function of the dimensionless parameter G_c/h_w . In panel a), the ratio F_G^R/F_G^V is equal to 1 at $G_c/h_w = 0$ and remains close to unity for small retreats. As G_c/h_w increases, the ratio decreases nearly linearly, reaching

a minimum of about 0.87 for the largest retreat. A similar trend is observed for F_T^R/F_T^V (panel b): the ratio decreases approximately linearly up to $\leq G_c/h_w \leq 1.2$, attaining a minimum of about 0.91, and then remains nearly constant for larger retreats. The behaviour of the ratio F_W^R/F_W^V (panel c)) is markedly different: it rapidly increases with G_c/h_w reaching values of about 1.5. This suggests an “on-off” type mechanism governing the increase in wall forces, followed by a saturation effect for larger retreat distances.

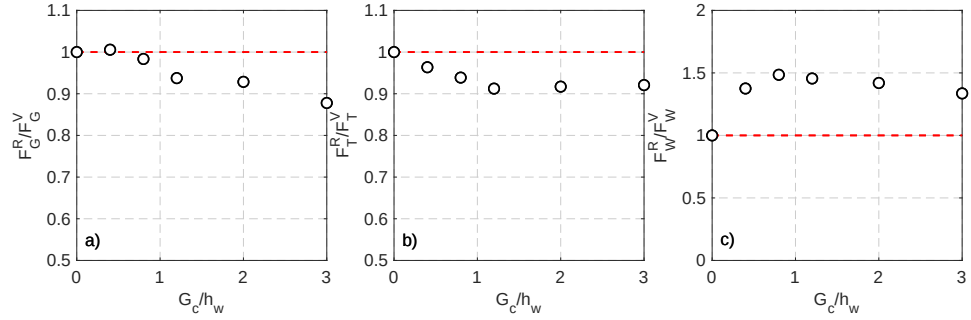


Figure 3.7: Average values (empty black markers) of the ratios F_G^R/F_G^V , F_T^R/F_T^V , and F_W^R/F_W^V (panel a), b), and c), respectively) as a function of the dimensionless parameter G_c/h_w .

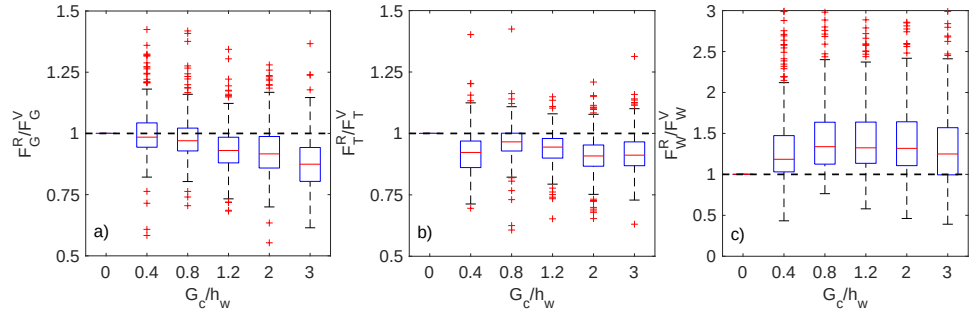


Figure 3.8: Boxplot of the ratios F_G^R/F_G^V , F_T^R/F_T^V , and F_W^R/F_W^V (panel a), b), and c), respectively) as a function of the dimensionless parameter G_c/h_w . Note: on each box, the central mark (red line) indicates the median, and the bottom and top edges of the box indicate the 25th and 75th percentiles, respectively. The whiskers extend to the most extreme data points not considered outliers, and the outliers are plotted individually using red cross markers.

To provide an indication of how the data are dispersed, the Figure 3.8 reports the boxplots as a function of G_c/h_w . On each box, the central mark (red line) indicates the median, and the bottom and top edges of the box

indicate the 25th and 75th percentiles, respectively. The whiskers extend to the most extreme data points not considered outliers, and the outliers are plotted individually using red cross markers. Similarly to what done in Figure 3.7, the panel a) refers to the ratios F_G^R/F_G^V , the panel b) to F_T^R/F_T^V , and the panel c) to F_W^R/F_W^V , respectively. These representations, together with the statistical indicators reported in the Tables 3.3, 3.4, and 3.5, can be useful for preliminary design purposes, as they provide a direct measure of the variability and the uncertainty associated with these phenomena. In particular, panel c) shows that the greatest variability and uncertainty, evidenced by the substantial gap between the 25th and 75th percentiles, concerns, as expected, the forces acting on the wave wall.

G_c/h_w	Mean	Median	25 th percentile	75 th percentile
0.00	1.00	1.00	1.00	1.00
0.40	1.01	0.98	0.94	1.04
0.80	0.98	0.97	0.92	1.02
1.20	0.93	0.93	0.87	0.98
2.00	0.92	0.91	0.85	0.98
3.00	0.87	0.87	0.80	0.94

Table 3.3: Statistical descriptors of the ratios F_G^R/F_G^V as a function of G_c/h_w .

G_c/h_w	Mean	Median	25 th percentile	75 th percentile
0.00	1.00	1.00	1.00	1.00
0.40	0.96	0.92	0.86	0.96
0.80	0.93	0.96	0.92	1.00
1.20	0.91	0.94	0.89	0.97
2.00	0.91	0.90	0.86	0.95
3.00	0.92	0.91	0.86	0.96

Table 3.4: Statistical descriptors of the ratios F_T^R/F_T^V as a function of G_c/h_w .

G_c/h_w	Mean	Median	25 th percentile	75 th percentile
0.00	1.00	1.00	1.00	1.00
0.40	1.40	1.18	1.03	1.47
0.80	1.50	1.33	1.12	1.64
1.20	1.45	1.32	1.13	1.64
2.00	1.42	1.31	1.11	1.64
3.00	1.34	1.25	1.00	1.57

Table 3.5: Statistical descriptors of the ratios F_W^R/F_W^V as a function of G_c/h_w .

3.3.2 Reflection coefficients

In this section, the influence of the wave wall position on the reflection coefficients is examined. For each experiment, the incident and reflected wave components were separated by means of the nonlinear method proposed by Andersen et al. (2017). The reflection coefficient, C_r , was then computed as the ratio between the reflected and the incident wave height H . The analysis was carried out over the same 20 s time window adopted for the force evaluation, during which stationary conditions are ensured. In the following, C_r^V denotes the reflection coefficient corresponding to the flushed wall configuration (G_{c0}), while $C_r^{R_i}$ refers to the five retreated wall configurations (G_{c1} - G_{c5}).

Figure 3.9 presents the results of the analysis. In line with the previous figures related to the forces, the experimental data are shown as black filled markers in scatter plot form, while the dashed and solid blue lines represent the bisector and the best-fitting line, respectively. The structure of the figure mirrors that of Figures 3.4, 3.5 and 3.6.

Panel a) of Figure 3.9 confirms that, for G_{c0} , all data lie exactly on the bisector, as expected. A different behaviour emerges when the wall is shifted landward (panels b) - f)). For small wall retreats (panels b) and c)), the data cloud, although characterized by noticeable scatter, tends to lie to the right of the bisector. This indicates that the reflection coefficient decreases, on average, as G_c increases. Even limited retreats perturb the pulsating flow regime typical of a flushed vertical wall, enhancing turbulence and dissipation and consequently reducing the reflected wave height. As the wall retreat increases further (panels d) - f)), this tendency becomes more marked: the data cluster progressively shifts to the right, as clearly shown

by the blue trend lines. In addition, the vertical spread of the measurements increases with G_c , and minimum reflection coefficients of the order of 0.5 are observed for the largest retreat distances. This trend, already noted by Romano and Bellotti (2023) based on a narrower experimental dataset, is corroborated by the present results.

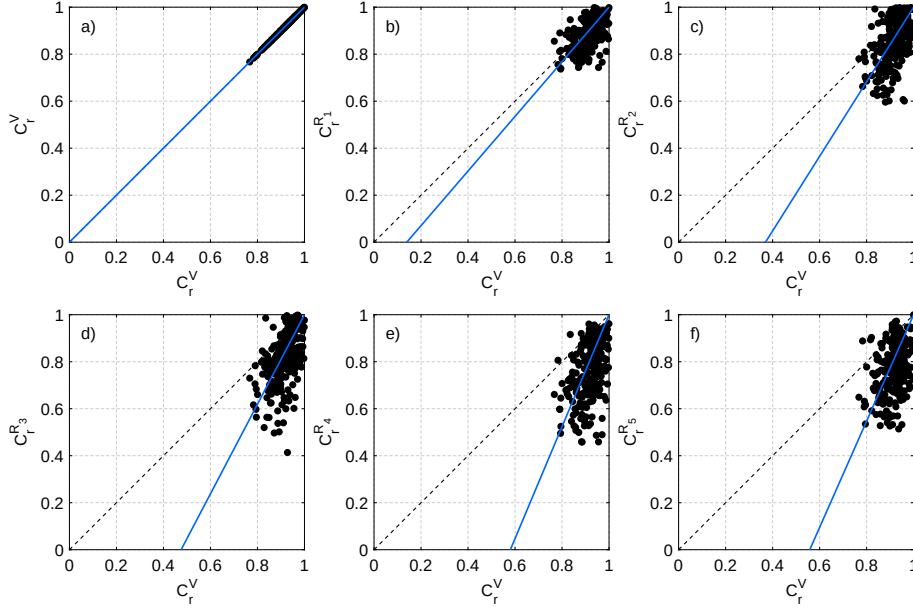


Figure 3.9: Scatter plots of the reflection coefficients (filled black markers). In the panels from b) (referring to G_{c1}) to f) (referring to G_{c5}), C_r^{Ri} are plotted against C_r^V , while in the panel a) the same quantities (C_r^V , referring to G_{c0}) are reported in both the x- and y-axes. Note: the dashed black and continuous blue lines identify the bisector and trend lines, respectively.

To quantify these effects, the ratios between the reflection coefficients measured for each retreated configuration (C_r^{Ri}) and those obtained for the flushed configuration (C_r^V) were computed, and statistical indicators were derived. As done for the force ratios, panel a) of Figure 3.10 reports the mean values of the ratios C_r^R/C_r^V (shown as empty black markers), whereas panel b) illustrates the distribution of the same ratios through boxplots as a function of G_c/h_w . The notation of the boxplots is consistent with that used in Figure 3.8.

From panel a), it is evident that the ratio C_r^R/C_r^V decreases almost linearly with G_c/h_w up to about 2, reaching a minimum of approximately 0.83. For $G_c/h_w > 2$, the ratio appears to stabilize, without further systematic

reduction or increase. So, it appears that, on average, also for the reflection coefficient, the reduction caused by the wall retreat seems to be limited and, at least for the considered range of parameters, does not further decrease beyond certain values of wall retreats. A similar behavior is confirmed looking at the boxplots represented in the panel b) of Figure 3.10. In fact, the medians of the C_r^R/C_r^V tends to decrease linearly up to $G_c/h_w = 2$ and then stabilize (see Table 3.6).

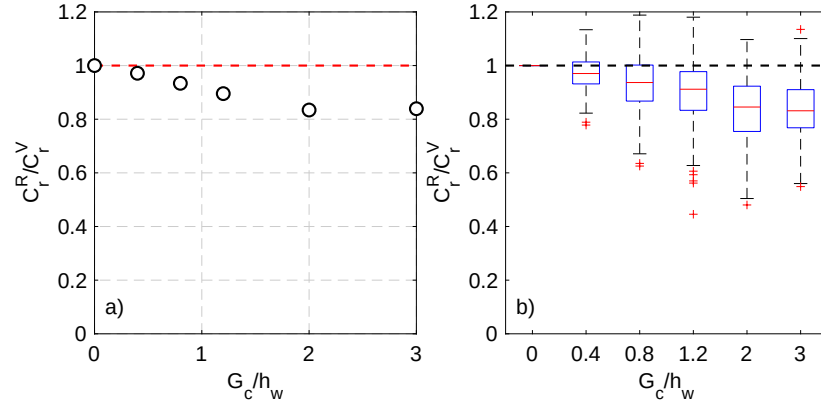


Figure 3.10: Panel a): average values (empty black markers) of the ratios C_r^R/C_r^V as a function of the dimensionless parameter G_c/h_w . Panel b): boxplots of the ratios C_r^R/C_r^V as a function of the dimensionless parameter G_c/h_w . Note: on each box, the central mark (red line) indicates the median, and the bottom and top edges of the box indicate the 25th and 75th percentiles, respectively. The whiskers extend to the most extreme data points not considered outliers, and the outliers are plotted individually using red cross markers.

G_c/h_w	Mean	Median	25 th percentile	75 th percentile
0.00	1.00	1.00	1.00	1.00
0.40	0.97	0.97	0.93	1.01
0.80	0.93	0.93	0.86	1.00
1.20	0.89	0.91	0.83	0.97
2.00	0.83	0.84	0.75	0.92
3.00	0.83	0.83	0.76	0.91

Table 3.6: Statistical descriptors of the ratios C_r^R/C_r^V as a function of G_c/h_w .

3.3.3 Wave overtopping

In this section, the analysis of the average wave overtopping discharge, q (l/m/s), as a function of the wall position is presented. The overtopped water was collected in the tank located behind the wave wall, and the total volume accumulated within the same 20 s stationary time window used for the force and reflection analyses was determined from measurements of the free surface elevation inside the tank. This volume was then divided by the width of the overtopping chute and by the duration of the selected time window, thus yielding the average overtopping discharge q for each experimental test.

Since regular waves were used and the analysis is performed over relatively short time windows, an important clarification is needed regarding the interpretation of the overtopping discharge. The values of q obtained here cannot be interpreted in the “classical” sense of mean overtopping discharge normally estimated through empirical formulae from the literature (EurOtop, 2018). It is well established that wave overtopping is a highly complex process, and the mean overtopping discharge is a hydraulic parameter that is typically defined under irregular sea states and is strongly affected by test duration, the number of waves, sequencing effects, and other factors not considered in the present study (Romano et al., 2015; Williams et al., 2019). Instead, the present values of q should be understood as comparative indicators of different structural layouts, following an approach similar to that adopted by Castellino et al. (2018, 2021). The goal of the analysis is therefore to assess how the overtopping behaviour of a caisson with a retreated wave wall differs from that of the same structure equipped with a flushed wall, under identical wave conditions.

Throughout the chapter, the symbols q^V and q^{R_i} denote, respectively, the overtopping discharge for the flushed wall configuration (G_{c0}) and that associated with the five retreated wall configurations (G_{c1} - G_{c5}). The analysis of the overtopping data is shown in Figure 3.11, which is organized in the same format as Figures 3.4, 3.5, 3.6, and 3.9. Overall, the overtopping discharge increases with increasing G_c , as clearly indicated by the slope of the trend lines. This behaviour, which was not investigated in Romano and Bellotti (2023) due to the absence of overtopping measurements, is nevertheless consistent with expectations. Indeed, the flow-structure interaction changes significantly when the wall is retreated: a bore with substantial horizontal momentum develops over the flat superstructure between the seaward

edge of the trunk and the wave wall. Upon impacting the wall, this bore appears to promote overtopping, particularly under “threshold conditions” where little or no overtopping would occur for the flushed wall geometry.

Figure 3.11 also shows that the experimental data are characterized by a considerable degree of scatter, as expected for a process as inherently variable as wave overtopping (Romano et al., 2015; Williams et al., 2019), even when regular waves are used. For small and intermediate wall retreats (panels b) - d)), the data cloud, despite its dispersion, reveals that overtopping discharge increases markedly compared with the flushed configuration. This is reflected in the consistent shift of the data to the left of the bisector, as emphasized by the trend lines. For larger retreats (panels e) and f)), the same trend persists but becomes less pronounced: although the scatter becomes even wider, the rate of increase in overtopping discharge appears to decrease.

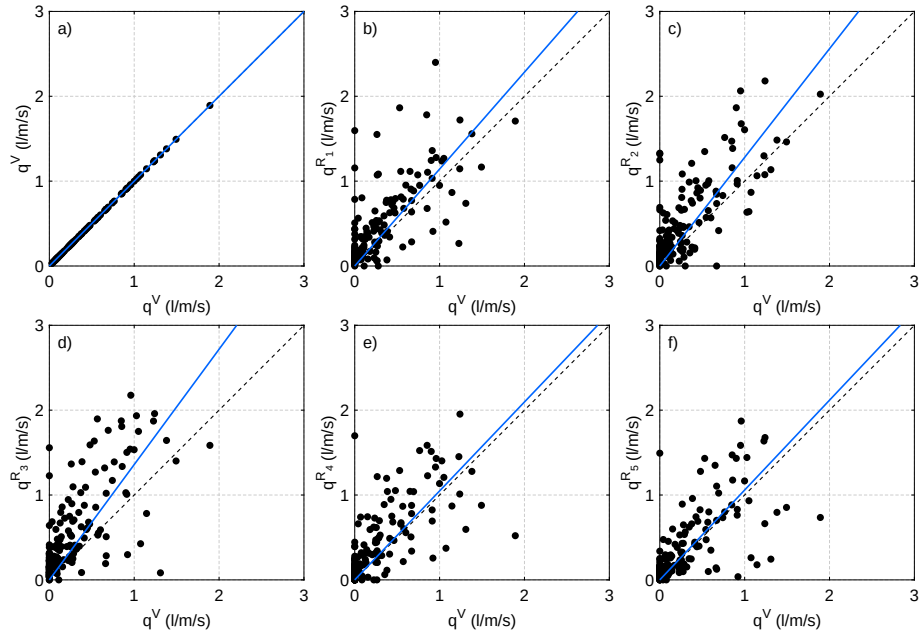


Figure 3.11: Scatter plots of the wave overtopping discharges (filled black markers). In the panels from b) (referring to G_{c1}) to f) (referring to G_{c5}), q^{R_i} are plotted against q^V , while in the panel a) the same quantities (q^V , referring to G_{c0}) are reported in both the x- and y-axes. Note: the dashed black and continuous blue lines identify the bisector and trend lines, respectively.

As done for the force and reflection analyses, the ratios q^R/q^V were computed for each test condition. Panel (a) of Figure 3.12 reports the mean values of these ratios (empty black markers), while panel (b) provides additional statistical descriptors in the form of boxplots. The notation used follows that of the previous boxplot figures. The ratio q^R/q^V increases rapidly with G_c/h_w , nearly linearly, up to a peak value of 2.55 at $G_c/h_w = 2$. Beyond this point, the ratio begins to decrease with a milder negative slope, reaching a minimum of 1.63 at $G_c/h_w = 3$ (corresponding to the largest retreat investigated, G_{c5}). A comparable behaviour is observed in the boxplots of panel (b). It is worth highlighting that these boxplots effectively capture the substantial variability and uncertainty associated with wave overtopping. In particular, large differences between the 25th and 75th percentiles are evident for configurations exhibiting the highest overtopping levels (see Table 3.7). Overall, the results confirm that, for certain wall positions, significant increases in overtopping may occur.

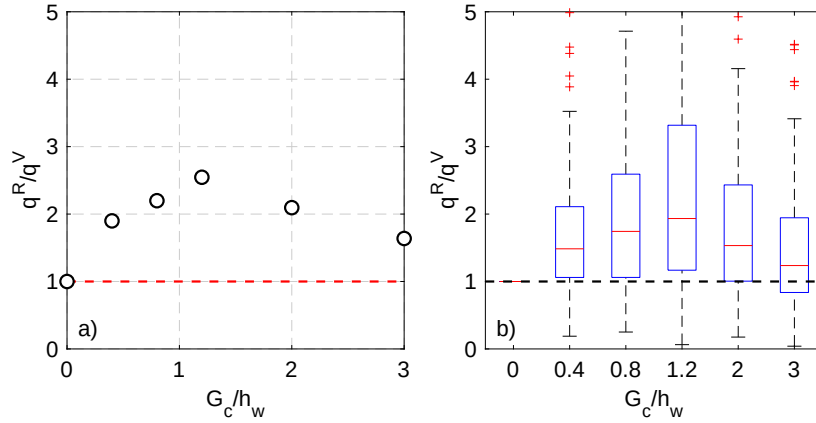


Figure 3.12: Panel a): average values (empty black markers) of the ratios q^R/q^V as a function of the dimensionless parameter G_c/h_w . Panel b): boxplots of the ratios q^R/q^V as a function of the dimensionless parameter G_c/h_w . Note: on each box, the central mark (red line) indicates the median, and the bottom and top edges of the box indicate the 25th and 75th percentiles, respectively. The whiskers extend to the most extreme data points not considered outliers, and the outliers are plotted individually using red cross markers.

G_c/h_w	Mean	Median	25 th percentile	75 th percentile
0.00	1.00	1.00	1.00	1.00
0.40	1.90	1.48	1.06	2.11
0.80	2.20	1.74	1.06	2.59
1.20	2.55	1.93	1.16	3.31
2.00	2.01	1.53	1.00	2.43
3.00	1.63	1.23	0.84	1.95

Table 3.7: Statistical descriptors of the ratios q^R/q^V as a function of G_c/h_w .

3.3.4 Limitations

In this section, the limitations of the present investigation, together with its consistency with previous findings, are discussed. The main limitation concerns the relatively small geometric scale of the model, which corresponds to approximately 1:100 when compared with the typical dimensions of vertical breakwaters. The motivation behind this choice is the following. The experimental observations reported by Romano and Bellotti (2023) highlighted the highly complex nature of the processes triggered by a retreated crown wall, which profoundly modifies the hydrodynamics of wave-structure interaction through the interplay of several, often competing, physical drivers. Although those drivers were clearly identified in the previous campaign, only a limited number of combinations of wave and structural parameters had been explored. Given the complexity and strong variability of the phenomenon, a large number of experiments were carried out in a small-scale wave flume, rather than performing a smaller number of tests in a larger facility as in the previous study. This strategy is commonly adopted when the primary objective is to unravel complex interaction mechanisms (Ravindar et al., 2021). The small-scale setup, identical to that used in Romano et al. (2015), provided the necessary flexibility, ensured short test durations, and substantially reduced the time required between consecutive experiments.

As a result, the present dataset is certainly affected by scale effects. However, it should be stressed that the aim of the study is not to determine absolute values of wave forces, reflection coefficients or overtopping discharges, nor to propose empirical design formulae. Instead, the focus is on comparing the hydraulic performance of a structure with a retreated wall against that of the same structure with a flushed wall under identical wave loading. Both configurations (flushed and retreated) are expected to be sim-

ilarly influenced by laboratory scale effects. However, some processes, such as air entrainment, bubble dynamics, and water compressibility, are more sensitive to scale than others (Bullock et al., 2001; Blenkinsopp and Chaplin, 2011; Ravindar et al., 2021). Nonetheless, the general trends, especially when expressed in comparative form, are expected to be captured reliably. Indeed, the new small-scale results confirm and extend the understanding of the complex physical mechanisms already identified by Romano and Bellotti (2023) at larger scale.

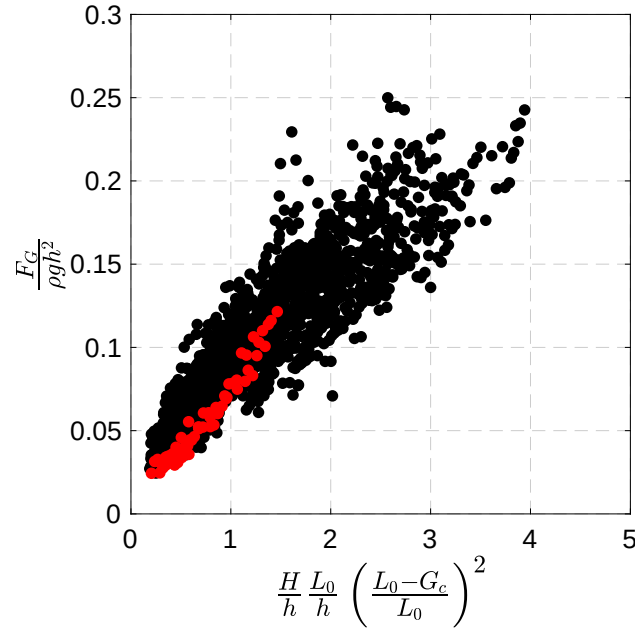


Figure 3.13: Dimensionless global $\frac{F_G}{\rho g h^2}$ forces obtained in all the experimental tests and for all the wall configurations as a function of the explanatory variable $\frac{H}{h} \frac{L_0}{h} \left(\frac{L_0 - G_c}{L_0} \right)^2$. Note: filled red and black circles represent data of Romano and Bellotti (2023) and new small-scale data, respectively.

In this regard, reference is made to Figure 3.13, which reproduces Figure 19 of Romano and Bellotti (2023). The figure reports the dimensionless global force $\frac{F_G}{\rho g h^2}$ obtained for all tests and for all wall configurations, plotted against the explanatory variable $\frac{H}{h} \frac{L_0}{h} \left(\frac{L_0 - G_c}{L_0} \right)^2$ introduced by Romano and Bellotti (2023). Filled red circles denote the large-scale data from Romano and Bellotti (2023), whereas filled black circles represent the new small-scale measurements. The figure clearly shows that the large-scale data fall within the cloud of the new results. Moreover, due to the greatly expanded range

of tested parameters and to the inherent complexity of the hydrodynamics, which leads to significant data dispersion, the adoption of a statistical approach, based on a large number of experiments, appeared more appropriate than attempting to identify explanatory variables and propose empirical formulations (e.g., Figures 21 and 22 in Romano and Bellotti, 2023).

In addition, the extensive number of tests performed with the small-scale setup represents a major advantage given the numerous sources of uncertainty associated with the studied phenomena (e.g., variability of impact loads, overtopping volumes, etc.), as documented in the literature on wave-structure interaction (Romano et al., 2015; Williams et al., 2019). The large dataset made it possible to compute robust statistical descriptors, providing confidence bounds for the hydraulic performance of caissons equipped with retreated walls, results that are directly relevant for engineering design.

Another limitation of the present work is the use of regular waves rather than irregular sea states. This choice was again motivated by the need to minimize the duration of individual tests, thereby enabling a very large number of experiments, and by the desire to avoid the interpretative uncertainties associated with wave randomness. The results cannot therefore be used directly to quantify the response of the structures under realistic sea states. Nevertheless, they can reliably be employed to compare the performance of different structural layouts under identical, albeit regular, wave conditions. A similar approach has been successfully adopted by Martinelli et al. (2018) and Castellino et al. (2021), who demonstrated that the maximum forces induced by irregular waves acting on a vertical breakwater can be satisfactorily reproduced using regular waves. Wave overtopping, however, is expected to be more sensitive to randomness and wave sequencing, and further research is required to investigate this hydraulic performance parameter more thoroughly.

3.4 Key insights of this chapter

In this chapter, the results of a new small-scale experimental campaign aimed at assessing the hydraulic performance, namely wave forces, reflection coefficients, and overtopping discharges, of composite vertical breakwaters equipped with a retreated wave wall have been presented and discussed. This campaign represents a substantial extension of the work by Romano and Bellotti (2023), which first identified the fundamental mechanisms in-

volved and the associated governing parameters.

Given the inherently complex nature of the hydrodynamic processes generated by a retreated wall, driven by the intricate interaction of multiple competing effects, the present campaign was designed to systematically explore a wide range of wave and geometric conditions. To this end, approximately 2000 small-scale tests with regular waves were conducted. The large experimental dataset enabled a robust statistical treatment, providing quantitative estimates of the increase or reduction in forces, reflection coefficients and overtopping discharges, relative to the flushed wall configuration, as a function of the wall retreat. These results offer valuable first step guidance for the preliminary design of such structures.

Overall, the experimental observations corroborate and broaden the understanding of the physical mechanisms previously highlighted by Romano and Bellotti (2023). Regarding wave forces, the wall loads were found to be, on average, roughly 1.5 times larger than those acting on a conventional flushed wall arrangement under the same wave conditions. This increase, which appears to follow an “on-off” mechanism, arises from impulsive impact conditions that occur for all wall-retreat configurations, and remains essentially constant even for large values of G_c . In contrast, the forces acting on the caisson trunk tend to decrease as the wall retreat increases, reaching an average minimum reduction factor of about 0.91. Beyond $G_c/h_w \approx 1.2$, this reduction factor remains almost unchanged. For the global forces, an overall decreasing trend with increasing G_c is observed, with a minimum reduction factor of 0.87 relative to the flushed configuration. It is important to note, however, that this general behaviour exhibits potentially critical exceptions: in the range $0 \leq G_c/h_w \leq 1$, some conditions produce global forces equal to or even exceeding those for the flushed wall. This phenomenon, already reported by Romano and Bellotti (2023), is attributed to the complex and sometimes antagonistic interaction among the governing drivers (e.g., impulsive wall impacts, phase shifts between wall and trunk loads, and pressure reduction near the seaward edge of the trunk), and deserves particular attention from a design standpoint.

With regard to wave reflection, the experimental results indicate that the reflection coefficient C_r generally decreases as the wall is retreated, reaching an average minimum ratio of 0.83. As observed for the trunk forces, this reduction tends to stabilize beyond a certain value of G_c , suggesting a limiting behaviour.

Wave overtopping, investigated here for the first time (it was not measured in Romano and Bellotti (2023)), also shows a clear dependence on the wall retreat. Despite the large intrinsic scatter that characterizes overtopping measurements, the results consistently demonstrate that retreated wall configurations experience larger overtopping discharges than flushed wall geometries subjected to the same incoming waves. This effect is especially evident under “threshold conditions”, where little or no overtopping would occur for a flushed wall. Quantitatively, the overtopping increase factor reaches a maximum mean value of 2.55 at $G_c/h_w = 2$, before decreasing to approximately 1.63 at $G_c/h_w = 3$ (corresponding to G_{c5} , the largest retreat tested).

To conclude, the combined evidence from Romano and Bellotti (2023) and from the present experimental campaign reinforces the idea that the wave-structure interaction processes occurring at breakwaters with retreated wave walls are highly complex. Consequently, particular caution is warranted when designing such structures. While the statistical indicators provided in this chapter may serve as useful preliminary design references, it is strongly recommended that dedicated physical or numerical model tests be planned for any specific engineering application.

While the results presented in this chapter provide a comprehensive overview of the governing trends through an extensive parametric investigation at small scale, they also highlight the intrinsic limitations associated with scale effects and highly simplified configurations. To address these aspects and to gain deeper insight into the underlying hydrodynamic processes, the following chapter focuses on a reduced number of representative test conditions, investigated at a larger physical scale. This approach allows for a more detailed and reliable characterization of wave-structure interaction mechanisms.

Chapter 4

Post-overtopping flows on composite vertical breakwaters with retreated wave walls: mapping of the hydrodynamic parameters

Parts of the work presented in this chapter have been included in the journal paper: Centorami, M., Romano, A., Cecioni, C., Bellotti, G. (2025). Experimental analysis of post-overtopping flows on composite vertical breakwaters with retreated wave walls: Mapping of the hydrodynamic parameters. Coastal Engineering, 104839.

4.1 Introduction

In the previous chapters, the general framework of coastal defense optimization and the preliminary results from small-scale experiments have been presented, with particular attention to the role of the retreated wave wall in modifying the hydraulic performance of composite vertical breakwaters. These early findings highlighted that such a configuration may reduce the loads acting on the caisson trunk, while introducing more complex hydrodynamic processes on the retreated wall.

Building on this background, the present chapter focuses on a crucial aspect of wave-structure interaction, namely the post-overtopping flows de-

veloping between the seaward edge of the caisson trunk and the retreated wave wall. As shown in Romano and Bellotti (2023) and in Chapter 3, the hydrodynamics of vertical breakwaters with retreated walls is largely governed by these flows, which directly affect both the impulsive loads acting on the wall and the so-called downfall pressures on the deck of the structure.

Historically, the hydrodynamics of post-overtopping flows has been mainly investigated in the field of naval and offshore engineering (where these flows are often called “green water” events). Specifically, several works focused on the study of the kinematic and dynamic (i.e., wave loads) properties of such flows on ships’ decks (Goda and Miyamoto, 1976; Buchner, 1995, 2002; Ryu et al., 2007; Lee et al., 2012; Song et al., 2015; Silva et al., 2017; Chen et al., 2019; Lee et al., 2020; Wang et al., 2023; Gao et al., 2025). Moreover, other studies focused on the classification of the different types of post-overtopping flows (Greco et al., 2005, 2007; Fontes et al., 2021).

These events can be mainly classified, by considering the interaction of the water flow with the deck’s edge, into three distinct categories, namely: Dam break (DB), Plunging-Dam break (PDB), and Hammer-Fist (HF). The transition between these types depends both on the wave steepness and the crest freeboard. These types, listed in order of increasing wave steepness, are differentiated on the basis of the propagation direction and on the characteristics of the flow front: I) DB-type has mainly an horizontal component and the front is compact and low-aerated; II) PDB-type, although having mainly an horizontal component, features a plunging-like front that induces the formation of a spherical or elliptical air cavity at the edge of the deck; III) HF-type, occurring under very steep wave conditions, is a sub-vertical water jet that abruptly collapses onto the deck.

As far as coastal structures are concerned, post-overtopping flows are less investigated, although some studies are available especially for smooth dikes (De Finis et al., 2020; Chen et al., 2021; Neves et al., 2021; Cao et al., 2022; Van Bergeijk et al., 2022; van Bergeijk et al., 2022). To the best of the author’s knowledge, there is a lack of literature on the study of post-overtopping flows on composite vertical breakwaters, especially if retreated wall configurations are used. Since, as highlighted by Romano and Bellotti (2023) and discussed in Chapter 3, the wave-structure hydrodynamics of vertical breakwaters with retreated walls is governed by the characteristics of post-overtopping flows, classifying these events is expected to improve the overall understanding of the phenomena and contribute to the development

of practical design tools for such structures. Moreover, the nature of the wave loads on the wall, and the related wave overtopping, has a direct influence also on the so-called “downfall pressures”, caused by the jet falling on the deck behind the wave wall. Such pressures can be also very large, as experimentally demonstrated by Wolters et al. (2005).

This chapter presents the results of a new 2D experimental campaign aiming at providing a comprehensive analysis of the post-overtopping flow events on composite vertical breakwaters with retreated wave walls. The methodology proposed follows a two-stage approach: firstly, the classification proposed in the literature by Greco et al. (2005, 2007); Fontes et al. (2021) is used to explore the onset of these types of events on such structures. To this end, in the first stage, the experiments have been carried out without the wave wall, using regular waves by varying the governing parameters (wave height, wave period, crest freeboard, etc.), to establish a baseline characterization. In the second stage, for each type of event (DB, PDB, HF), some selected representative conditions have been reproduced by varying the wave wall position (i.e., wall retreat).

The aim of this experimental campaign is to carry out an in-depth analysis of the processes occurring during one wave period (i.e., intra-wave process). Therefore, an advanced image-clustering analysis technique has been applied to visualize the process and estimate those quantities which are difficult to measure with direct measurement techniques: e.g., air content, presence of bubbles, etc. Moreover, the wave-induced pressures, including downfall on the deck, acting on the structure are measured, exploring how different flow types, wall retreats, and aeration levels affect impact loads and flow dynamics.

The detailed analysis of the post-overtopping flows dynamics, combined with the measurements of the pressures acting on the wave wall, allows to obtain a comprehensive parameter map, based on the flows classification and on the geometrical parameters of the problem. The map, which is inspired to that developed within the PROVERBS Project (PRObabilistic design tools for VERtical BreakwaterS) and reported by Oumeraci et al. (2001), is to provide practical guidelines to design these structures.

The chapter is structured as follows. After this introduction, the description of the methodology is given, covering the experimental setup and the data processing techniques (forces and image-clustering method). Then,

results and discussion section follows. Finally, key insights of this section close the chapter.

4.2 Methodology

4.2.1 Experimental setup

The experimental campaign described in this chapter were carried out in the medium-scale wave flume of Roma Tre University (see Figure 4.1, panel a)). Since the general characteristics of the facility and its wave generation and absorption systems have already been presented in Section 2.3.1, only the elements that are directly relevant to the present experimental campaign are described below.

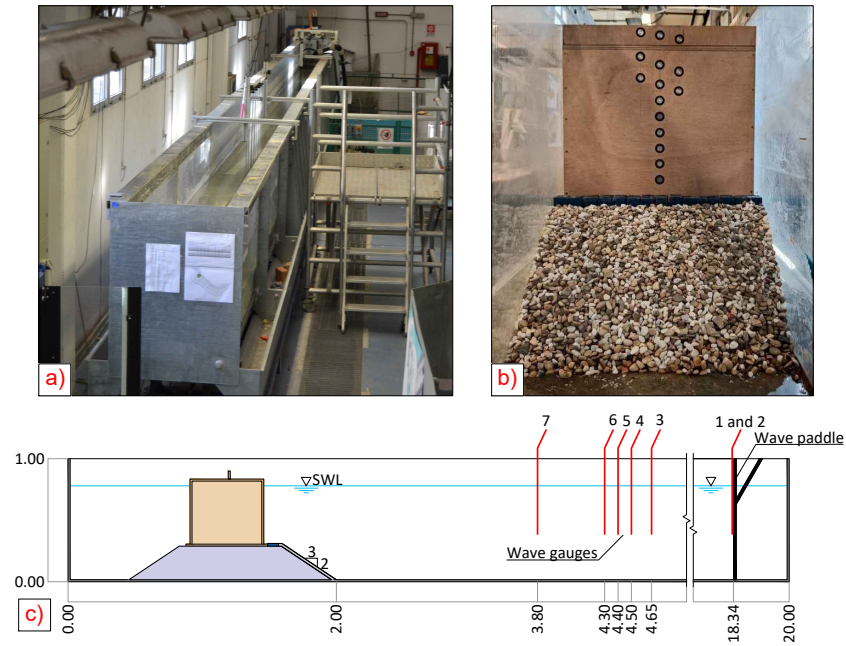


Figure 4.1: Panel a): Picture of the wave flume. Panel b): Front view picture of the composite vertical breakwater model. Panel c): Longitudinal section of the model structure and wave gauges setup in the wave flume.

The modeled structure replicates a vertical breakwater placed on top of a rubble mound foundation. Rather than replicating a specific prototype structure, the physical model is designed to represent, in Froude similarity, a broad range of vertical breakwater configurations in relatively deep wa-

ter. It might roughly resemble the new vertical breakwater of the Genoa port expansion project (Mediterranean Sea, Northern Italy), considering a reduction scale of approximately 1:55.

The vertical breakwater is made by a caisson trunk and of a superstructure with a vertical wall, both made of 0.015 m thick marine plywood (see Figure 4.1, panel b)). The caisson trunk is $h_t = 0.50$ m high and the crown wall is $h_w = 0.06$ m high; their width is 0.60 m to fit inside the wave flume. The caisson trunk and wall dimensions remain constant during all tests. The wave wall can easily moved and fixed at different horizontal positions to test different wall retreats with respect to the seaward face of the caisson trunk. The rubble mound foundation is $h_b = 0.25$ m high. It is made up of two parts: an inner core of uniform-sized rock material and an external rubble mound layer made of rocks with weight range of 20.0-30.0 g, shown in Figure 4.1, panel b).

Panel c) of Figure 4.1 reports a sketch of the longitudinal section of the breakwater and wave gauge setup, inside the wave flume, with measures in meters. A total of seven resistive wave gauges are placed along the flume to measure the free surface elevation. Two wave gauges are mounted on the piston-type wave-maker for the active absorption system (WG1 and WG2). Five wave gauges have been aligned in the central longitudinal axis of the flume, to measure the wave field in front of the structure (WG3-WG7). The acquisition frequency of the wave gauges is set at 50 Hz, and a calibration procedure is carried out at least twice a day to account for environmental variations.

A total of 21 pressure transducers are fixed on the structure. As illustrated in the longitudinal views in Figure 4.2, panels a) and b), 12 sensors (P1-P12) are placed on the caisson trunk seawards facing, 3 (P13-P15) on the crown wall and 6 (P16-P21) on the top of the caisson to measure the pressures induced by the post-overtopping volumes of water falling on the deck of the structure (i.e., downfall pressures). The position of these sensors is also shown in panels c) and d) of Figure 4.2, from a front view and a plan view, respectively. To maximize the number of sensors on the crown wall and around the mean sea level, the mutual vertical distance between the pressure sensors is not constant, spanning in the range 0.01 m-0.05 m. Table 6.1 reports the names, positions and influence area of the pressure sensors, differentiating the front sensors (P1-P15) from the top sensors (P16-P21). The acquisition frequency of all these pressure transducers is 7 kHz to ac-

curately measure impulsive load peaks.

A particular care has been adopted in minimizing potential laboratory effects. In particular, to equip the caisson trunk and wall with a large number of pressure measurement points and to ensure that its seaside face was as rigid and flat as possible, some special pieces (or masks) have been ad-hoc designed and 3D-printed for housing the pressure sensors (Similar to what have been done in Chapter 3). These masks, shown in panel b) of Figure 4.1, ensure the perfect alignment between the pressure sensors and the vertical front face of the structure.

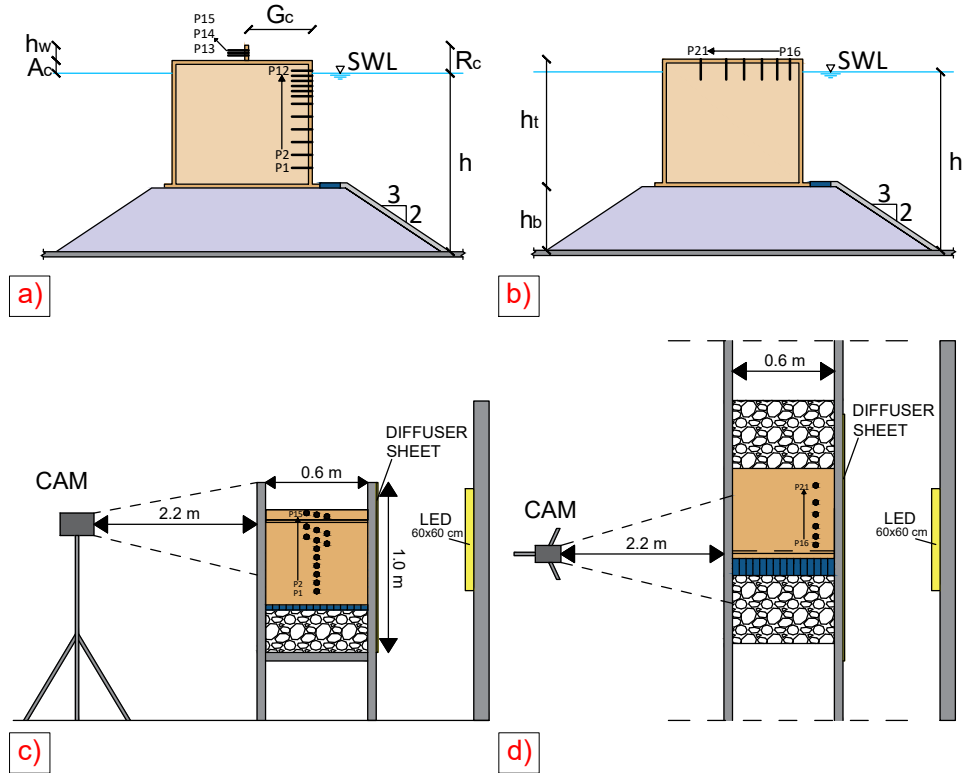


Figure 4.2: Top panels: longitudinal view of the composite breakwater, with indication of the geometrical parameters and positions of the front (panel a)) and top (panel b)) pressure sensors respectively. Bottom panels: front (panel c)) and plan (panel d)) views of the experimental set-up.

An optimized setup was designed to accurately video record the dynamic flow phenomena during the experiments. To this end, a camera was placed at a distance of 2.2 meters from one side the wave flume, pointing perpendicularly to the physical model (Figure 4.2 panels c) and d)). To ensure

consistent and clear visual data, an illumination system, composed of 60 cm x 60 cm LED panel with 45 W power, was installed on the opposite side of the wave flume with respect to the camera, providing uniform lighting across the observed area. Furthermore, a diffuser paper sheet was placed between the channel and the LED panel to evenly distribute the light and reduce reflections or shadows. This setup helped to distribute the light evenly and minimized reflections on the water surface, which was particularly important for maintaining consistent pixel intensity in the captured images. This arrangement was critical for achieving high-quality images that would be used for the segmentation of air bubbles, water, and background regions.

Front pressure sensors		
Pressure sensor	Vertical position (m)	Influence area (m^2/m)
$P1$	0.080	0.090
$P2$	0.130	0.050
$P3$	0.180	0.050
$P4$	0.230	0.050
$P5$	0.280	0.050
$P6$	0.330	0.040
$P7$	0.360	0.025
$P8$	0.380	0.020
$P9$	0.400	0.020
$P10$	0.420	0.020
$P11$	0.440	0.020
$P12$	0.460	0.050
$P13$	0.520	0.025
$P14$	0.530	0.010
$P15$	0.540	0.025

Top pressure sensors		
Pressure Sensor	Horizontal position (m)	Influence area (m^2/m)
$P16$	0.050	0.075
$P17$	0.100	0.055
$P18$	0.160	0.065
$P19$	0.230	0.070
$P20$	0.300	0.075
$P21$	0.400	0.150

Table 4.1: Pressure sensor names (first column), positions (second column) and areas of influence (third column). For the sensors placed on the seaward face of the caisson trunk ($P1$ - $P15$), the position is the vertical distance from the bottom corner of the trunk. For the sensors placed on the deck ($P16$ - $P21$) the position is the horizontal distance from the trunk sea-face.

4.2.2 Test program

Table 4.2 lists the ranges of the wave and structure parameters for the 97 tests performed. Regular waves were used in all tests to ensure controlled and repeatable conditions, allowing for a systematic analysis of the sea state influence without adding the complexity of irregular wave patterns. The duration of each test was approximately 80 s to obtain a wave time series long enough to reach stationary conditions acting on the structure.

As can be noted in Table 4.2, 52 tests were conducted varying wave conditions and water depth on the same composite vertical breakwater ($h_b = 0.25$ m and $h_t = 0.50$ m) without the crown wall. During these experiments, wave height H , wave period T and water depth h were systematically varied to explore the onset of the different types of post-overtopping flow events (DB, PDB, HF) and to establish a baseline characterization. The ranges of these parameters are reported in Table 4.2, along with dimensionless values. In particular, three values of water depth are tested, $h = 0.67, 0.70, 0.73$ m, corresponding to three values of crest freeboard $R_c = 0.08, 0.05$ and 0.02 m. In Table 4.2, L_0 is the deep water wave length, defined as $gT^2/(2\pi)$, with g the gravity acceleration. As stated, these tests were planned in a structural configuration without the retreated crown wall (in the following referred to as “baseline configuration”), to characterize at first the post-overtopping events varying the incident wave conditions.

Subsequently, among these 52 tests, 15 were selected as representative wave conditions of each post-overtopping flow type, and reproduced after installation of the wave wall on top of the structure. Specifically, three different configurations were tested, varying the horizontal position of the wall, G_c , with respect to the seaward face of the caisson: $G_{c1} = 0.08$ m (“small” wall retreat), $G_{c2} = 0.18$ m (“intermediate” wall retreat), and $G_{c3} = 0.35$ m (“large” wall retreat), leading to a total of 45 tests. For consistency, all these 45 tests were conducted at a fixed water depth of $h = 0.70$ m, as this condition exhibited the most relevant and interesting hydrodynamic interactions.

Tests without the crown wall (N° tests = 52)		
Parameters	Min. Value	Max. Value
H (m)	0.0695	0.1830
T (s)	0.8000	1.3000
h (m)	0.6700	0.7300
H/h (-)	0.1038	0.2581
h/L_0 (-)	0.2541	0.7312
H/L_0 (-)	0.0383	0.1093
R_c/H (-)	0.1093	1.1507
h_b/h (-)	0.3425	0.3731

Tests with the crown wall (N° tests = 45)		
Parameters	Min. Value	Max. Value
H (m)	0.0790	0.1619
T (s)	0.8000	1.3000
H/h (-)	0.1129	0.2313
h/L_0 (-)	0.2655	0.7011
H/L_0 (-)	0.0394	0.0981
R_c/H (-)	0.3088	0.6329
G_c/L_0 (-)	0.0303	0.3506

Table 4.2: Ranges of explored parameters tested during the experimental campaign.

4.2.3 Data processing

4.2.3.1 Forces analysis

The approach used in this study to measure wave-induced loads on the vertical caisson and on the wave wall follows the procedure outlined in Romano and Bellotti (2023) and in the same of the previous Chapter 3. Each test includes an initial transient period, followed by a steady wave phase of about 20 s during which the pressure signals are nearly repetitive from cycle to cycle. The recorded pressures p_i , from each sensor i are integrated according to a rectangular scheme, associating to each sensor an “influence area”, defined as half the distance above and half the distance below that sensor. Summing these contributions provides the instantaneous horizontal force on the structure:

$$F(t) = \sum_i p_i(t) \Delta A_i \quad (4.1)$$

where ΔA_i denotes the influence area of sensor i . By isolating the sensors placed on the trunk from those on the wave wall, it is possible to compute separate force contributions. Once these force time series are obtained, a

peaks-over-threshold (POT) analysis is performed to identify the load peaks in each wave cycle. Under regular wave conditions, these peaks are generally consistent, so their average is taken to represent the peak load. Table 6.1 reports the names, positions and influence area of the pressure sensors.

4.2.3.2 Image analysis

Following the framework proposed in Formentin et al. (2021) and later refined in Formentin et al. (2024), an image-based clustering technique was adopted to quantify the air content in the overtopping flows impacting the retreated wave wall. A Sony FDR-AX53 camera (8.29 megapixels, 25 fps, 1920x1080 pixels) was positioned as illustrated in Figure 4.2, panels c) and d). The Region of Interest (ROI), fixed for each test (Figure 4.3, panel a)), extends from the horizontal position of the crown wall toward the seaward side. It encompasses the entire useful field of view where reflected jets re-enter the sea and generate significant aeration, as reported by Romano and Bellotti (2023). Specifically, the visible area to the right of the caisson trunk covers approximately 0.40 m in the horizontal (x) direction and 0.33 m vertically (y), plus the additional rectangle associated with the retreat G_c . Pre-clustering steps included static background subtraction to isolate dynamic flow features, conversion to gray-scale to simplify the segmentation, and noise-reduction procedures. In particular, median and Gaussian filtering were applied to suppress salt-and-pepper noise and smooth high-frequency disturbances. Pixel intensities were then amplified to enhance contrast, and thresholding was applied to remove outliers (Figure 4.3, panel b)). The core step consisted of K-Means clustering algorithm (Forgy, 1965; Lloyd, 1982), used to segment each frame into three classes: (i) background (darkest pixels), (ii) aerated flow regions (highest intensities), and (iii) intermediate tones typically associated with free-surface reflections or refraction effects. Pixel intensity served as the main classification feature and was complemented by edge-gradient information to improve the identification of transitions between regions.

Post-clustering refinements included the removal of isolated artifacts smaller than 200 pixels through standard morphological operations (Figure 4.3, panel c)). To further sharpen the flow boundaries, Canny's edge detection algorithm (Canny, 1986) was applied (Figure 4.3, panel d)). The water region was not identified directly by clustering but rather defined geometrically as the area bounded by the free surface and the structure. The

resulting segmented frames were used to compute two quantities: (i) the fraction of fluid relative to the total area in the ROI, $\left(\frac{P_{fluid}}{P_{tot}}\right)$; (ii) the relative amount of air pixels within the fluid region, $R_a = \left(\frac{P_{air}}{P_{fluid}}\right)$, hereafter referred to as the air content. Tracking these indicators over time allowed quantification of the influence of aeration levels on wave impact forces, as discussed later.

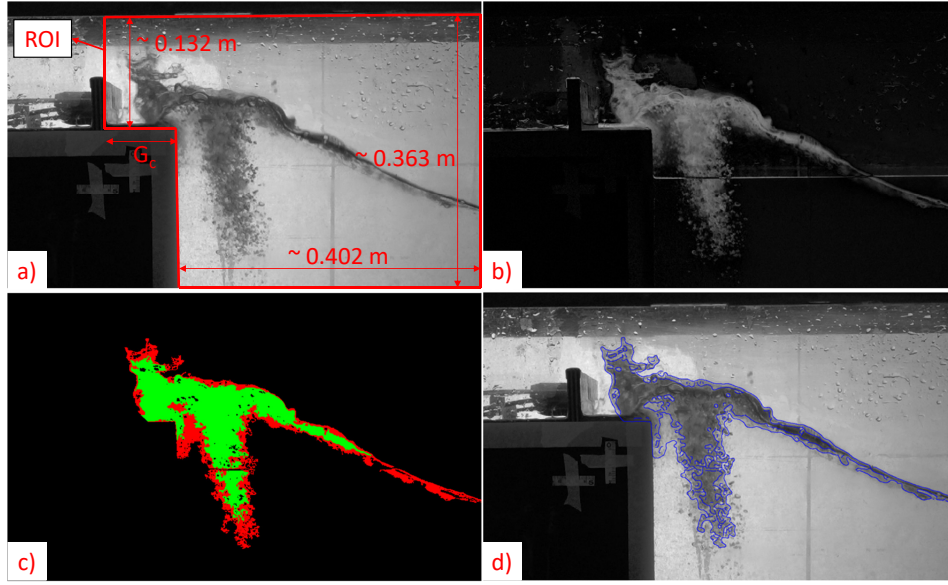


Figure 4.3: Panel a): example of a gray-scale frame with ROI definition. Panel b): example of a frame after the pre-clustering optimization. Panel c): example of a frame at the end of the method (k-means algorithm). Panel d): definition of cluster edges.

4.3 Results and discussion

4.3.1 Classification of post-overtopping events

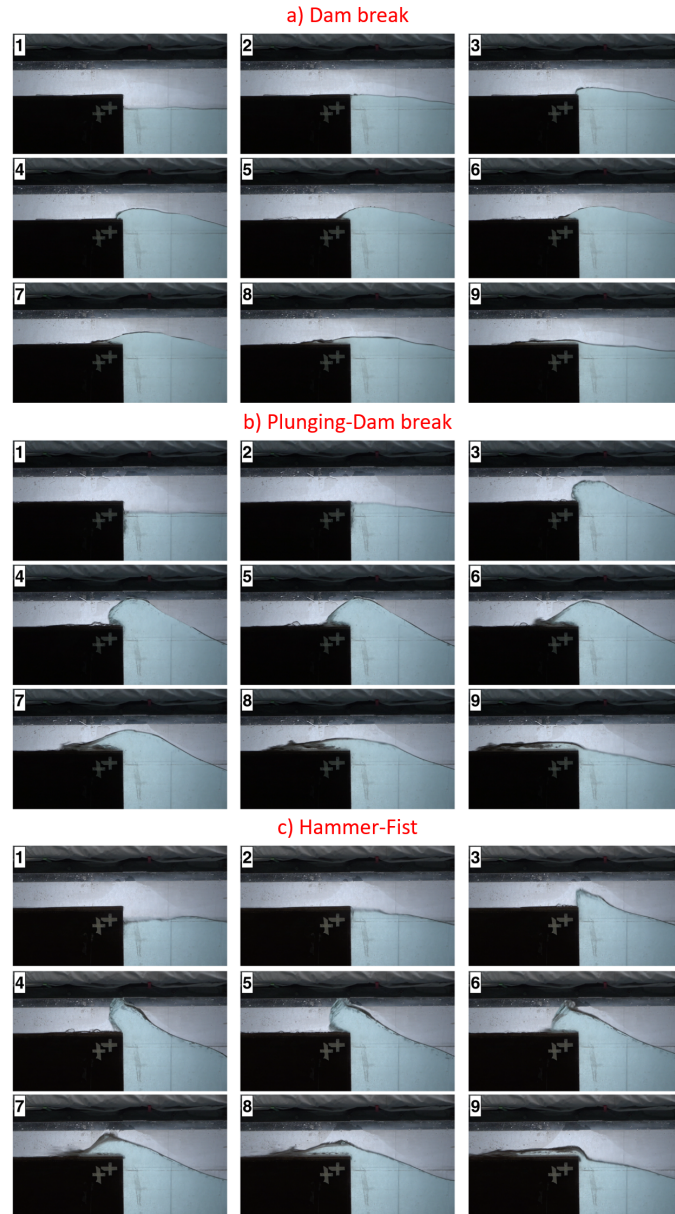


Figure 4.4: Representative example of post-overtopping event-type interacting with the baseline configuration (no-wall). From top to bottom: a) Dam break-type ($H/L_0 = 0.039$, $A_c/H = 0.483$) ; b) Plunging-Dam break-type ($H/L_0 = 0.082$, $A_c/H = 0.321$) c) Hammer-Fist-type ($H/L_0 = 0.096$, $A_c/H = 0.335$).

In this section the classification of post-overtopping flow events (DB, PDB, HF) of the 52 wave tests carried for the baseline configuration (i.e., without the wave wall) is presented. It has been carried out by a visual analysis of the flow, according to the criteria reported by Greco et al. (2005, 2007); Fontes et al. (2021). Examples of the three types of events are reported in Figure 4.4, where three panels containing 9 frames are presented for each event type, namely: DB (panel a)), PDB (panel b)), and HF (panel c)). Figure 4.4 shows the onset of the different types of post-overtopping flows at composite vertical breakwater having the characteristics described in the literature. Another schematic sketch is represented in Figure 4.5.

Once the post-overtopping flows events have been visually classified, it is of interest to better explore their characteristics in terms of waves and structural parameters. To this end, the Figure 4.6 is used. In the figure, both the panels report on the x-axis the wave steepness $\left(\frac{H}{L_0}\right)$, while on the y-axis the ratio between the height of the trunk freeboard A_c (measured from the undisturbed water level) and the incident wave height H is represented. The left panel refers to the 52 baseline configuration tests, therefore $A_c = R_c$. The post-overtopping flows, previously classified, are reported with different markers, namely: DB (circles), PDB (diamonds), and HF (triangles). Three distinct regions, each pertaining to a different type of event, are visually identified with dashed black lines. Specifically, the line that separates DB from PDB is defined by the threshold $H/L_0 = 0.045$, while the one that divides PDB from HF follows the linear relation $A_c/H = -17.33(H/L_0) + 1.88$. Although the wave steepness is crucial in triggering different event types, the combined effect of this quantity and of the structure geometry governs the transition especially from PDB to HF. As previously mentioned, among these 52 tests, 15 were selected as representative wave conditions of each post-overtopping flow type, and are carried out again after installing the wave wall on top of the structure. These events are identified with red markers in the left panel.

The right panel reports the test configurations with retreated wave wall, thus $A_c = R_c - h_w$. Here are shown the selected subset of the new tests (red dots) and all the data obtained during the previous experimental campaigns, which have been retrospectively classified according to the post-overtopping flow event types. These are represented as black dots (Romano and Bellotti, 2023) and gray dots (small-small experiments of Chapter 3), respectively.

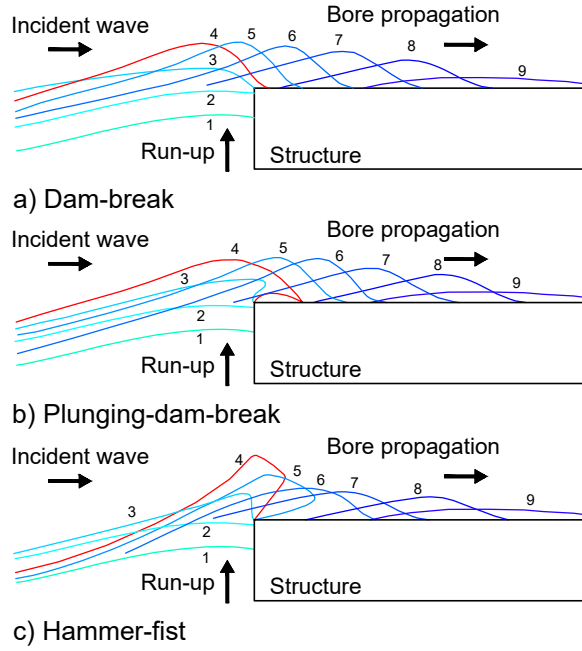


Figure 4.5: Sketch of representative post-overtopping event (adapted from Fontes et al., 2022).

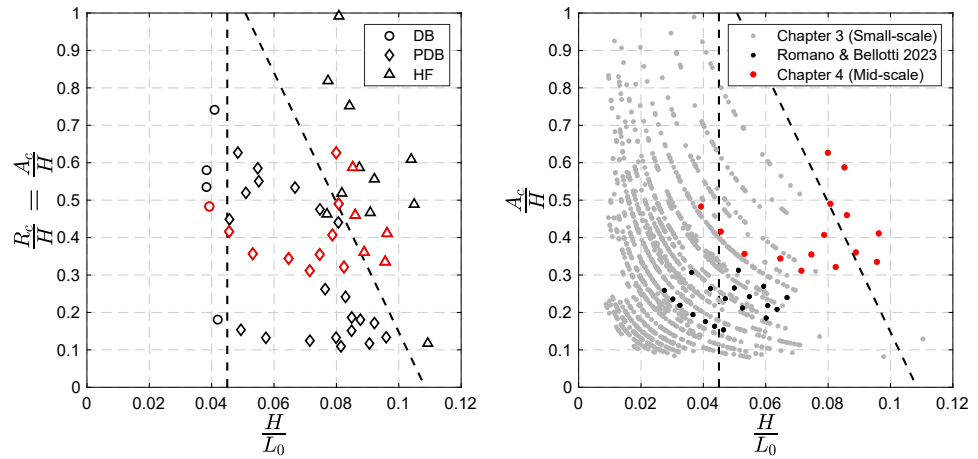


Figure 4.6: Left panel: classification of the 52 new tests carried out without a wave wall (DB, circles; PDB, diamonds; HF, triangles). The dashed black lines divide the three distinct regions pertaining to each event type. Red markers identify the subset made of 15 selected representative wave conditions to be reproduced with wave wall. Right panel: selected subset of the new tests (red dots) and all the data obtained during the previous experimental campaigns, black dots refer to Romano and Bellotti (2023) and gray dots to Chapter 3.

4.3.2 Hydrodynamic mapping of wave forces on retreated walls

In this section, the results related to the tests with the wave wall are presented and discussed, considering the subset of 15 runs repeated for three wave wall retreat configurations, obtaining a total number of 45 tests. Since regular waves are used, the results are analyzed in a time window that has a duration of one wave period. Within this window, both wave-induced loads and image-based analysis of the wave-structure interaction process are discussed.

An example of the analysis carried out is given in the Figures 4.7, 4.8, and 4.9. These three figures refer to different representative post-overtopping event types selected among the subset, specifically Figure 4.7 for DB, Figure 4.8 for PDF and 4.9 for HF. Note that the classification (DB, PDB, and HF) is retained throughout the following discussion, although the presence of the wave wall would alter the evolution of post-overtopping flows compared to the baseline configuration. The figures are made of three row panels, each referring to a different G_c and organized as follows: on the left nine selected time stacks of the wave-structure interaction process captured by the camera; on the right a dual-axis plot, for which the x-axis represents the time normalized with respect to the wave period (t/T). The left y-axis reports the time series of the wave-induced forces on the wall (black line). The right y-axis displays two air-content indicators: $\frac{P_{fluid}}{P_{tot}}$ (blue line) and R_a (red line). Moreover, stars and numbers identify the selected time instant represented in the time stacks. Finally, the vertical dashed gray line refers to the time at which the maximum force on the trunk occurs (t_{F_T-max}).

This combined representation helps to visualize and interpret how the presence of the wave wall affects both the magnitude and timing of impacts, as well as the air content in the overtopping flow. In fact, compared to the baseline configuration, the post-overtopping flows encounter an obstacle that significantly alter their hydrodynamics and the air content. In the following sub-sections each event type is analyzed separately to explore the effect of the wave wall retreat position.

4.3.2.1 Dam break-type events

Figure 4.7, representative of a BD-type event ($H/L_0 = 0.039$, $A_c/H = 0.483$), shows, as expected, that the magnitude and timing of the force peaks on the wall vary according to the wall position. The maximum force on the wave wall ranges from about 15 N/m (G_{c3}) to 25 N/m (G_{c1} and G_{c2}). The flow exhibit a low amount of air, with an estimated air content always smaller or equal than 10% during the impact on the wall. A clear time shift between forces on the caisson trunk and on the wave wall is evident for G_{c2} and G_{c3} configurations, while for G_{c1} , the these forces appear nearly in phase.

As far as the nature of the forces signal is concerned, it should be noted that non-impulsive conditions are detected for the wall retreat positions considered. In fact, the force increases gradually and is characterized by smooth peaks. The shape of the force signal is always double-peaked, where in general the first is larger, although it can happen that the second one, caused by the jet falling back after hitting the wall exceeds the first (Peregrine, 2003; Greco et al., 2005).

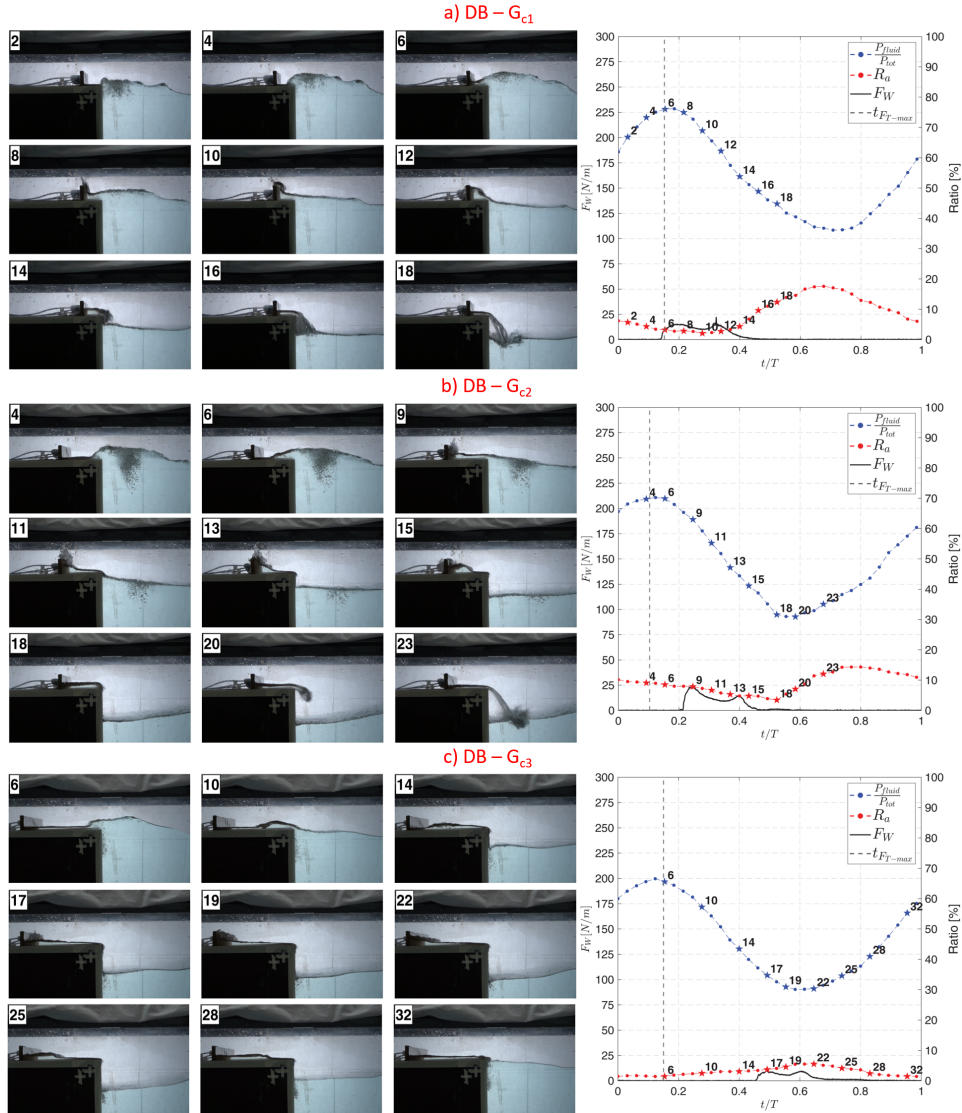


Figure 4.7: Representative DB-type event ($H/L_0 = 0.039$, $A_c/H = 0.483$) across three wall retreat positions: a) G_{c1} , b) G_{c2} , c) G_{c3} . Each panel shows 9 time stacks (left) and synchronized signals (right): wall force (black line), $\frac{P_{fluid}}{P_{tot}}$ (blue line), and R_a (red line). Time is normalized by T ; stars mark selected frames; dashed gray line indicates the time of the peak trunk force.

4.3.2.2 Plunging-Dam break-type events

Figure 4.8, representative of a PBD-type event ($H/L_0 = 0.082$, $A_c/H = 0.321$), shows that for this event the force peaks are significantly larger than those observed for DB. For G_{c1} it reaches a value of about 300 N/m , while it drops to around 75 N/m and 30 N/m for G_{c2} and G_{c3} , respectively. The nature of the forces signal is impulsive (church-roof-like) for G_{c1} , while it has the characteristics of a quasi-static signal for both G_{c2} and G_{c3} . This behavior is justified looking at the evolution in time of the flow. For G_{c1} the impinging jet, after overtopping the edge of the trunk, immediately clashes with the wave wall, inducing an impulsive load. Moreover, as observed for the DB-type event, the time-shift between forces on the wall and on the trunk is negligible for G_{c1} , while is relevant for increasing wall retreat G_{c2} and G_{c3} . Note that this configuration of waves and structural parameters represents one of those “worst-case scenarios” discovered by Romano and Bellotti (2023). In these conditions the forces acting on the wave wall, which exhibit an impulsive nature, are in phase with those acting on the trunk, inducing a general loading configuration of the structure that is way more severe than that occurring for a flushed-wall configuration under the same wave conditions.

On the contrary, for G_{c2} and G_{c3} , the increasing wall retreat, together with the interaction between the incoming and the reflected jets (see frames 4 and 25, panel b) and frame 6, panel c)), favor the dissipation of the post-overtopping flow, resulting in reduced values of the force on the wall. This behavior is consistent with the preliminary findings of Romano and Bellotti (2023). As for the air content, Figure 4.8 shows that, regardless the position of the wall, it is generally larger than that observed for DB, spanning in the range 10-25%.

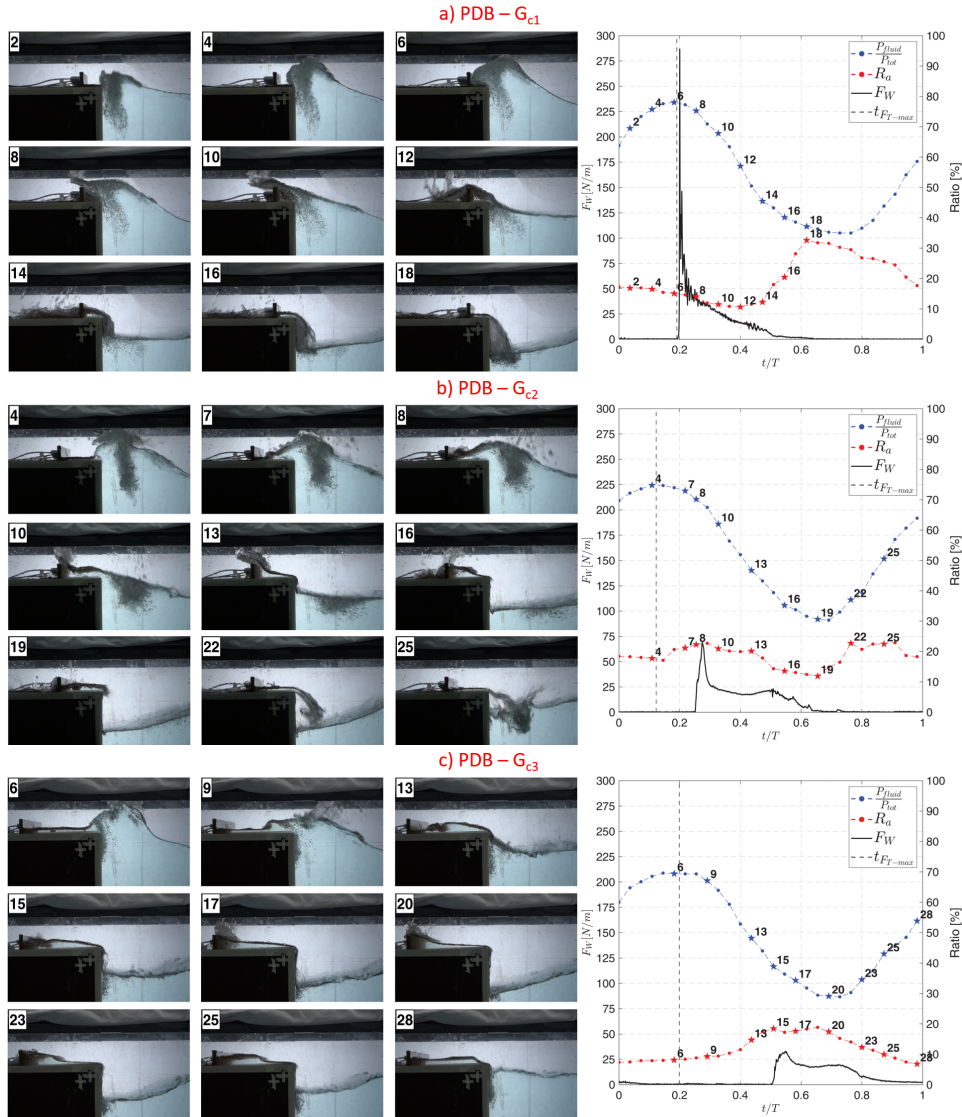


Figure 4.8: Representative PDB-type event ($H/L_0 = 0.082$, $A_c/H = 0.321$) across three wall retreat positions: a) G_{c1} , b) G_{c2} , c) G_{c3} . Each panel shows 9 time stacks (left) and synchronized signals (right): wall force (black line), $\frac{P_{fluid}}{P_{tot}}$ (blue line), and R_a (red line). Time is normalized by T ; stars mark selected frames; dashed gray line indicates the time of the peak trunk force.

4.3.2.3 Hammer-Fist-type events

Figure 4.9, representative of a HF-type event ($H/L_0 = 0.096$, $A_c/H = 0.335$), highlights that this event type has several points in common with PDB. The force peak is rather impulsive for G_{c1} , reaching a maximum value of about 190 N/m , while drops down to 75 N/m and 50 N/m for G_{c3} and G_{c2} , respectively. This dynamics is very similar to that described for PDB and it is related to the interaction between the incoming and the reflected jets (see frames 4 and 25, panel b) and frame 4, panel c)). Also in this case the time-shift between the forces on the trunk and on the wall is not relevant for G_{c1} , although they are not perfectly in phase, due to the larger vertical component of the overtopping jet (see frame 4, panel a)) compared to that shown for PDB. This time lag is more pronounced for increasing wall retreat G_{c2} and G_{c3} . Moreover, for HF-event type the quasi-static load for G_{c2} and G_{c3} has a longer duration than that observed for PDB. Similarly to what observed for PDB, the air content is in the range 10-25%, regardless the wall position.

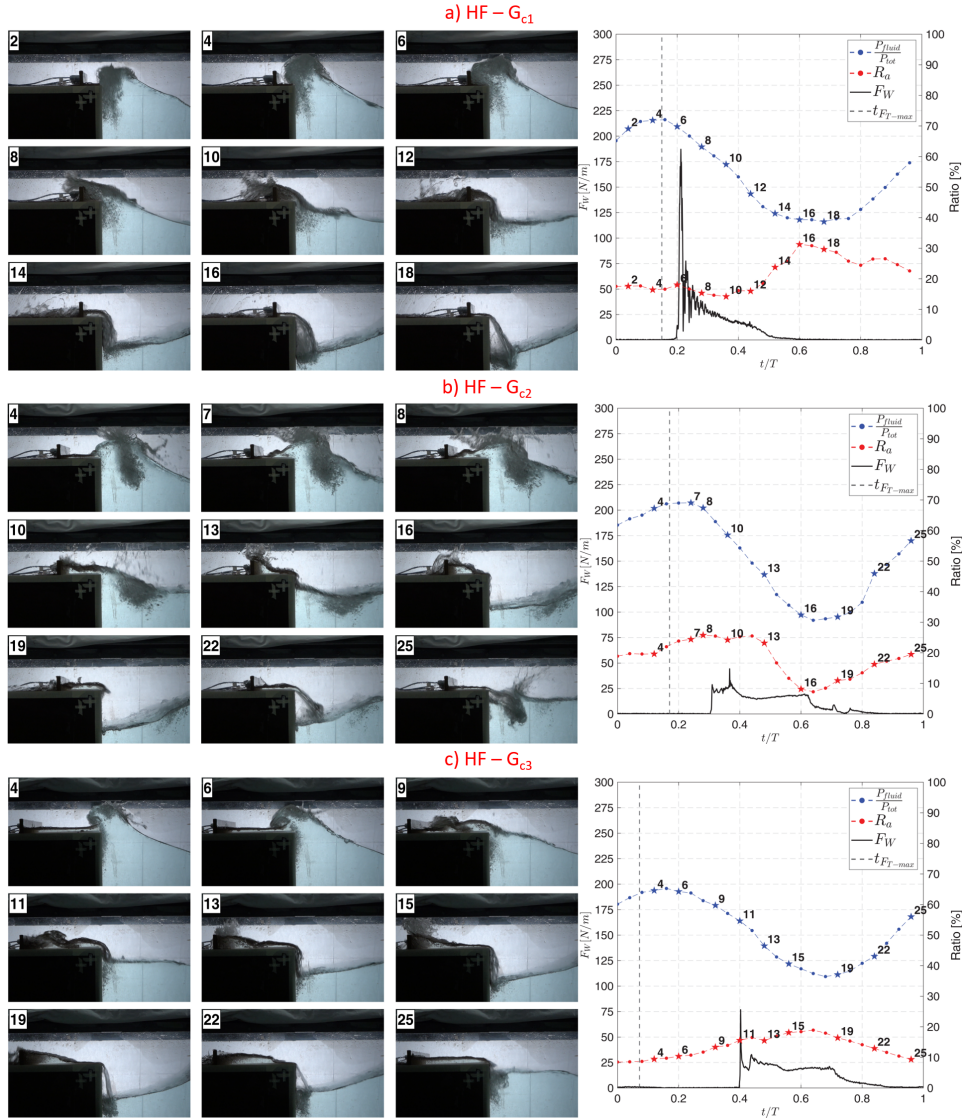


Figure 4.9: Representative HF-type event ($H/L_0 = 0.096$, $A_c/H = 0.335$) across three wall retreat positions: a) G_{c1} , b) G_{c2} , c) G_{c3} . Each panel shows 9 time stacks (left) and synchronized signals (right): wall force (black line), $\frac{P_{fluid}}{P_{tot}}$ (blue line), and R_a (red line). Time is normalized by T ; stars mark selected frames; dashed gray line indicates the time of the peak trunk force.

4.3.2.4 Synthetic parameters

All the new 45 tests, carried out varying the wave wall position, have been analyzed with the same methodology of that shown in Figures 4.7, 4.8, and 4.9. Therefore, for each test, some synthetic indicators, of both air content and forces, are extracted, as shown in the Figure 4.10, and considered for the following analysis, namely: I) $\bar{R}_a$ is the time-average of R_a over one wave period (red dashed line in Figure 4.10); II) ΔR_a is the difference between maximum and minimum values of R_a over one wave period; III) R_a^P is the value of R_a at the time instant at which the maximum peak force occurs; IV) $\frac{F_W^{1P}}{\rho g H^2}$ is dimensionless peak force; V) $\frac{F_W^{1P}}{F_W^{2P}}$ is the ratio between the first and second force peaks; VI) $\frac{t_r}{T}$ is the dimensionless rise time of the first peak.

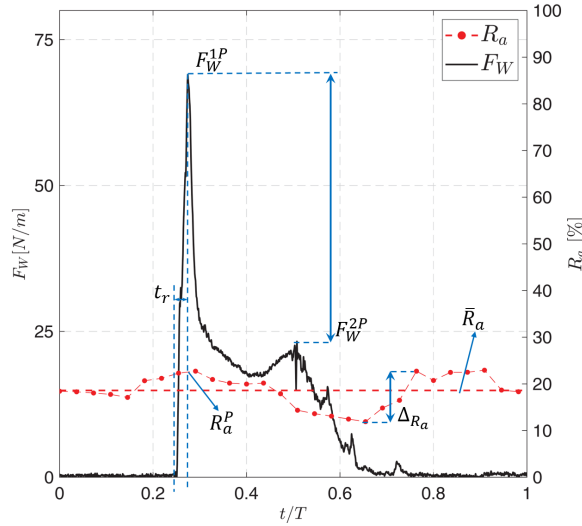


Figure 4.10: Definition of synthetic parameters from force on the wall (black line) and air content (red dashed line) signals over a wave period.

The values of the synthetic parameters are represented in Figure 4.11 in the form of scatter plots, organized in a 3×3 matrix form. The aim of this figure is to evaluate if a relationship exists between these six indicators. To this end, the force and the air content variables are reported on the x- and y-axis, respectively. In the figure, the markers' type refers to the post-overtopping flow event (circles for DB, diamonds for PDB, and triangles for HF), while the markers' color refers to the wall retreat position (red for G_{c1} , green for G_{c2} , and blue for G_{c3}). The dashed vertical red lines in the central column represent the value of $\frac{F_W^{1P}}{F_W^{2P}} = 1$. Figure 4.11 shows that the largest

values of the force variables ($\frac{F_W^{1P}}{\rho g H^2}$ and $\frac{F_W^{1P}}{F_W^{2P}}$) are related to the PDB- and HF-type events for the G_{c1} configuration. HF-type events show the highest values of $\frac{F_W^{1P}}{F_W^{2P}}$, while PDB-type events exhibit the shortest $\frac{t_r}{T}$. Moreover, the figure shows that, in general, the air content tends to increase for increasing forces, while it shows a decreasing trend for increasing rise time. It should be noted that no clearly recognizable patterns can be identified among these synthetic parameters. Nevertheless, a fairly meaningful correlation seems to exist between $\overline{R_a}$ and $\frac{F_W^{1P}}{\rho g H^2}$ (see upper left panel). The ranges of the values of the indicators shown in Figure 4.11 are reported in Table 4.3. Note that for DB-type events, no range is reported in the table as a single test has been carried out for each wall retreat configuration.

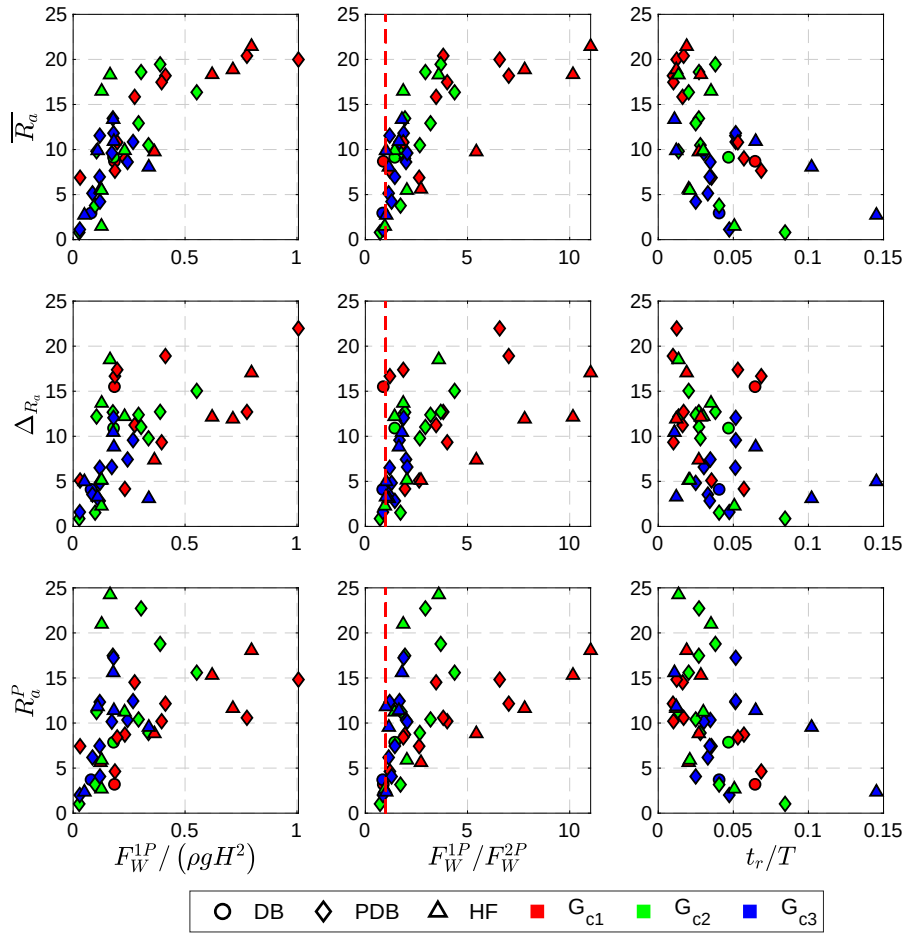


Figure 4.11: Scatter plots of synthetic force and air content indicators from all 45 impact events. Marker shape indicates event type (circles: DB, diamonds: PDB, triangles: HF); marker color denotes wall retreat position (red: G_{c1} , green: G_{c2} , blue: G_{c3}).

	G_{c1}	G_{c2}	G_{c3}
DB	$F_W^{1P} / (\rho g H^2) \approx 0.18$	$F_W^{1P} / (\rho g H^2) \approx 0.18$	$F_W^{1P} / (\rho g H^2) \approx 0.08$
	$F_W^{1P} / F_W^{2P} \approx 0.68$	$F_W^{1P} / F_W^{2P} \approx 1.44$	$F_W^{1P} / F_W^{2P} \approx 0.85$
	$t_r / T \approx 0.06$	$t_r / T \approx 0.05$	$t_r / T \approx 0.04$
	$\overline{R_a} \approx 8.70$	$\overline{R_a} \approx 9.14$	$\overline{R_a} \approx 2.96$
	$\Delta_{R_a} \approx 15.51$	$\Delta_{R_a} \approx 10.91$	$\Delta_{R_a} \approx 4.11$
	$R_a^P \approx 3.20$	$R_a^P \approx 7.87$	$R_a^P \approx 3.71$
	$0.03 < F_W^{1P} / (\rho g H^2) < 1.01$	$0.03 < F_W^{1P} / (\rho g H^2) < 0.55$	$0.03 < F_W^{1P} / (\rho g H^2) < 0.27$
PDB	$1.20 < F_W^{1P} / F_W^{2P} < 7.02$	$0.72 < F_W^{1P} / F_W^{2P} < 4.37$	$< 0.86 < F_W^{1P} / F_W^{2P} < 2.05$
	$0.01 < t_r / T < 0.07$	$0.01 < t_r / T < 0.08$	$0.02 < t_r / T < 0.05$
	$6.88 < \overline{R_a} < 20.40$	$0.80 < \overline{R_a} < 19.45$	$1.13 < \overline{R_a} < 11.81$
	$4.16 < \Delta_{R_a} < 21.97$	$0.86 < \Delta_{R_a} < 15.04$	$1.59 < \Delta_{R_a} < 12.05$
	$4.63 < R_a^P < 14.82$	$1.03 < R_a^P < 22.72$	$2.01 < R_a^P < 17.24$
	$0.12 < F_W^{1P} / (\rho g H^2) < 0.80$	$0.13 < F_W^{1P} / (\rho g H^2) < 0.23$	$0.05 < F_W^{1P} / (\rho g H^2) < 0.34$
	$2.72 < F_W^{1P} / F_W^{2P} < 11.03$	$0.95 < F_W^{1P} / F_W^{2P} < 3.59$	$1.01 < F_W^{1P} / F_W^{2P} < 1.79$
HF	$0.01 < t_r / T < 0.03$	$0.01 < t_r / T < 0.05$	$0.01 < t_r / T < 0.15$
	$5.61 < \overline{R_a} < 21.44$	$1.48 < \overline{R_a} < 18.28$	$2.72 < \overline{R_a} < 13.34$
	$5.11 < \Delta_{R_a} < 17.04$	$2.26 < \Delta_{R_a} < 18.50$	$3.09 < \Delta_{R_a} < 10.43$
	$5.62 < R_a^P < 18.05$	$2.68 < R_a^P < 24.24$	$2.34 < R_a^P < 15.59$

Table 4.3: Synthetic parameters, reported in Figure 4.11, for different wall retreat distances and post-overtopping event types (DB, PDB, HF).

4.3.3 Parameters map

In this section, a parameters map is presented to summarize the results previously described and to contribute developing practical design tools for vertical breakwaters with retreated wall in deep-water conditions. Both the purpose and the structure of this map are inspired to the PROVERBS parameters map proposed by Oumeraci et al. (2001). Nevertheless, the present map refers to loads acting on the wave wall when placed on a retreated position with respect to the seaward face of the caisson.

The purpose of the map is to guide the design of these structures, providing a tool to estimate the forces acting on the wall, given the waves and structure geometry parameters. Figure 4.12 shows the structure of the map, which is organized in three rows. The first row depicts the sketch of the cross section with governing waves and geometrical parameters. The second row presents the proposed fitting formula for estimating the peak value of wave force on the wall, while the third row shows the shape of the dimensionless force time series.

The process for using the map can be summarized in the following workflow: the first step (see first row) consists in identifying the post-overtopping event type (DB, PDB, or HF) depending on both H/L_0 and A_c/H , as proposed in Figure 4.6. In this regard, if $H/L_0 < 0.045$ the event is classified as DB, if $H/L_0 > 0.045$ and $A_c/H < -17.33(H/L_0) + 1.88$ the event is classified as PDB, finally if $H/L_0 > 0.045$ and $A_c/H > -17.33(H/L_0) + 1.88$ the event is classified as HF.

Depending on the dominant event type, the map divides into three distinct branches. For each branch it is possible to calculate, depending on G_c , the dimensionless peak value of wave forces on the wall ($\frac{F_W}{\rho g H^2}$) using a proposed fitting formula as a function of the dimensionless parameter $\frac{R_c G_c}{H h_w}$ (see second row). The formula reads as follows:

$$\frac{F_W}{\rho g H^2} = \exp\left(\alpha \frac{R_c G_c}{H h_w}\right) + \beta, \quad (4.2)$$

where $\alpha = -0.06$ and $\beta = 0.7634$. It should be noted that Equation 4.2, represented by a red dashed line in the figure, has been obtained by fitting the new experiments combined with all the data collected during the previous experimental campaigns on vertical breakwaters with retreated wall (Romano and Bellotti (2023) and small-scale experiments from Chapter 3).

The entire dataset is represented with gray dots, and the black markers highlight the data related to each post-overtopping event type for the specific branch. The blue dashed lines represent the error bounds of 95%. Notice that the presented fitting exhibits a significant data scatter ($R^2 = 0.085$). Nevertheless, it should be pointed out that the variability of impulsive impacts is well recognized in the literature. In fact, similar observations have been widely reported in previous studies on wave impact forces (Stagonas et al., 2016; Marzeddu et al., 2017; Raby et al., 2022), where the inherently chaotic nature of wave impacts, combined with the role of air entrainment in wave-structure interaction, lead to significant variability in measured loads. As also noted by Peregrine (2003); Bullock et al. (2007) and Hofland et al. (2010), the lack of repeatability is not merely an experimental limitation but a physical characteristic of impulsive wave impacts, strongly influenced by transient flow features, residual motions, and even the presence of micro-scale instabilities near the crest tip (Lubin et al., 2019; van Meerkerk et al., 2021).

Moreover, since often structural dynamic analysis is required to properly design such structures under impulsive loads, it is important to have an idea of the nature of the force time series acting on the wall. Thus, the proposed map allows to estimate the shape of the dimensionless force time series. Specifically, within each branch (i.e., event type) the map distinguishes three levels of wave wall retreat: small ($0.5 < G_c/H < 1$), medium ($1 \leq G_c/H \leq 2.2$), and large ($2.2 < G_c/H < 4.5$). For each level of wall retreat, within each event type, a dimensionless force time series acting on the wave wall is provided in the figures of the third row. The data represented in the figures are extracted from the present experimental campaign and are represented as a function of t/T with gray lines, while their average value in time is shown with red lines.

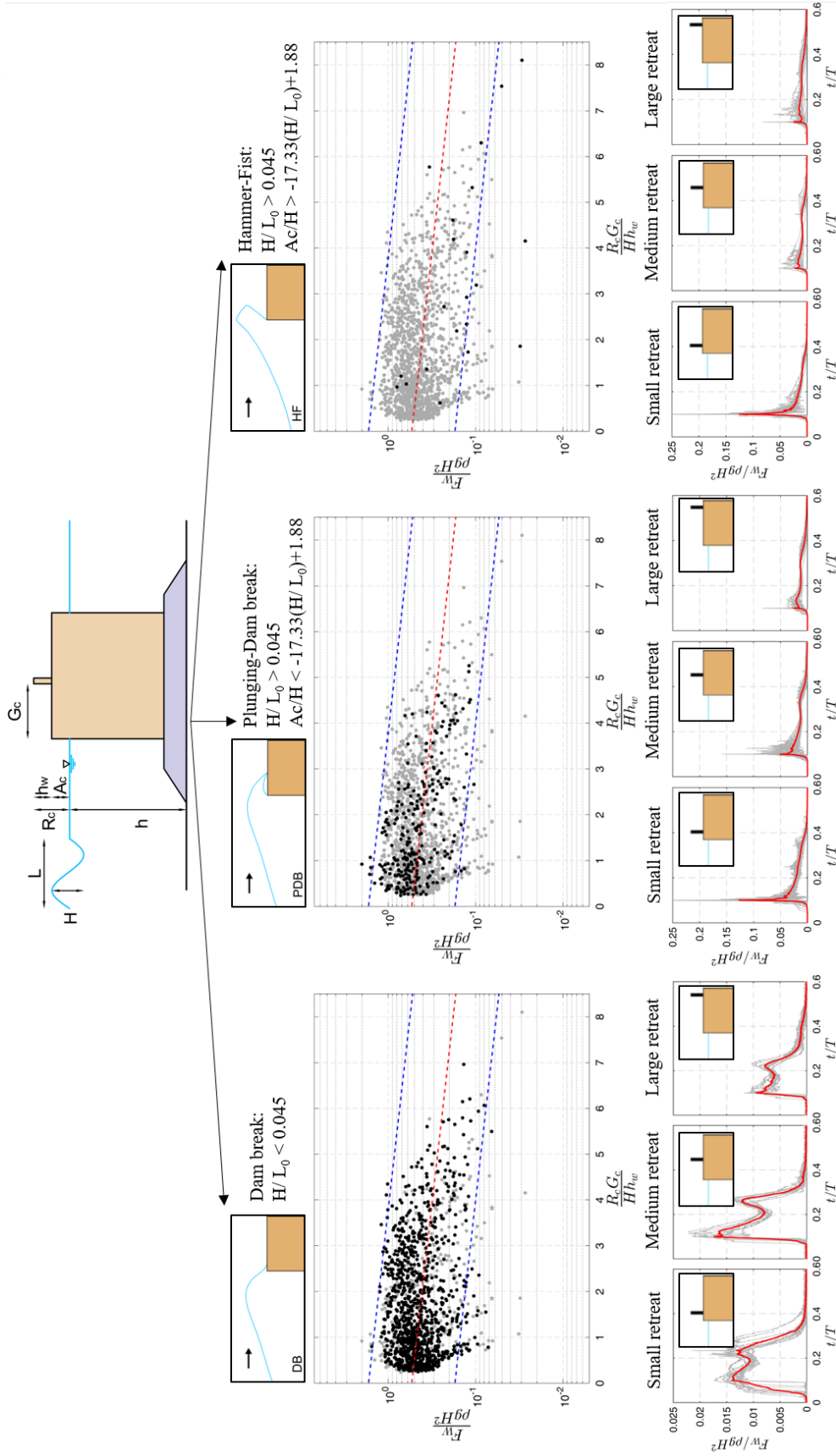


Figure 4.12: Synthetic parametric map for classifying green water events and predicting impulsive loads on composite breakwaters with retreated wave walls.

4.3.4 Downfall pressures

Finally, this section presents the analysis of the downfall pressures. Despite their potential significance in structural loading, particularly important under impulsive overtopping conditions, these pressures have received limited attention in traditional design guidelines and only few studies are available on the topic (Wolters et al., 2005).

Downfall pressures have been measured for all the tests carried out during the present campaign. Figure 4.13 illustrates the asynchronous maximum pressure (P_{df}) diagrams of both frontal and downfall pressures obtained for each event type and for both baseline and retreated wall configurations. The tests represented in the figure have been selected, among the whole dataset, according to the maxima values of the downfall pressures. The three columns represent the different event types: DB, PDB, and HF (first, second, and third column, respectively). Each row refers to the baseline (i.e., no wave wall), G_{c1} , G_{c2} , and G_{c3} configurations (first, second, third and fourth row, respectively). In each panel, the x- and y-axis are expressed in terms of both spatial scale in meters and meters of water column (all the values refer to laboratory scale); the black lines represent the structure geometry, the blue lines indicate the still water level, the gray circles identify the positions of the pressure sensors, the red lines represent recorded values of pressures, and the gray dashed lines represent the pressures calculated using Goda's formulae (Goda, 2010) for the considered wave conditions.

As reported in Figure 4.13, the presence of the wave wall changes the dynamics of the downfall pressures. For DB-type events the downfall pressures exhibit a slight increase both in front and behind the wall for G_{c1} and G_{c2} , while for G_{c3} the loading pattern is very similar to the baseline configuration. For PDB-type events, which conversely to DB are characterized by a large vertical momentum component, the downfall pressure are significant for every configuration. The worst loading is noticed for G_{c1} , where large pressures occur both in front and behind the wall. A similar pattern of downfall pressures, is noticed for HF-type events where the largest value occur for the baseline configuration and G_{c1} .

To summarize, as a general result it appears that, regardless the event type, the downfall pressures increase both in front and behind the wall for small/medium retreats, while for large retreats the downfall pressures pattern is milder. Finally, it should be mentioned that the severest loading

conditions occur for PDB and HF when the wall retreat is small. Table 4.4 summarizes the maxima downfall pressure values obtained for every event type and wall configuration in dimensionless form ($P_{df}^{max}/\rho gH$), following the approach proposed by Wolters et al. (2005), who reported maxima values in the order of $12\rho gH$. It should be noted that the downfall pressures values for retreated wall configurations range between $0.35\rho gH$ (DB) and $4.51\rho gH$ (HF). Therefore, their order of magnitude is coherent with that mentioned by Wolters et al. (2005), although their work refers to impacts of breaking waves in shallow waters on a pure vertical structure with no wave wall on top.

	No-Wall	Small Retreat $0.5 < \frac{G_c}{H} < 1$	Medium Retreat $1 \leq \frac{G_c}{H} \leq 2.2$	Large Retreat $2.2 < \frac{G_c}{H} < 4.5$
DB	0.46	0.94	0.85	0.35
PDB	3.36	3.56	1.96	1.38
HF	4.71	4.51	2.21	1.05

Table 4.4: Summary of maximum vertical downfall pressures ($P_{df}^{max}/\rho gH$) recorded during different overtopping events (DB, PDB, HF) as a function of the incident wave height and wave wall retreat distances.

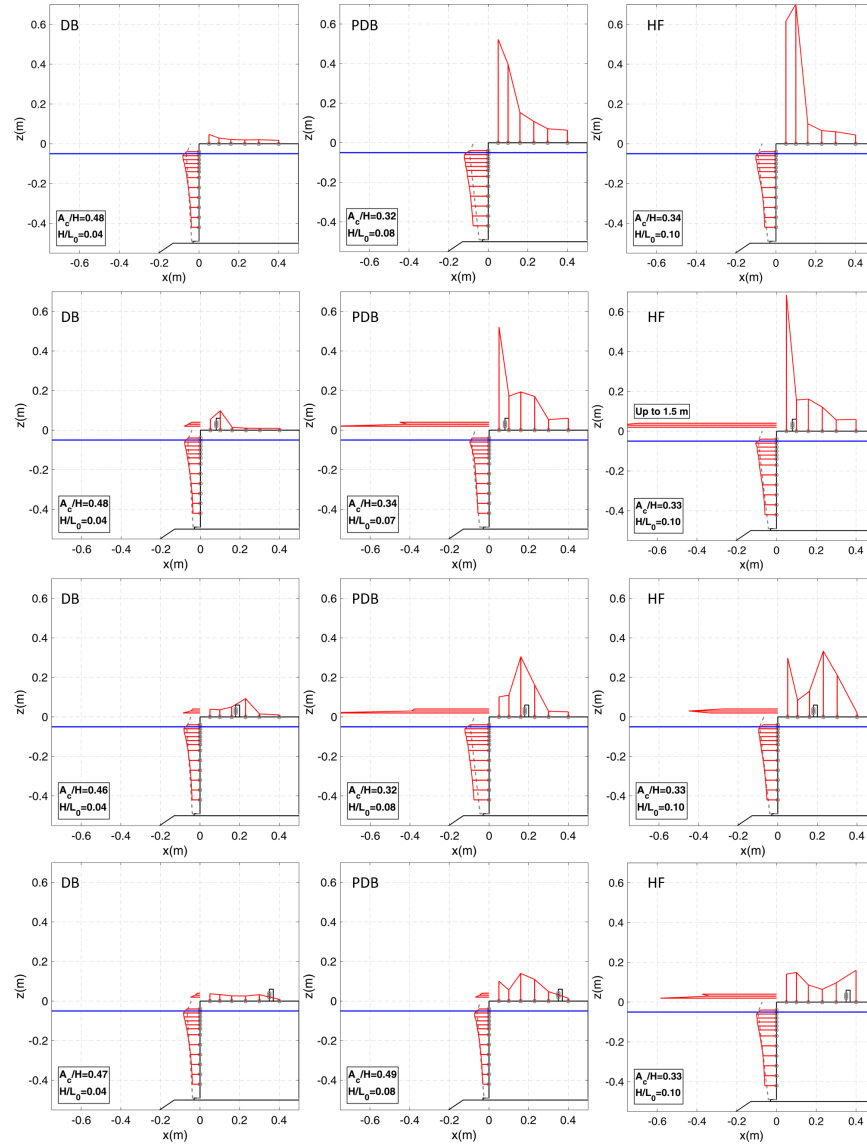


Figure 4.13: Asynchronous maximum pressure diagrams of both frontal and downfall pressures. The three columns represent the different event types: DB, PDB, and HF (first, second, and third column, respectively). Each row refers to the baseline (i.e., no wave wall), G_{c1} , G_{c2} , and G_{c3} configurations (first, second, third and fourth row, respectively). In each panel, the x- and y-axis are expressed in terms of both spatial scale in meters and meters of water column (values in laboratory scale); the black lines represent the structure geometry, the blue lines indicate the still water level, the gray circles identify the positions of the pressure sensors, the red lines represent recorded values of pressures, and the gray dashed lines represent the pressures calculated using Goda's formulae (Goda, 2010).

4.3.5 Limitations

While the present chapter offers new insights into the hydrodynamics of post-overtopping flows on composite vertical breakwaters with retreated wave wall, some limitations should be considered when interpreting the results. The experiments were carried out in a 2D wave flume using fresh water and scaled conditions. These factors may introduce scale effects, particularly in processes involving air entrainment and compressibility during wave impacts that might vary under full-scale and saltwater conditions. These phenomena have been largely studied in the literature and the following effects have been pointed out: e.g., the aeration levels affect the pressure rise times and peak pressures during violent wave impacts (Bullock et al., 2001; Chang et al., 2011). Moreover, the compressibility of air and the subsequent “bounce back” effect during wave impacts demonstrated that compressed air pockets or clusters of bubbles could amplify impact pressures significantly (Wood et al., 2000). Additionally, Hattori et al. (1994) and Cox and Shin (2003) showed that a certain degree of aeration could cushion direct impact forces, but larger, more densely compressed air bubbles could generate pressure spikes.

Comparisons between laboratory and field measurements have revealed significant differences in impact behavior between waves breaking in freshwater and seawater. Bullock et al. (2001) demonstrated that the void fraction in seawater is an order of magnitude greater than in freshwater under similar conditions. The persistence of smaller but more numerous air bubbles in seawater complicates pressure analyses, as also highlighted by Blenkinsopp and Chaplin (2007, 2011). Field observations by Blackmore and Hewson (1984) noted discrepancies between measured and theoretical impact pressures, which were attributed to factors like air entrapment, wave steepness, celerity, and period. Small-scale laboratory experiments, particularly those in freshwater, face challenges related to scale effects. The buoyancy and compressibility of aerated green water flows are not well represented in small-scale laboratory experiments, leading to limitations in accurately reproducing large-scale flow behavior and air entrainment characteristics (Kobus and Koschitzky, 1991; Chanson et al., 2006). Bullock et al. (2001) also highlighted that bubbles in controlled environments differ in size and behavior from those in the open sea, limiting the applicability of Froude scaling for precise replication of full-scale phenomena. Despite these challenges, research by Deane and Stokes (1999, 2002) found similarities in bubble size distributions between laboratory and oceanic breaking waves, suggesting

consistent bubble formation mechanisms. Kobus and Koschitzky (1991) also noted that bubble sizes in aerated flows remained consistent across scales, supporting the validity of some physical models used in engineering predictions.

Another limitation consists in using regular waves. This decision was made following the previous approach of Romano and Bellotti (2023). In fact, the wave impacts, which are object of the study, are characterized by a large variability and uncertainty, also related to the aspects previously mentioned. Therefore, in order to focus on the physics of the post-overtopping flows, regular waves are used and the related phenomena are investigated over one wave period (i.e., intra-wave analysis). As a result, the outputs cannot be directly used to estimate the effects of irregular sea states on such structures. Nevertheless, they remain valid for studying the hydraulic performances of these types of breakwaters under identical, even though regular, wave conditions, as done in previous studies on vertical breakwaters (Martinelli et al., 2018; Castellino et al., 2021).

Furthermore, the structure was modeled as perfectly rigid, thus neglecting any potential influence of structural flexibility or local deformations on the distribution and intensity of impact pressures. At full scale, material compliance may mitigate or amplify pressure peaks, especially under highly impulsive loads. Moreover, the experiments were conducted in a 2D setup, which does not capture eventual 3D effects.

Finally, a limitation can be related to the image-clustering technique used to estimate air content. In fact, although this technique is non-invasive, it has some limitations: I) the method only captures 2D view of the flow, so it might wrongly detect some regions due to overlapping bubbles or strong three-dimensional turbulence; II) the precision of clustering results can be sensitive to lighting conditions, particularly in highly aerated and reflective scenarios. However, even if affected by a degree of uncertainty, such estimates of air content can still provide valuable input for the calibration and validation of numerical models. Despite these limitations, the experimental framework developed in this study provides a reliable basis for investigating the influence of retreated wave walls on wave-structure interaction.

4.4 Key insights of this chapter

In this chapter, a new 2D experimental campaign to explore the hydrodynamics of post-overtopping flows on vertical breakwaters with retreated wave walls has been presented. The experiments, carried out using regular waves, have been performed in two stages: firstly, post-overtopping flows have been studied for a structure without the wave wall (i.e., baseline configuration), in order to be classified into three main event types (DB, PDB, and HF), according to a classification proposed in the literature by Greco et al. (2005, 2007); Fontes et al. (2021) for flows on ships' decks. The experiments demonstrated that the same types of events also occur for composite vertical breakwaters. Moreover, criteria have been provided to predict the onset of specific event types based on wave characteristics and structural parameters.

In the second stage of the experimental campaign, some selected representative conditions of each event type have been reproduced, by varying the wave wall retreat position, in order to investigate how different flow event types and wall retreats affect the nature of impact loads on the structure. Therefore, during the experiments, pressures acting both on the wave wall and on the superstructure's deck have been measured and analyzed. Moreover, an optimized camera setup was specifically designed to record the hydrodynamics of post-overtopping flows and to estimate some quantities that are difficult to measure with direct measurements (e.g., air content). Then, an advanced image-clustering analysis technique has been applied to investigate in detail the processes occurring over a wave period.

The analysis of the experimental results pointed out that, depending on the wall's position, the hydrodynamic forces and characteristics vary significantly for a given event type. Specifically, the most critical loading configurations for the wave wall were observed for PDB- and HF-type events with small wall retreat ($0.5 < G_c/H < 1$). These results, confirm and extend previous findings pointed out by Romano and Bellotti (2023), who highlighted the occurrence of those “worst-case scenarios”. In those conditions the impulsive forces acting on the wave wall, might be in phase with those acting on the trunk, thus leading to a general loading combination on the structure that is way more severe than that occurring for a flushed-wall configuration under the same wave conditions.

Moreover, the combined analysis of forces/pressures and the image-clustering technique allowed to obtain, for each test, some synthetic indica-

tors of both air content and forces. Referring to the air content indicators, it should be mentioned that, although affected by a degree of uncertainty, they can provide valuable estimates, useful to guide the application, and the results interpretation, of numerical models applied to simulate the hydrodynamics of such flows, especially if computational fluid dynamics (CFD) models are used.

Finally, the new results of wave forces acting on the wave wall, combined with the previous data, retrospectively analyzed and classified in terms of post-overtopping flow types, have been used to define a parameters map. The aim of this map, which is inspired to the structure of the PROVERBS map proposed by Oumeraci et al. (2001), is to contribute to develop practical design tools for composite vertical breakwaters with retreated wall in deep-water conditions. Specifically, the map, which branches out depending on the post-overtopping flow type and wall position, allows to estimate the loads acting on the retreated wave wall, given the waves and geometrical parameters. Moreover, since often structural dynamic analysis is required to properly design such structures under impulsive loads, the map also provides the dimensionless time series of the force.

Finally, the analysis of the downfall pressures has pointed out that the presence and the position of the wave wall changes dramatically also the nature of these vertical loads. Specifically, the worst loading configuration is noticed for PDB- and HF-type events with small wall retreats ($0.5 < G_c/H < 1$), where large pressure values (up to $4.51\rho gH$) occur on the superstructure deck.

Although the analyses presented so far have been carried out under regular wave conditions, real sea states are inherently irregular and characterized by a stochastic nature. In order to extend the present findings, the next chapter investigates wave-structure interaction under irregular waves, explicitly addressing the role of stochastic variability through random phase seeding.

Chapter 5

Influence of random seeding on overtopping and loads for a flushed-wall vertical breakwater

Parts of the work presented in this chapter will be included in the journal paper currently in preparation for submission: Centorami, M., Cecioni, C., Romano, A., Bellotti, G. (2026). Assessment of Variability in Physical Modeling of Wave Overtopping on Vertical Breakwaters.

5.1 Introduction

This chapter follows naturally from the trajectory established in the previous ones. In Chapter 3, a comprehensive small-scale experimental campaign quantified, on caisson breakwaters with retreated wave walls, the geometric effects on hydraulic behaviour (reflection, overtopping, and global/local forces) as functions of the wall retreat distance G_c and wave parameters, with appropriate non-dimensionalization of the key quantities. Chapter 4 then examined the dynamics of post-overtopping flows, delineating the main typologies and documenting the associated pressures fields and the mechanisms underlying impulsive loads on the seaward face and the deck.

Building on the previous chapters, Chapter 5 shifts the focus from geometric optimization to the intrinsic variability of the processes under irregular waves. Specifically, the influence of the choice of the seed number

(i.e., the random phases assigned to the harmonic components for a given target spectrum) on measured overtopping quantities and structural loads for a deliberately simple reference configuration (a vertical breakwater with flushed wall ($G_c = 0$) has been investigated. Consistently with the notation used in the previous chapters, the flushed configuration is denoted as G_{c0} , and quantities in this chapter are reported with respect to this layout.

For the theoretical and practical background already established in the thesis, the reader is referred to the relevant chapters and cited literature concerning wave actions on vertical walls (Goda, 2010; Takahashi, 1996; Oumeraci et al., 2001; Cooker and Peregrine, 1990; Peregrine, 2003; Bullock et al., 2007; Bredmose et al., 2009; Cuomo et al., 2010a; Bredmose et al., 2015), and for overtopping, the EurOtop manual (EurOtop, 2018) is the main reference for design guidance. It consolidates decades of experimental research and provides empirical formulas for estimating mean overtopping discharge as a function of structural and wave parameters. In addition to average discharge, EurOtop incorporates safety thresholds for pedestrian risk and structural performance. More recent studies (after the EurOtop (2018)) have advanced the understanding of overtopping on vertical and steep walls, both in shallow and deep water conditions, by considering wave-by-wave dynamics and spectral features such as steepness, peakness and relative free-board. These studies have improved the accuracy and range of applicability of overtopping formulas for both impulsive and non-impulsive conditions (Salaudhin and Pearson, 2019; Shaeri and Etemad-Shahidi, 2021; Salaudhin et al., 2022; van Gent et al., 2024; Cecioni et al., 2025).

A growing body of literature has highlighted the significant variability inherent in overtopping and wave loading processes, even under nominally identical wave conditions. This variability arises fundamentally from the way irregular waves are generated in laboratory environments. The free surface elevation is reconstructed as the linear superposition of harmonic components derived from a target spectrum, with each component assigned a random initial phase. A single number “seed” determines the full set of phases and thus uniquely defines one physical realization of the wave field (Tuah and Hudspeth, 1982). Although all realizations share the same spectral characteristics, differences in phase composition lead to distinct wave trains that can produce significantly different hydrodynamic responses at the structure. The implications of this phase variability have been well documented in numerical and experimental studies. Pearson et al. (2001) was among the first to demonstrate experimentally that even small differences

in wave phase can generate substantial scatter in overtopping rates and individual volumes. Numerical studies by McCabe et al. (2013) and Williams et al. (2014) confirmed that this variability can be especially pronounced under low overtopping probability scenarios, resulting in wide dispersion of discharges and wave-by-wave volumes. Building on these foundations, Romano et al. (2015) conducted a systematic physical model study to quantify the variability induced by seeding number in rubble mound breakwaters, demonstrating that shorter wave series (e.g., 500 waves, 300 waves, etc.) can yield average overtopping values comparable to the standard 1000 waves tests, although with greater uncertainty. This variability was further explored by Williams et al. (2019), who performed an extensive experimental campaign on both sloped and vertical structures. Their results confirmed that, for low overtopping probabilities ($P_{ov} < 5\%$), the mean discharge could vary by up to 75% between different seeds.

Beyond average discharge, individual overtopping volumes are essential in design assessments, as a limited number of high-volume events may dominate impacts (Koosheh et al., 2021). Early work established that exceedance probabilities of individual volumes can be represented by a two-parameter Weibull distribution, initially adopting an almost constant shape factor for emergent slopes and vertical structures Su et al. (1992); Franco et al. (1994); van der Meer (1995); Besley (1999). Later studies retained the Weibull framework but refined the parameterization and discussing the role of wave steepness Victor et al. (2012); Gallach Sanchez (2018). Extensions were also proposed for specific conditions such as shallow-water waves and negative freeboard/combined overtopping-overflow Hughes and Nadal (2009); Nørgaard et al. (2013); Pan et al. (2015). Importantly, when overtopping is rare and the tail controls design, several authors improved robustness by focusing the fit on the upper tail (threshold selection) or by weighted fitting that emphasizes the largest events Bruce et al. (2009); Molines et al. (2019); Mares-Nasarre et al. (2020), with additional geometrical effects such as berms also investigated Ju et al. (2019).

In parallel with overtopping, a degree of variability has been observed in wave impact pressures. Even under controlled laboratory conditions with nominally identical wave inputs, impact magnitudes can differ significantly from one wave to another. This variability has been linked to multiple sources, including turbulence, residual long-wave motion in the wave basin/flume, local crest instabilities, and the entrainment or entrapment of air at the moment of impact (Bagnold, 1939; Hattori et al., 1994;

Peregrine, 2003; Bullock et al., 2007; Cuomo et al., 2010a; Lubin et al., 2019; van Meerkerk et al., 2021). Its effects have been clearly documented in repeatable laboratory tests, where even under nearly identical wave conditions, large differences in impact pressures have been observed (Chan and Melville, 1988; Bogaert et al., 2010). For example, Marzeddu et al. (2017) observed that even under nearly identical regular wave conditions, the maximum recorded impact pressure could vary up to 500% of the minimum value in repeated tests, highlighting the significant intrinsic variability of wave loading on vertical structures. Raby et al. (2022) provided a comprehensive review of the mechanisms driving the variability of the wave impact pressure, including the influence of some wall geometry and local flow kinematics. Through high-resolution pressure and velocimetry measurements, they confirmed that wave impacts are highly sensitive to small-scale variations and follow identifiable statistical distributions, with rare events significantly exceeding mean values.

In this chapter, the sensitivity to random phase seeding of the main hydraulic responses of a flushed-wall vertical breakwater ($G_c = 0$) has been systematically investigated. The analysis focused on wave overtopping processes, including mean overtopping discharge, maximum overtopping volumes, as well as on wave-induced pressures and global forces acting on the structure and on the superstructure. By comparing multiple independent realizations of identical spectral conditions, the study provides a quantitative assessment of the intrinsic variability associated with irregular wave trains.

The chapter is structured as follows. Following this introduction, the experimental setup is described, including the model configuration and the wave conditions. The methodological framework adopted for the analysis is then presented, together with a discussion of the experimental results. Finally, the chapter is concluded by summarizing the key insights.

5.2 Experimental setup

5.2.1 2D physical model

The experimental campaign described in this chapter were carried out in the medium-scale wave flume of Roma Tre University (Section 2.3.1). The model structure represents a vertical breakwater and, as sketched in Figure 5.1, it is composed by a caisson trunk, a superstructure with a vertical wall and a rubble mound foundation. Both the caisson and the superstructure are made by marine plywood panels (0.015 m thick) assembled to ensure a total model width of 0.60 m, matching the flume's cross section. The geometrical parameters of interest for the structure, as shown in Figure 5.1 (panels b and c), are the water depth inside the flume, $h = 0.635$ m; the water depth at the toe of the caisson, $d = 0.485$ m; and the crown wall freeboard, $R_c = 0.075$ m. The rubble mound foundation has a height of $h_b = 0.15$ m and consists of an inner core of rock material, covered by an external layer of rocks weighing between 20.0 and 30.0 g. The vertical composite breakwater dimensions and the water level inside the flume remain constant throughout all the experimental tests.

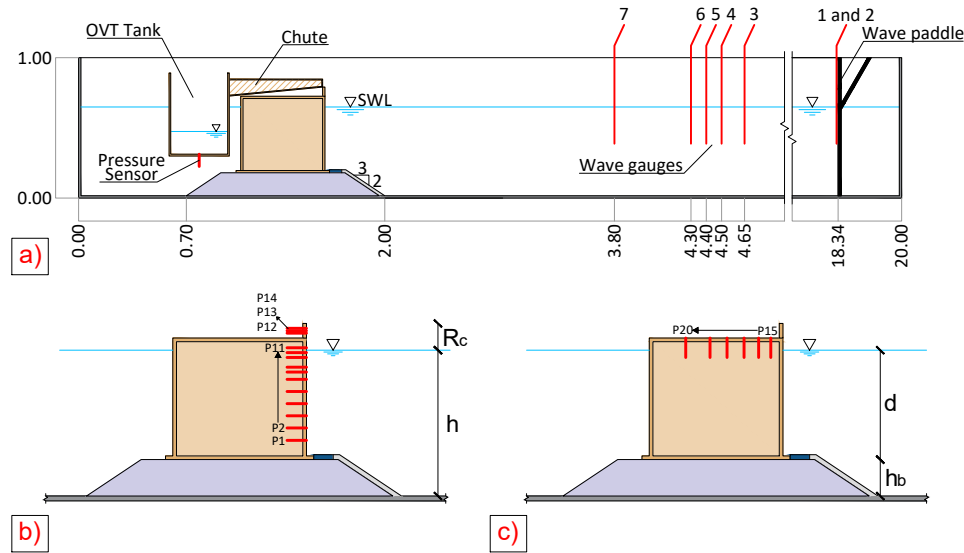


Figure 5.1: Panel a): longitudinal section of the flume with the wave gauges arrangement, the model structure, the wave overtopping tank and the wave chute (measures are in m). Panels b) and c): zooms on the model structure, including indication of the geometrical parameters and positions of the front and top pressure sensors, panel b) and c) respectively.

The measurements of overtopping discharge and wave-by-wave volumes are taken by recording the water level inside an overtopping tank positioned behind the model structure (see panel a) of Figure 5.1). The water overtopping the wall is collected inside the tank through a 0.08 m wide chute, located at the center of the structure to minimize side effects. A pressure transducer (TRAFAG Submersible Pressure Transmitter NAL 8838, range 0-200 mbar) is installed at the bottom of the overtopping tank to accurately record the pressure time series corresponding to the accumulation of the overtopped water volumes.

Front pressure sensors		
Pressure sensor	Vertical position (m)	Influence area (m^2/m)
$P1$	0.080	0.090
$P2$	0.130	0.050
$P3$	0.180	0.050
$P4$	0.230	0.050
$P5$	0.280	0.050
$P6$	0.330	0.040
$P7$	0.360	0.025
$P8$	0.380	0.030
$P9$	0.420	0.030
$P10$	0.440	0.020
$P11$	0.460	0.050
$P12$	0.520	0.025
$P13$	0.530	0.010
$P14$	0.540	0.025

Top pressure sensors		
Pressure Sensor	Horizontal position (m)	Influence area (m^2/m)
$P15$	0.050	0.075
$P16$	0.100	0.055
$P17$	0.160	0.065
$P18$	0.230	0.070
$P19$	0.300	0.075
$P20$	0.400	0.150

Table 5.1: Pressure sensor names, positions and areas of influence. For the sensors placed on the sea-ward face of the caisson trunk ($P1$ - $P14$), the position is the vertical distance from the bottom corner of the trunk. For the sensors placed on the superstructure ($P15$ - $P20$) the position is the horizontal distance from the trunk sea-face.

Other pressure transducers are installed on the structure. As shown in Figure 5.1, panels b) and c), 14 sensors ($P1$ - $P14$) were placed on the seaward

face of the vertical caisson, and 6 sensors (P15-P20) on the top of the superstructure to capture the pressures induced by post-overtopping water falling (downfall pressures). Table 6.1 lists the identifiers, positions, and influence areas of the pressure sensors, distinguishing between front sensors (P1-P14) and top sensors (P15-P20). All pressure transducers operate at a sampling frequency of 7 kHz, allowing accurate detection of impulsive pressure peaks.

5.2.2 Test program

The experimental campaign consists of a total of 125 tests, designed to simulate 5 different incident wave energy spectra. For each spectrum, the seeding of the random number generator used for the starting phases distribution is changed 25 times. Therefore, firstly five JONSWAP wave energy density spectra are defined, each characterized by a specific pair of significant wave height (H_{m0}) and peak period (T_p) values. The peak enhancement parameter was kept constant and equal to $\gamma = 3.3$. Then, 25 free surface elevation time series are generated for each wave spectrum, by using the random phase method to change the phase distribution. Thus, 125 wave sequences were generated in the wave flume, the overtopping volumes (if any) were collected into the overtopping tank and the pressures on the structure were recorded. Each test lasted for 1000 incident waves, thus ensuring a robust statistical basis for the subsequent analysis. This configuration ensured that within each group of 25 realizations the spectral properties remained fixed, and that the observed variability could be directly attributed to the random seeding procedure.

Wave Spectrum	H_{m0} (m)	T_p (s)	t_{test} (s)	R^* (-)	h/L_0 (-)	h/H_{m0} (-)
WS1	0.102	1.62	1283	0.73	0.16	6.22
WS2	0.075	1.39	1133	1.00	0.21	8.47
WS3	0.063	1.27	1021	1.19	0.25	10.08
WS4	0.055	1.19	984	1.36	0.29	11.55
WS5	0.048	1.11	933	1.56	0.33	13.23

Table 5.2: Summary of wave spectrum conditions and test parameters.

Table 5.2 reports the test parameters for all the five incident wave spectrum reproduced. It has to be noted that the geometrical dimensions of the structure and the water level inside the flume remain fixed over the 125 tests. The values of the significant wave height, H_{m0} , have been fixed in order to test the overtopping variability for a quite wide and mostly used range of the non-dimensional freeboard, i.e. $R^* = R_c/H_{m0}$. The R^* values

ranges approximately from 0.7 to 1.6. The peak wave period, T_p of the five incident spectra has been calculated by imposing a constant wave steepness, $H_{m0}/L_0 = 0.025$, and using $L_0 = gT_p^2/(2\pi)$. The wave sequence duration, t_{test} , allows for all the test the reproduction of 1000 waves. In Table 5.2 are reported also the values of the ratio h/L_0 , indicating that the structure is in intermediate waters, since $0.05 < h/L_0 < 0.5$. However, the structure can be considered in relatively deep water regarding wave overtopping, since the water depths are at least four times the wave height, $h/H_{m0} > 4$ (EurOtop, 2018). Furthermore, the influence of the rubble mound foundation can be considered negligible, since the ratio between the water depth at the caisson toe and the total water depth is $d/h = 0.76$, greater than 0.6 (EurOtop, 2018).

5.3 Methodology for the analysis

The experimental data obtained from the 125 tests were processed through a series of standard analysis aimed at quantifying the variability induced by random seeding in the generated wave fields. The methodology includes (i) estimation of incident wave parameters, (ii) processing of overtopping measurements, and (iii) computation of forces on the structure via pressures.

For each test, the incident and reflected wave components were separated through the non-linear reflection analysis method of Andersen et al. (2017), applied to the free surface elevation records from the five wave gauges positioned in front of the structure. This procedure provided the incident significant wave height H_{m0} , which was subsequently used to determine the dimensionless parameters governing the analysis, such as the relative freeboard $R^* = R_c/H_{m0}$.

The overtopping discharges are obtained from the pressure signal recorded by the transducer located inside the overtopping tank. For each test the pressure signal is converted into water level variation inside the overtopping tank, and then into volumes of overtopping water per unitary width of the structure. From the cumulative volumes time series of each experimental test, it has been possible to calculate three main overtopping parameters: 1) dimensionless mean overtopping discharge (q^*), 2) percentage of overtopping (P_{ow}), and 3) maximum overtopping volume (V_{Max}).

The integration of the overtopping volume time series over the total duration of each experiment, allows to derive the total overtopped volume V_{tot} in m^3/m . Subsequently, the mean overtopping discharge is calculated as: $q = \frac{V_{tot}}{t_{test}}$. Where t_{test} is the test duration for reproducing 1000 waves. The dimensionless discharge, namely q^* , is computed as $q^* = \frac{q}{\sqrt{gH_{m0}^3}}$.

The overtopping cumulative volumes signal has been filtered with a moving-average procedure to enhance the detection of distinct overtopping events. Each event, corresponding to a water mass entering the tank, was identified manually, for a more precise procedure, to calculate individual volumes (V_i). Therefore, the proportion of overtopping waves has been calculated as: $P_{ovt} = \frac{N_{ovt}}{N_w} \cdot 100$. Where N_{ovt} is the number of overtopping events during each test.

The detection of all the individual volumes during each test, allowed also to obtain the maximum overtopping volume, defined as $V_{max} = \max(V_i)$. Figure 5.2 shows an example of time series of cumulative volume, during one of the 125 tests performed. The original signal, acquired at 7.0 kHz, is reported with the gray line, while the blue line indicates the mean-moved averaged signals that allow the identification of each single overtopping volumes.

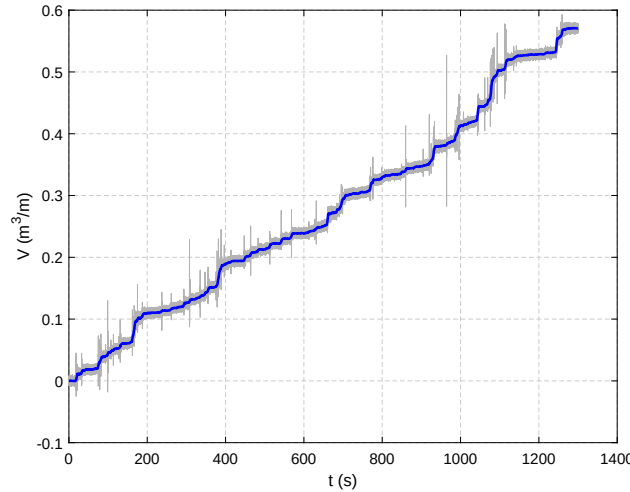


Figure 5.2: Example of the cumulative wave overtopping volume time series recorded during one test with $R^*=0.75$ by the pressure sensor located inside the overtopping tank.

Pressure data from the 21 transducers installed on the front face and

on the superstructure of the structure were used to obtain both local and global wave-induced loads. Each pressure signal was multiplied by the corresponding influence area to obtain the instantaneous local force, and the global horizontal and vertical forces were obtained by integrating the contributions from all sensors acting on each surface. The detection of peaks was based on the zero-crossing method, allowing the identification of maximum values (P_{Max} and F_{Max}) and of the $P_{1/250}$ and $F_{1/250}$ indexes, defined as the mean of the upper 0.4% of peaks in each time series. These indicators provide a robust measure of “severe but not singular” impacts, minimizing the influence of isolated outliers while retaining the relevance of the design.

Finally, all the overtopping and loads quantities are analyzed to explore their variability among the 25 realizations, solely induced by the random phase composition of the incident wave field. The variability of each analyzed quantity is expressed in term of the variation coefficient (in percentage) $\sigma^* = \frac{\sigma}{|\mu|} \cdot 100$, where σ and μ are respectively the standard deviation and the mean value. It is worth to stress that the variation coefficient, so defined, is associated with variables that have a normal distribution, and it is commonly used in EurOtop (2018).

5.4 Results and discussion

This section presents and discusses the experimental results. The analysis is organized into two parts. The analysis of wave overtopping discharges and the analysis of the induced wave loads that act on the structure.

5.4.1 Variability of global wave overtopping in physical modeling

Wave overtopping quantities are measured for all 125 tests by means of a pressure sensor fixed at the bottom of the overtopping tank, as described in section 5.3. Firstly, the measured mean overtopping discharges are compared with those obtained using existing prediction tools. Particularly, the empirical EurOtop (2018) formula and the artificial neural network (van Gent et al., 2007). Moreover, also the mean overtopping discharges measured during other experimental tests and collected in the available CLASH database (CLASH database, 2005), are used for comparison.

In Figure 5.3 these results are reported in term of non-dimensional mean

overtopping discharge versus the non-dimensional crest wall freeboard. In the figure, the black markers refer to the new 125 experimental tests (25 for each tested R^*); the gray markers refer to the CLASH dataset appropriately selected (later explained); the red circles refer to the mean value estimation using the artificial neural network, while the red triangles are relative to the 5% and 95% percentiles. The CLASH database has been filtered to select only tests on structures similar to that of the present campaign. Particularly, tests on: vertical breakwater structure with a vertical crown wall aligned with the caisson front face, positioned over a low rubble mound foundation, $d/h > 0.6$; no influence of the foreshore slope and incident wave orthogonal to the breakwater. These filters are shown in Table 5.3. The black line in Figure 5.3 refers to the 7.1 formula proposed by EurOtop (2018), that states:

$$q^* = 0.047 \exp \left[- \left(2.35 \frac{R_c}{H_{m0}} \right)^{1.3} \right] \quad (5.1)$$

and it is valid for vertical breakwater with no influence of foreshore. The dashed black lines refer to the 90% confidence band of equation (5.1). On the right y-axis of Figure 5.3 is reported the distribution of the selected CLASH tests over classes of R^* 0.1 wide.

Parameter	Filter
q [$m^3/s/m$]	$\geq 10^{-6}$
m [-]	≥ 1000
β [$^\circ$]	$= 0$
$\cot \alpha_d$ [-]	$= 0$
$\cot \alpha_u$ [-]	$= 0$
$\cot \alpha_{inc}$ [-]	$= 0$
G_c [m]	$= 0$
d/h [-]	> 0.6

Table 5.3: List of filters applied to CLASH database to get vertical breakwater test conditions similar to those of the present experimental campaign. The parameters are defined as: q is the mean overtopping discharge; m is a measure of the slope of the foreshore $= 1:m$ (unit vertically:units horizontally); β is the angle of wave attack relative to the normal on the structure; $\cot \alpha_d$ is the cotangent of the angle that the structure part below the berm makes with a horizontal; $\cot \alpha_u$ is the cotangent of the angle that the structure part above the berm makes with a horizontal; $\cot \alpha_{inc}$ is the cotangent of the mean angle that the structure makes with a horizontal, including the berm; G_c is the crest width of the structure; d/h is the ratio between the water depths at the toe of the structure and in front of the toe of the structure.

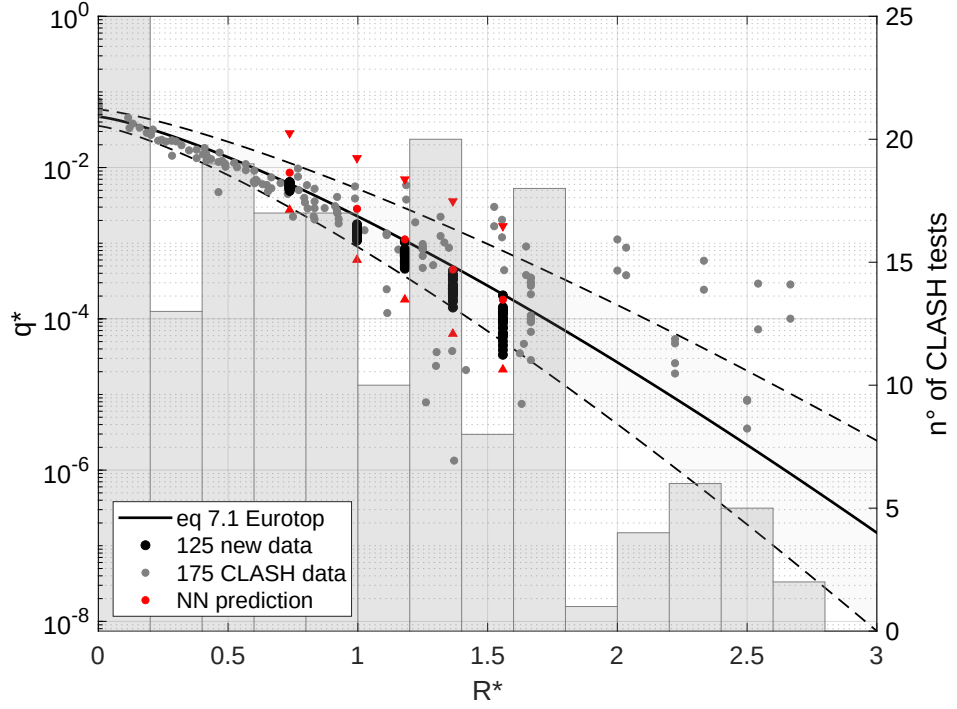


Figure 5.3: Non-dimensional mean overtopping discharge versus the non-dimensional freeboard. The black circles indicate the new 125 physical data; the gray circles indicate selected physical data from CLASH database; the red markers indicate the data by the artificial neural network of van Gent et al. (2007) providing as input the data of the present experimental campaign (red circles represent the mean value, while red triangles represent the 5% and 95% percentiles). On the right y-axis is reported the number of data for classes of R^* .

Overall, Figure 5.3 highlights the robustness of the present experimental dataset, which includes 125 tests and is comparable in size to the selected CLASH subset used for comparison. The measured non-dimensional mean overtopping discharges q^* show a good agreement with both the EurOtop empirical formulation and the neural network predictions. In particular, almost all experimental points fall within the confidence band of the EurOtop equation, while all data lie within the confidence interval provided by the neural network model. The scatter of the experimental results increases with increasing relative freeboard R^* , corresponding to lower overtopping discharges. This behaviour, observed for both the present dataset and the selected CLASH tests, reflects the intrinsic variability of wave overtopping under low-probability conditions (Romano et al., 2015; Williams et al., 2019). At high relative freeboards, overtopping is governed by a limited number of

events, making the computed mean discharge particularly sensitive to random phase initialization **seeding?**. Consequently, small differences in the incident wave time series, and in the number and magnitude of overtopping waves, may result in large relative variations of q^* . It is also noted that the mean values obtained from the present experimental campaign tend to lie slightly below the EurOtop curve (Equation 5.1). This behaviour can be attributed to the fact that the EurOtop reference relationship is derived from a best-fit across multiple and diverse structures, including perforated caisson chambers, horizontal composite breakwaters, and caissons with wave walls of various geometries (Franco et al., 1994; van der Meer and Bruce, 2014), whereas the present data refer to a single, well-defined vertical structure.

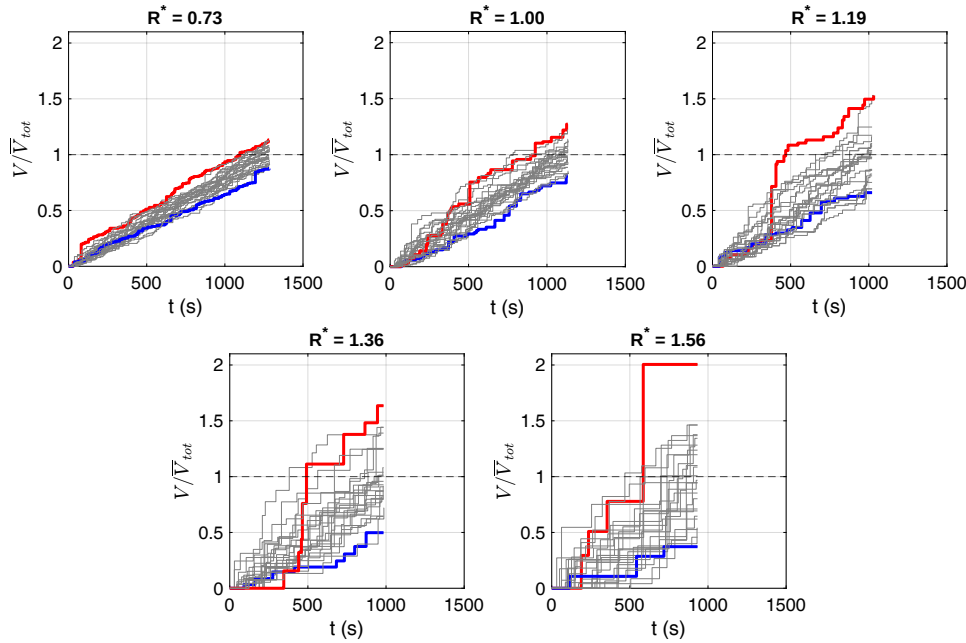


Figure 5.4: Time evolution of cumulative overtopping volumes per unit width over the final mean volume for different relative freeboards R^* . The gray curves correspond to the 25 independent realizations for each R^* , the red curve indicates the maximum mean cumulative overtopping, and the blue curve indicates the minimum mean cumulative overtopping.

To better illustrate the influence of random seeding on the overtopping process, Figure 5.4 shows the time series of the cumulative volume (gray lines) for each test, grouping all 25 repetitions for each R^* value. The red and blue lines indicate, for each R^* , the test among all the 25 realizations that measured respectively the maximum and the minimum mean overtopping discharges. It can be noted that the cumulative volumes are normalized

by the average total volume among all the 25 repetitions, $\overline{V_{tot}}$.

These plots clearly illustrate how, with increasing R^* , overtopping events become progressively less frequent and more intermittent. This behavior is reflected in the increasingly step-like shape of the cumulative volume curves, indicating that the total overtopped volume is governed by a smaller number of individual events. At the same time, the dispersion of the volumes, among the different random seeds, increase markedly with R^* . While for low R^* values the cumulative volume time series are relatively clustered, for higher R^* even a limited number of incident waves can produce substantial differences in the cumulative overtopped volume. As a consequence, the variability of the mean overtopping discharge across the 25 repetitions increases significantly with R^* .

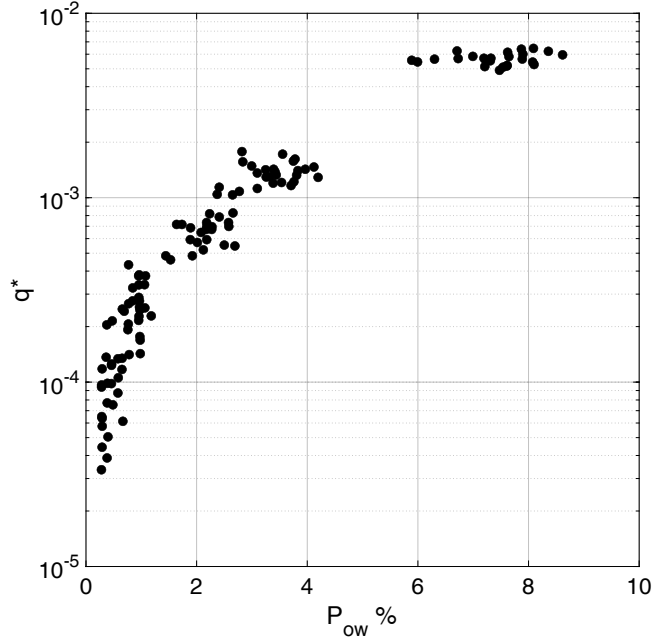


Figure 5.5: Joint variability of dimensionless mean overtopping discharge q^* and proportion of overtopping waves P_{ow} across all realizations (each point = one seed).

In addition to the mean overtopping discharge, a further key descriptor of the hydraulic response is the proportion of overtopping waves in percentage (P_{ow}). The joint variability of q^* and P_{ow} across all realizations is illustrated in Figure 5.5, where each dot corresponds to one seed. The plot highlights a clear monotonic relation between the two indicators: tests with larger P_{ow} generally correspond to higher mean discharges, while a substantial scatter

remains due to seed-to-seed variability. Notably, at low overtopping probabilities the dispersion is large, reflecting the fact that small differences in the incident wave sequence may lead to significant differences in the number of effective overtopping waves, and therefore in the computed discharge.

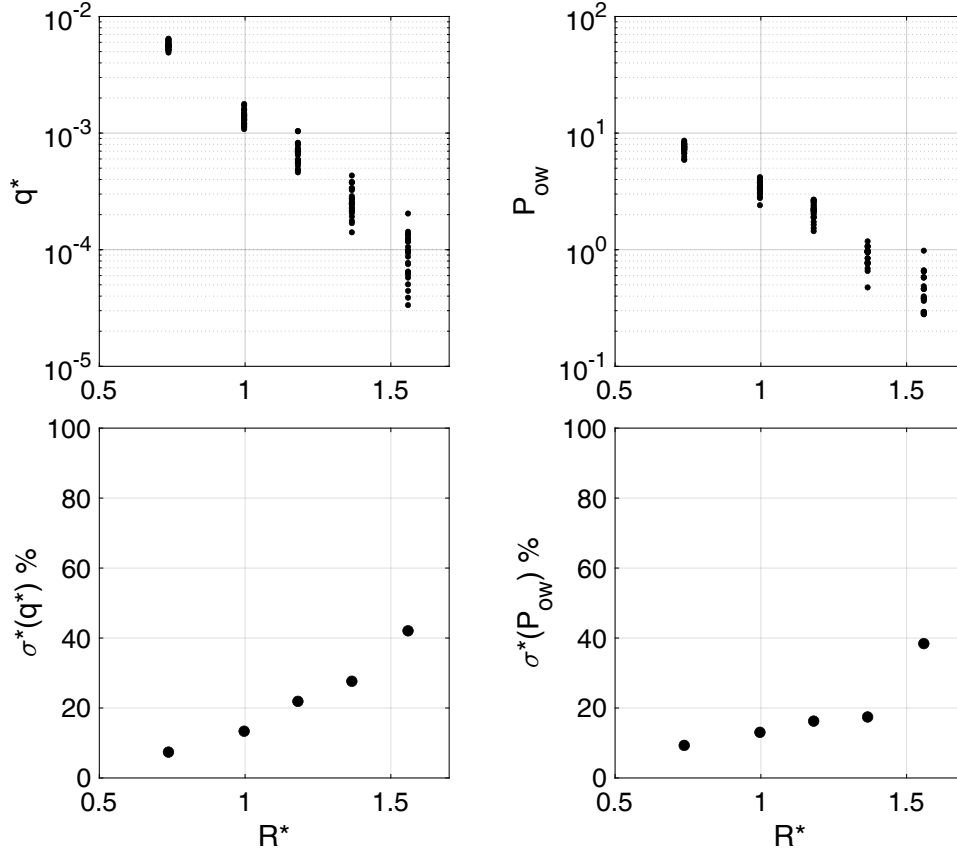


Figure 5.6: Top: scatter of q^* (left) and P_{ow} (right) against relative freeboard R^* . Bottom: corresponding coefficients of variation $\sigma^*(q)$ and $\sigma^*(P_{ow})$, defined as the ratio between standard deviation and mean across 25 realizations.

A more systematic analysis of this variability is reported in Figure 5.6, which plots both q^* and P_{ow} against the relative freeboard R^* , together with their coefficients of variation (σ^*). As expected, both q^* and P_{ow} decrease with increasing R^* , following an approximately exponential trend consistent with previous literature findings (Franco et al., 1994; Besley, 1999; EurOtop, 2018). At low relative freeboard (small R^*), overtopping is frequent and the dispersion between seeds is limited (coefficients of variation below 10–15%). Conversely, at large relative freeboard values (high R^*), overtopping becomes rare: the number of overtopping waves N_{ow} decreases, and both

q^* and P_{ow} exhibit higher relative scatter (coefficients of variation around 40%). This behaviour is coherent with earlier observations on vertical walls (Williams et al., 2019).

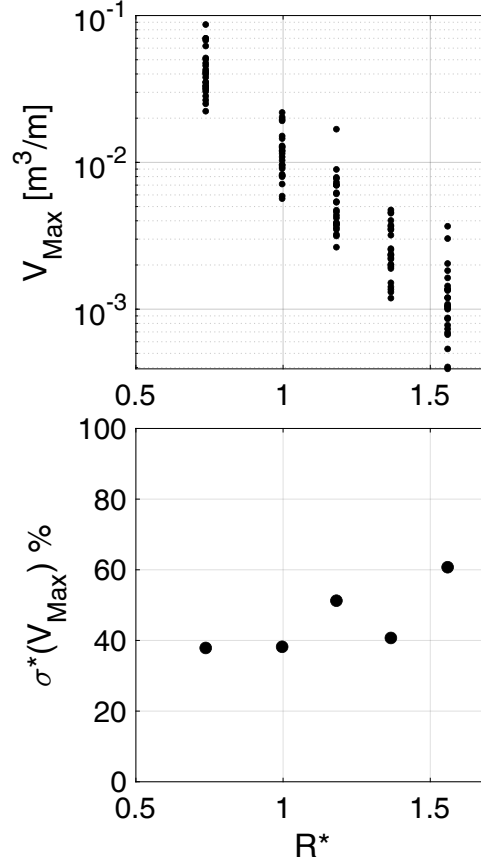


Figure 5.7: Top: scatter of V_{max} against relative freeboard R^* . Bottom: corresponding coefficient of variation $\sigma^*(V_{max})$, defined as the ratio between standard deviation and mean across 25 realizations.

To further investigate the variability associated with extreme overtopping events, Figure 5.7 focuses on the maximum individual overtopping volumes observed during the experimental campaign. The figure summarizes the variability of individual overtopping volumes by plotting the maximum recorded volume V_{max} against the relative freeboard R^* . Each symbol corresponds to the largest event observed among the 25 realizations for a given spectral condition. As expected, V_{max} decreases with increasing relative freeboard, reflecting the lower intensity of overtopping events at higher crest elevations. Nevertheless, even at large R^* values, isolated waves can produce extreme overtopping volumes, several times larger than the mean. The

lower panel shows the coefficient of variation $\sigma^*(V_{max})$ across the 25 realizations. Unlike the mean discharge and the proportion of overtopping waves, here the scatter remains high for all R^* values, with coefficients of variation generally between 40% and 60%. This indicates that the maximum event in each test is highly sensitive to the random seed. In other words, while the expected magnitude of V_{max} decreases with R^* , its statistical variability remains consistently large across the tested conditions, reflecting the inherently stochastic nature of extreme overtopping events.

5.4.2 Wave loads on the breakwater

In addition to overtopping, another aspect of the analysis focuses on the wave-induced loads acting on the structure. These actions have been investigated by distinguishing between those that act on the front face of the structure (horizontal actions) and those that act on the upper face (vertical actions). In particular, horizontal actions are generated by the oscillatory pressure field induced by incident and reflected waves on the seaward face of the caisson, and they govern the global stability of the structure (Goda, 2010; Oumeraci et al., 2001). Vertical actions, on the other hand, are here considered only in relation to post-overtopping events, i.e. the impulsive pressures and forces produced by the downfall of overtopped water masses onto the superstructure (deck), which can locally reach significant intensities (Wolters et al., 2005).

To quantify the influence of random seeding on the loading processes, both local and global quantities have been examined. For horizontal actions, the analysis considered synchronous pressures and synchronous forces. In this case, given the oscillatory nature of the wave-induced pressure field and the absence of significant phase shifts along the vertical wall, synchronous and asynchronous values are effectively equivalent. Consequently, only the maximum pressure P_{Max} recorded at each transducer and the corresponding maximum global horizontal force F_{Max} were analyzed.

For vertical post-overtopping actions, the analysis focused instead on synchronous forces and asynchronous pressures. In this case, the downfall of overtopped water masses generates impulsive and spatially non-uniform impacts on the superstructure, causing the individual pressure transducers to reach their local maxima at different instants. Therefore, asynchronous pressures were analyzed in terms of the $P_{1/250}$ index, defined as the arith-

metric mean of the upper 0.4% tail of each pressure time series. The global vertical forces were examined through their maximum value F_{Max} and their averaged-peak indicator $F_{1/250}$, computed in an analogous manner. Both $P_{1/250}$ and $F_{1/250}$ serve as robust indicators of “severe but not singular” impacts, effectively reducing the influence of isolated outliers while retaining design relevance and statistical stability.

For each analyzed quantity, the coefficient of variation σ^* was computed across the 25 seeds for each spectral condition, providing a quantitative measure of the variability induced solely by the random phase composition of the incident wave field.

5.4.2.1 Global loads on the breakwater

The analysis of global horizontal loads acting on the structure is presented in Figures 5.9 and 5.8. Figure 5.9 shows the distribution of the horizontal P_{Max} values, defined as the maximum pressures recorded synchronously with the instant of maximum horizontal global force, and their variability along the caisson trunk and wave wall (i.e., the vertical front face). The figure is organized in five rows (increasing relative freeboard R^* , i.e., different energy spectra) and two columns. In each row, the left panel reports the synchronous pressure diagram (pressure values in “meters of water column” versus sensor position in meters), while the right panel presents the corresponding coefficient of variation σ^* , which describes the spatial distribution of seed-to-seed variability for the same quantity. Recorded pressures exhibit a pronounced concentration of peaks near the still water level and a rapid attenuation with depth, in agreement with classical observations on oscillating pressures at vertical structures (Goda, 2010). As the relative freeboard R^* increases, absolute pressure levels decrease, while the relative variability grows slightly, with σ^* tending to higher values moving away from the still water level. Quantitatively, σ^* values range overall between 5% and 20%. The variability is lowest (about 5-10%) near the sensors located near the still water level, where pressures are highest. In contrast, where pressures are lower, towards the upper and lower regions, σ^* increases, spanning 10-20%, reflecting a greater sensitivity of these zones to phase differences in the incoming wave field.

Figure 5.8 summarizes the corresponding global horizontal forces derived from the integration of pressures. The upper panel reports the dimensionless maximum forces $F_{Max, Horiz}/(\rho g H_{Max}^2)$, which is a common parameter

used in the literature (Oumeraci et al., 2001), as a function of the relative freeboard R^* , while the lower panel presents the associated coefficients of variation σ^* computed across the 25 realizations. In particular, H_{Max} is not the measured maximum wave height during the tests, but is calculated as $H_{Max} = 1.8H_{m0}$ according to Goda (2010). This facilitates comparison with design conditions and allows designers to estimate expected load levels even when only spectral wave parameters are available. The variability of the global horizontal force remains moderate, with σ^* typically in the range of 10-20%. Slightly higher variability (up to about 20%) is observed for the least energetic wave conditions, where the overall force magnitude is smaller and apparently more affected by phase-to-phase fluctuations in the incident wave field.

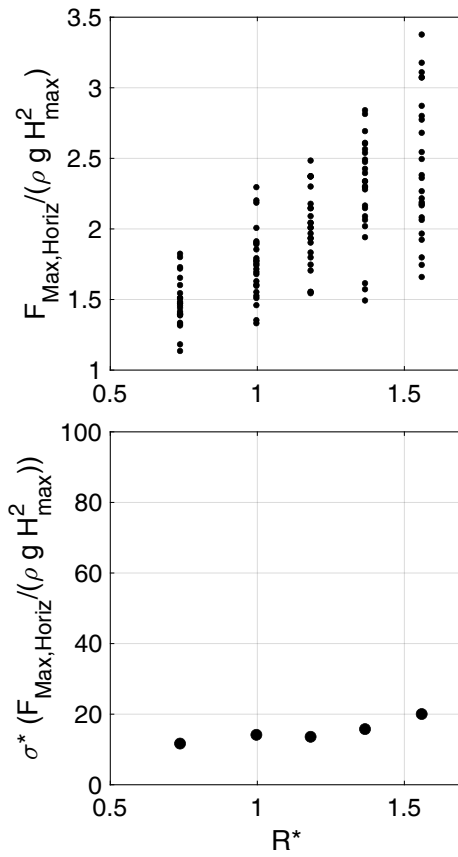


Figure 5.8: Maximum synchronous global horizontal forces acting on the break-water. Upper panel: dimensionless maximum horizontal force $F_{Max, Horiz} / (\rho g H_{Max}^2)$ as a function of the dimensionless freeboard R^* . Lower panel: coefficient of variation computed across the 25 random-phase realizations.

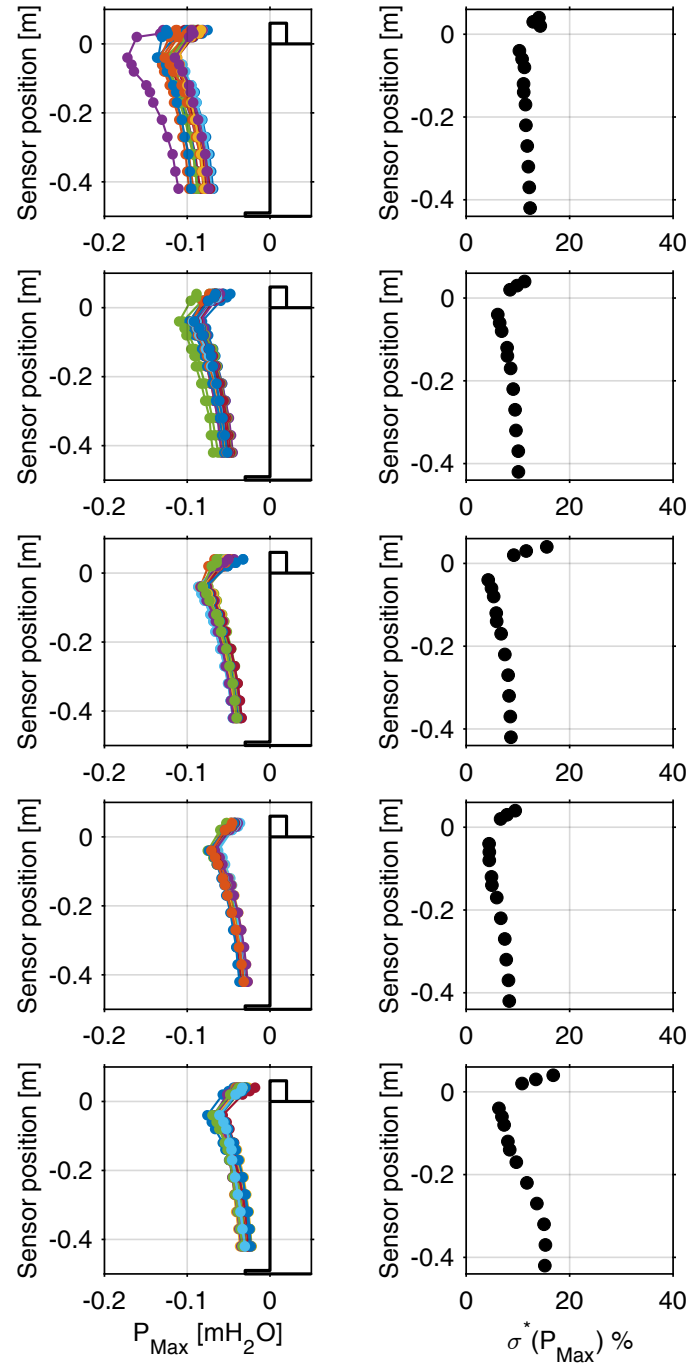


Figure 5.9: Maximum synchronous horizontal pressure distribution (at the moment of the maximum force) along the caisson front face: (left) maximum recorded pressure values P_{Max} for all sensors and spectral conditions (increasing relative freeboard R^* from top to bottom); (right) corresponding coefficient of variation σ^* across the 25 random seeds.

5.4.2.2 Post-overtopping loads on the superstructure

The vertical post-overtopping loads acting on the superstructure are presented in Figures 5.11 and 5.10. These actions result from the downfall of overtopped water masses impacting the deck surface, producing short-duration and spatially non-uniform pressures and forces.

Figure 5.11 shows the distribution of the asynchronous $P_{1/250}$ values recorded by the transducers on the superstructure and the corresponding coefficient of variation σ^* . The figure is organized in five rows (increasing relative freeboard R^*) and two columns. In each row, the left panel reports the asynchronous pressure levels (in “meters of water column”) as a function of the sensor position, while the right panel presents the seed-to-seed variability σ^* expressed in percentage. The variability is relatively high, with σ^* values typically spanning 20-50% for the most energetic sea states. For less energetic conditions, where the overtopping volumes are smaller and the downfall becomes more fragmented, σ^* can locally exceed 100%. This large scatter, both for the most energetic conditions and for the less energetic one, reflects the strong dependence of the process on the random phase composition and on the inherently localized nature of post-overtopping impacts.

Figure 5.10 presents the corresponding global vertical forces generated by post-overtopping downfall events. The upper panels show the dimensionless maximum and averaged-peak forces ($F_{Max,Downfall}$ and $F_{1/250,Downfall}$, normalized by $\rho g H_{Max}^2$) as a function of the relative freeboard R^* , while the lower panels report the corresponding coefficients of variation σ^* . In particular, H_{Max} is evaluated as $H_{Max} = 1.8H_{m0}$ according to Goda (2010), allowing comparison with design-based formulations. The variability of the vertical forces is significant, with σ^* generally ranging between 20-50%, for both graphs. When considering the $F_{1/250,Downfall}$ indicator, which mitigates the influence of isolated outliers, a slightly increasing trend of σ^* with R^* can be observed.

Although the present phenomenon differs from direct wave impacts on vertical walls, a comparable degree of variability has been extensively documented for impulsive wave loading under laboratory conditions. Even under nominally identical incident waves, impact magnitudes have been observed to vary markedly between realizations. This intrinsic variability has been associated with several factors, including turbulence, residual long-wave motion in the flume or basin, local crest instabilities, and air entrainment or

entrapment at the moment of impact (Bagnold, 1939; Hattori et al., 1994; Peregrine, 2003; Bullock et al., 2007; Cuomo et al., 2010a; Lubin et al., 2019; van Meerkerk et al., 2021). Its effects have been demonstrated in repeatable laboratory tests, where even when the identical paddle signal was re-run, and thus the same nominal wave train was generated, large differences in impact pressures were observed from one realization to another (Chan and Melville, 1988; Bogaert et al., 2010; Marzeddu et al., 2017). In the present experiments, the downfall loads show the same impulsive and highly variable behavior as direct wave impacts, mainly governed by turbulence, local instabilities, and air entrainment. Beyond these intrinsic mechanisms, the variability observed here is further amplified by the random phase composition of the incident wave field (seed variability).

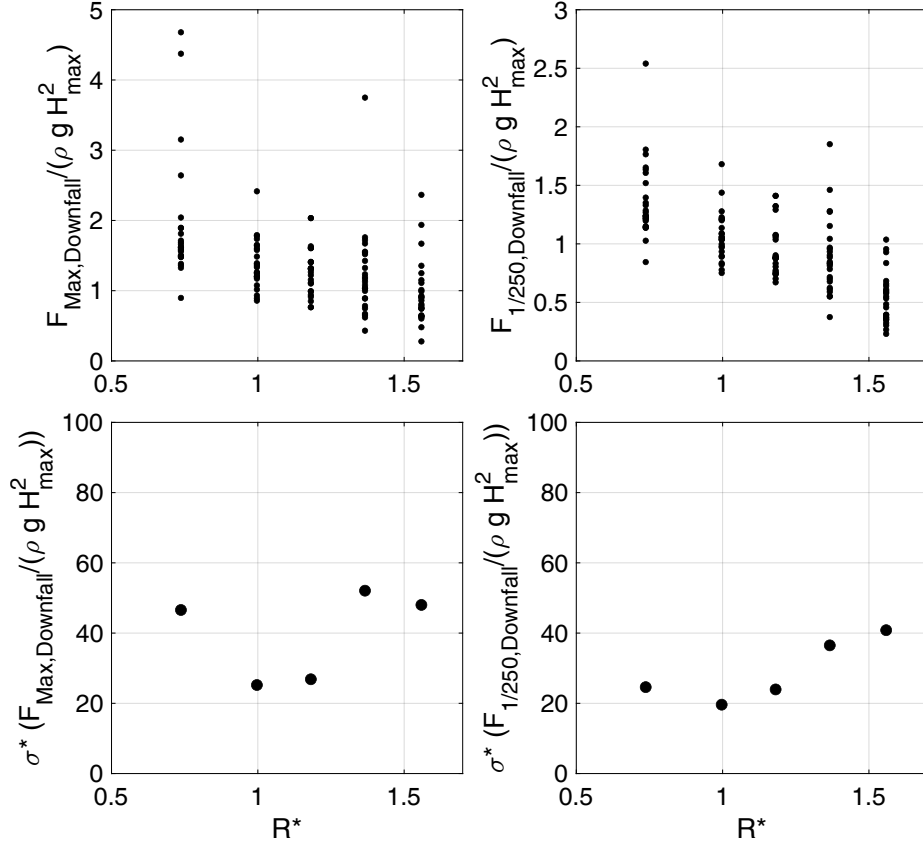


Figure 5.10: Maximum synchronous global vertical forces generated by post-overtopping downfall on the superstructure. Upper panels: dimensionless maximum force $F_{Max,Downfall}/(\rho g H_{max}^2)$ and averaged-peak force $F_{1/250,Downfall}/(\rho g H_{max}^2)$ as a function of the dimensionless freeboard R^* . Lower panel: coefficient of variation computed across the 25 random-phase realizations.

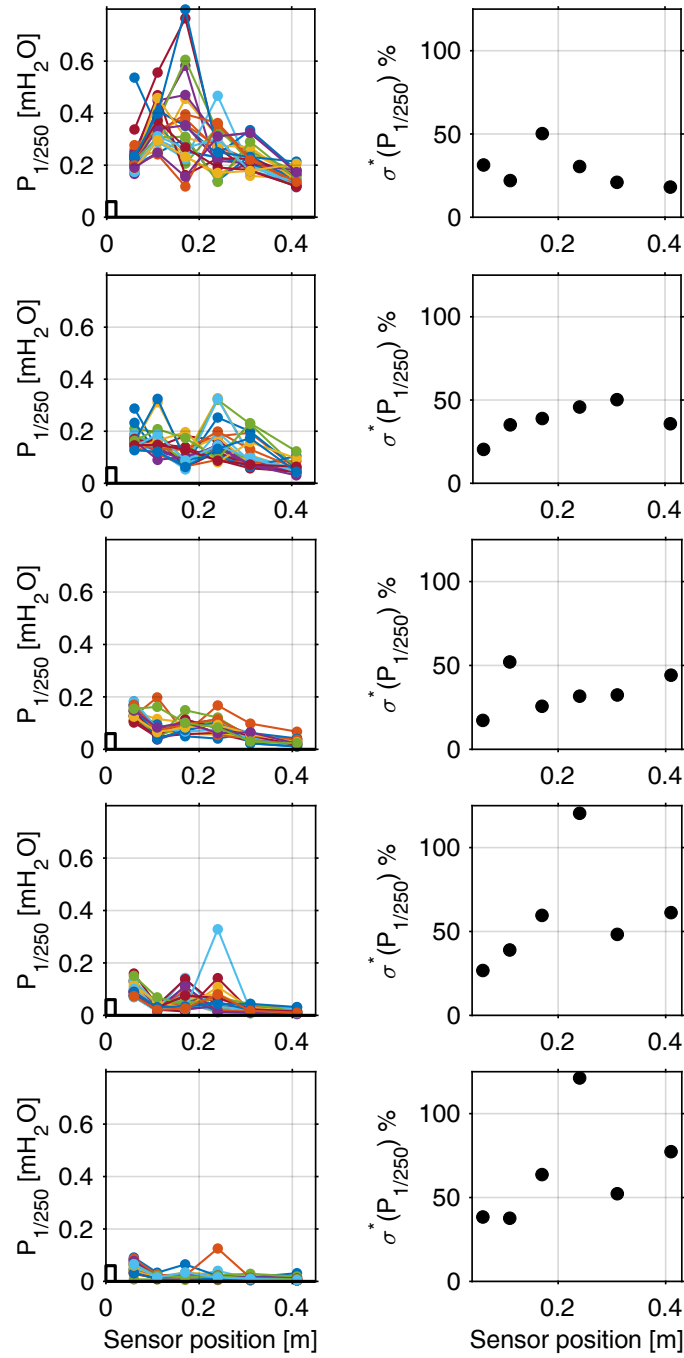


Figure 5.11: Asynchronous vertical pressure (downfall) distribution along the caisson superstructure: (left) mean of the upper 0.4% pressure tail $P_{1/250}$ for all sensors and spectral conditions (increasing relative free-board R^* from top to bottom); (right) corresponding coefficient of variation σ^* across the 25 random seeds.

5.5 Key insights of this chapter

This chapter investigated the influence of random phase seeding on wave overtopping and wave-induced loads acting on a flushed-wall vertical breakwater, with particular emphasis on the intrinsic variability associated with irregular sea states. The experimental results clearly demonstrate that random seeding plays a significant role in the estimation of both mean and extreme hydraulic responses, especially when limited realizations are considered. Significant scatter was observed in global overtopping indicators, such as the mean overtopping discharge, the proportion of overtopping waves, and the maximum individual overtopping volumes, with coefficients of variation that strongly depend on the relative freeboard.

For low relative freeboards, characterized by frequent overtopping, the coefficient of variation of the dimensionless mean overtopping discharge and of the proportion of overtopping waves remains typically below 20 %, reflecting the stabilizing effect of a large number of overtopping events within each realization. Conversely, for high relative freeboards, where overtopping becomes rare and is governed by a limited number of waves, the variability increases markedly. In this regime, coefficients of variation of 40-50 % are observed for mean overtopping discharge and overtopping probability, while the coefficient of variation of the maximum individual overtopping volume can exceeds 50-60 %, with marked differences between the realizations under identical spectral conditions.

Wave-induced loads exhibit a lower sensitivity to random seeding compared to overtopping quantities. Maximum global horizontal forces acting on the caisson show coefficients of variation generally below 20 % across the investigated freeboard range. Also local pressures, in particular maximum synchronous pressures along the caisson front face, display a coefficients of variation generally below 20 %. Post-overtopping downfall pressures and the associated vertical forces on the superstructure show higher variability levels, with coefficients of variation typically in the range 20–50 % and with some peaks that reach the 100% for the impulsive pressures.

It should be emphasized that the present assessments and analysis can be significantly expanded in future work, for instance by extending the statistical treatment to the full wave-by-wave distribution of individual overtopping volumes (and their grouping) and to their joint occurrence with impact/pressure extremes.

The investigations presented so far have focused on the hydraulic response of vertical breakwaters under operational conditions, addressing wave-induced loads and overtopping processes for fully functional structures. However, critical and potentially vulnerable scenarios may also arise outside the operational phase. For this reason, the following chapter shifts the focus to the construction stage of caisson breakwaters, investigating wave-induced loads acting on internal cells temporarily exposed before the completion of the cast-in-situ superstructure.

Chapter 6

Phenomenological investigation of wave-induced loads on the internal cells of caisson breakwaters during the construction phase

6.1 Introduction

Previous chapters have explored the optimization of vertical breakwaters equipped with flushed or retreated wave walls during the operational phase. In those configurations, the presence of a crest superstructure played a central role in shaping the post-overtopping dynamics and the resulting impact loads. In contrast, the present chapter addresses a different physical problem, relevant to the construction phase of caisson breakwaters, when the structure is without the cast-in-situ superstructure and the internal compartments are open. Under these conditions, post-overtopping flows can directly enter the caisson and interact with the internal walls, generating hydrodynamic processes significantly different from those documented in Chapter 4. Addressing this knowledge gap is essential to improve safety margins and design guidelines for breakwaters exposed to moderate storm conditions during incomplete construction stages.

Nowadays a typical vertical breakwater, in deep water, consists of a rubble-mound foundation, a monolithic, cellular, and reinforced concrete

caisson, and a cast-in-situ superstructure equipped with a wave wall. This configuration is designed to ensure structural integrity under complex hydrodynamic loads, including wave-induced pressures and overtopping.

The construction cycle involves three main phases: (i) the launching of the pre-fabricated caisson, (ii) the temporary installation on the seabed using seawater ballast, and (iii) the final filling with sand or concrete and the completion of the superstructure (Franco, 1994). The launching represents the first crucial moment of the process: once completed at the construction site or in a dry dock, the caisson, a large hollow reinforced concrete structure equipped with internal compartments (cells) separated by diaphragms (partitions), It is progressively flooded in construction tanks or through mobile bulkheads, until it becomes floating.

At this point, the caisson behaves like a large barge and is towed, with the aid of tugboats, from the prefabrication site to the exact installation point at sea. During navigation, the floating trim and stability are controlled by adjusting the amount of water in the temporary ballast cells. The launching and transfer require detailed planning and favorable weather windows: the large dimensions of the caissons and their high moment of inertia make them vulnerable to wave motion and even moderate currents, which could compromise the safety of the operation or cause significant deviations from the planned trajectory.

Once positioned at the designated point in the worksite, the ballasting phase on the seabed begins, but until the superstructure is completed and the final filling is carried out, the internal cells, bounded by concrete diaphragms not yet protected or covered with temporary concrete cover slabs, remain exposed to post-overtopping flows that exceed the crest freeboard. The post-overtopping flow impacts the caisson's internal diaphragms and can generate over-pressures that are still difficult to quantify. The actual duration of the temporary phase, estimated during the design stage, may vary and is influenced by meteorological, logistic, and environmental factors that can expose the caisson to significant sea states, capable of compromising the integrity of the temporary structure.

As extensively discussed in previous chapters, the existing technical literature provides detailed descriptions of the loads acting on the seaward face and crown wall of a completed structure, relying mainly on the formulations by Goda (Goda, 2010), later extended to impulsive breaking conditions by

Takahashi (1996), and on the comprehensive guidelines developed within the European PROVERBS Project, which offers probabilistic design tools for composite vertical breakwaters and seawalls under both breaking and non-breaking waves (Oumeraci et al., 2001). These recommendations also include procedures for estimating the magnitude and duration of wave impacts on vertical structures (Martinelli and Lamberti, 2011), while the evaluation and verification of wave overtopping discharges, essential for assessing the hydraulic performance of coastal defenses, can be effectively conducted using the predictive methods outlined in the Manual EurOtop (2018).

Over the past decades, as discussed in the Introduction 1, numerous optimization strategies for vertical breakwaters, from perforated and multi-chamber caissons to modified crown geometries and retreated wave walls, have proven effective in enhancing hydraulic performance. Despite these improvements, post-overtopping flows may still develop and generate complex hydrodynamic interactions, as highlighted in Chapter 4. While such phenomena have been extensively studied in naval and offshore engineering, their implications for coastal structures remain only partially understood. This issue becomes particularly relevant during construction phases, when the internal cells of a caisson are open or only partially filled and therefore more vulnerable to wave-induced pressures.

Existing research, however, focuses mainly on operational configurations and largely overlooks the hydrodynamic processes occurring inside these temporarily open cells. Experimental data and reliable models are still lacking for the spatio-temporal distribution of pressures, the behaviour of entrapped water, and the potential role of air compressibility following overtopping. In the absence of such information, the assessment of construction-stage scenarios necessarily relies on conservative assumptions, which may lead either to unnecessary over-dimensioning or, conversely, to insufficient safety margins, ultimately affecting the resilience and cost-efficiency of the structure.

In this chapter, attention is focused on the hydrodynamic processes occurring within the internal cells of a caisson during the construction phase, when the compartments are open and therefore directly exposed to waves. Using a combined experimental and numerical (CFD) approach, the chapter investigates the phenomenological impact mechanisms generated by water propagation across the caisson, with particular emphasis on the differences between partially and fully filled configurations.

The chapter is conceived as an exploratory and phenomenological investigation rather than a design-oriented study, and it does not aim to provide generalized predictive relationships or formulations for wave-induced loads on internal caisson cells. Instead, the objective is to improve the physical understanding of the main loading mechanisms acting on internal walls. This preliminary investigation is intended as a first step toward a more comprehensive understanding of wave-induced loads on caisson cells during construction, providing a framework upon which future, more extensive analyses may build. The geometrical configuration investigated in this chapter is an idealized representation of a vertical cellular caisson breakwater, defined to be representative of typical Italian deep-water vertical breakwaters (Franco, 1994). Accordingly, the study is not intended to describe any site-specific structure, but to investigate the phenomenology of wave-induced loads within open internal cells during construction.

The chapter is structured as follows. After this introduction, the description of the physical model within the numerical model is provided. Then, a detailed description and discussion of the hydrodynamic processes is given. Finally, some insights close the chapter.

6.2 Experimental model

The experimental campaign was conducted, in the mid-scale wave flume of Roma Tre University, using a scaled physical model of a vertical cellular caisson breakwater without the cast in situ superstructure (see Figure 6.1, panel c), representing a vertical breakwater (height $h_t = 0.50$ m) positioned on top of a rubble mound foundation (height $h_b = 0.25$ m). The foundation consisted of an inner core of uniform sized rock material and an external rubble mound layer made of rocks weighing 20.0-30.0 g. The external walls of the caisson trunk were built with 15 mm thick marine plywood panels to ensure structural rigidity, while the internal vertical cells (height $h_c = 0.20$ m) reproduced the geometry of a real caisson compartments, constructed using 4 mm thick plywood sheets for the cells in front of the instrumented one and 3D-printed plastic for the cell equipped with pressure sensors (i.e., the last measuring cell). The water depth, and consequently the crest freeboard, were maintained constant at $h=0.7$ m and $R_c=0.035$ m, respectively.

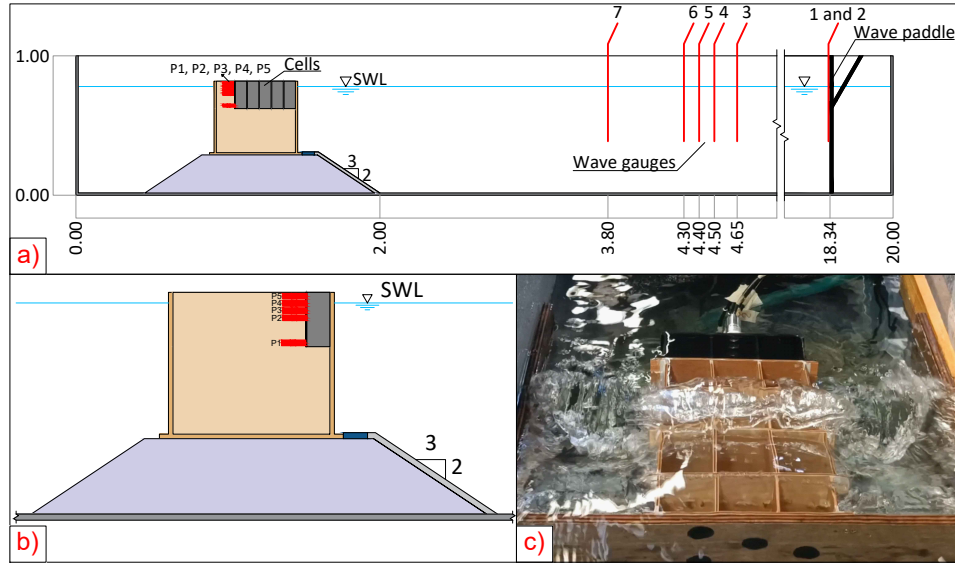


Figure 6.1: Panel a): Longitudinal section of the model structure and wave gauges setup in the wave flume. Panel b): Longitudinal view of the caisson breakwater, with indication of the geometrical parameters and positions of the pressure sensors. Panel c): Photo of the model structure with internal walls interacting with a wave.

To better clarify the experimental layout, Figure 6.2 shows a render of the separated internal cells tested. Due to the limited space within the scaled model did not allow simultaneous installation of pressure sensors on

all internal walls, the experimental campaign was conducted in five repeated test series, with sensors installed on a different wall in each series, from the first to the fifth cell. The identical free surface elevation time series was reproduced in every repetition to ensure identical experimental conditions and directly comparable results, allowing reliable and consistent measurements of pressures exerted by post-overtopping flows on all internal diaphragms of the caisson while guaranteeing repeatability and robustness of the data collected. Rather than replicating a single site-specific prototype, the physical model was designed according to Froude similarity to reflect a wide range of vertical breakwater configurations typical of Italian deep-water ports (Franco, 1994). The experimental model represents a 1:55 geometrical scale reproduction of the numerical model, which was developed at full prototype scale.

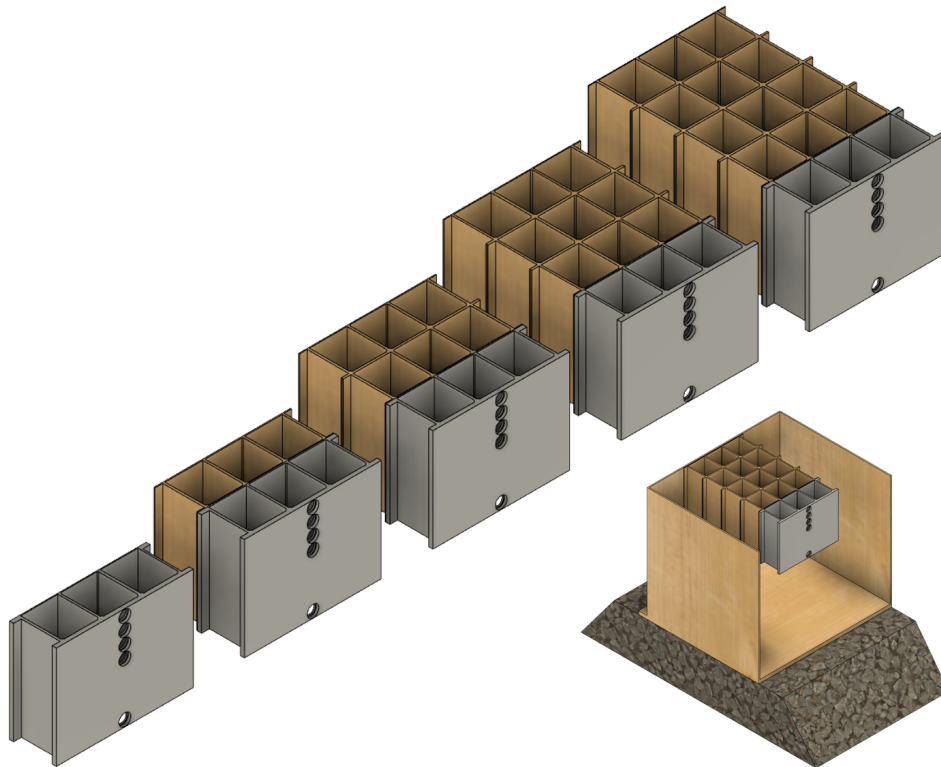


Figure 6.2: Measurement sequence and render of the caisson components forming the internal cells.

Pressure sensor	Vertical position (m)	Influence area (m^2/m)
$P1$	0.1675	0.0540
$P2$	0.0845	0.0535
$P3$	0.0605	0.0240
$P4$	0.0365	0.0240
$P5$	0.0125	0.0245

Table 6.1: Pressure sensor names (first column), positions (second column) and areas of influence (third column). The position is the vertical distance from the highest point of the structure (the crest), so P1 is the lowest sensor (immersed) and P5 is highest sensor (emerged).

Panel a) of Figure 6.1 reports a sketch of the longitudinal section of the breakwater and wave gauge setup, inside the wave flume, with measures in meters. A total of seven resistive wave gauges are placed along the flume to measure the free surface elevation. Two wave gauges are mounted on the piston-type wave-maker for the active absorption system (WG1 and WG2). Five wave gauges have been aligned in the central longitudinal axis of the flume, to measure the wave field in front of the structure (WG3-WG7). The acquisition frequency of the wave gauges is set at 50 Hz, and a calibration procedure is carried out at least twice a day to account for environmental variations. A total of 5 pressure transducers are fixed on the measuring cell, which have been moved from test to test. Panel b) of Figure 6.1 reports a sketch of the section of the structure with the pressure sensors installed inside the cells. Table 6.1 reports the names, positions and influence area of the pressure sensors (so called P1-P5).

6.2.1 Test Program

The experimental campaign was designed to investigate the wave-induced pressures acting on the internal walls of the caisson under regular wave conditions, eight wave conditions were tested with target wave heights (H) ranging from 0.06 m to 0.09 m and periods (T) between 0.9 s and 1.3 s, as summarized in Table 6.2. For each wave condition, as stated, five repetitions were carried out, each time instrumenting a different internal wall with the pressure sensors. This approach was necessary due to the limited space within the scaled model, which did not allow simultaneous placement of the sensors on all internal partitions. To ensure data consistency and comparability among the different cells, exactly the same wave time series was reproduced in each repetition, guaranteeing that all five measurements were performed under identical hydrodynamic conditions. This methodology enabled the construction of a complete pressure profile across all internal compartments of the caisson, capturing the evolution of wave-induced loads from the seaward cell (Cell 1) to the most landward one (Cell 5). The duration of each test series was approximately 80 seconds, a time long enough to reach stationary conditions on the structure.

Sea State ID	T (s)	H (m)
RW1	0.9	0.06
RW2	0.9	0.07
RW3	1	0.06
RW4	1	0.07
RW5	1.1	0.07
RW6	1.1	0.09
RW7	1.3	0.07
RW8	1.3	0.09

Table 6.2: Summary of the parameters (H and T) for the eight regular sea states used for the experimental tests.

6.3 Numerical model

The numerical simulations presented in this work were performed using the IHFOAM solver (Higuera et al., 2013a,b), an open-source extension of the OpenFOAM[®] CFD framework (Jasak, 1996). IHFOAM is based on the *interFoam* solver and includes specialized boundary conditions for wave generation and absorption, as well as porous media formulations, making it particularly suitable for coastal and offshore engineering applications (Higuera et al., 2014a,b; Castellino et al., 2018; Romano et al., 2020). The model

is capable of solving both the three-dimensional Reynolds-Averaged Navier-Stokes (RANS) equations and the Volume-Averaged Reynolds-Averaged Navier-Stokes (VARANS) equations for two-phase flows. The RANS and VARANS formulations are coupled with the Volume of Fluid (VOF) method (Berberović et al., 2009) to capture the interface between air and water. In this section, the governing equations and the main features of the adopted numerical approach are briefly described. The RANS equations, applied to the clear fluid region, are derived through the Reynolds decomposition, which separates each flow variable (e.g., velocity and pressure for incompressible flows) into mean and fluctuating components. These equations express the conservation of mass and momentum, and are coupled with the VOF equation as follows:

$$\frac{\partial u_i}{\partial x_i} = 0 \quad (6.1)$$

$$\frac{\partial \rho u_i}{\partial t} + u_j \frac{\partial \rho u_i}{\partial x_j} = -g_j x_j \frac{\partial \rho}{\partial x_i} - \frac{\partial p^*}{\partial x_i} - f_{\sigma i} - \frac{\partial}{\partial x_j} \mu_{\text{eff}} \left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) \quad (6.2)$$

$$\frac{\partial \alpha}{\partial t} + \frac{\partial u_i \alpha}{\partial x_i} + \frac{\partial u_{c,i} \alpha (1 - \alpha)}{\partial x_i} = 0 \quad (6.3)$$

In the above equations, the subscript $i = 1, 2, 3$, and j , represent the three coordinate directions, obeying the Einstein summation convention, the u_i (m/s) are the ensemble-averaged velocity components, x_i (m) the Cartesian coordinates, g_j (m/s²) the components of the gravitational acceleration, ρ (kg/m³) the density of the fluid, $p^* = p - \rho g_j x_j$ (Pa) is the dynamic pressure in excess of the hydrostatic component, $\mu_{\text{eff}} = \mu + \rho \nu_t$ (Pa·s) is the effective viscosity accounting for both molecular and turbulent contributions, $f_{\sigma i} = \sigma \kappa \frac{\partial \alpha}{\partial x_i}$ (N/m³) represents the surface tension force (Brackbill et al., 1992), and α is the VOF scalar denoting the water volume fraction ($\alpha = 1$ for water, $\alpha = 0$ for air). The interface compression velocity $u_{c,i}$ is computed as $u_{c,i} = \min[c_\alpha |u_i|, \max(|u_i|)] \frac{\partial \alpha / \partial x_i}{|\partial \alpha / \partial x_i|}$, where $c_\alpha = 1$ is the compression coefficient controlling interface sharpness (Weller, 2008; Marschall et al., 2012). To close the system equations, turbulence have been modeled through the k - ω SST model (Menter, 1994), which provides a robust formulation for simulating highly sheared and separated flows near the free surface. The model includes the improved near-surface correction proposed by Larsen and Fuhrman (2018) to mitigate turbulence overproduction in wave-affected regions.

6.3.1 Implementation of the numerical simulation

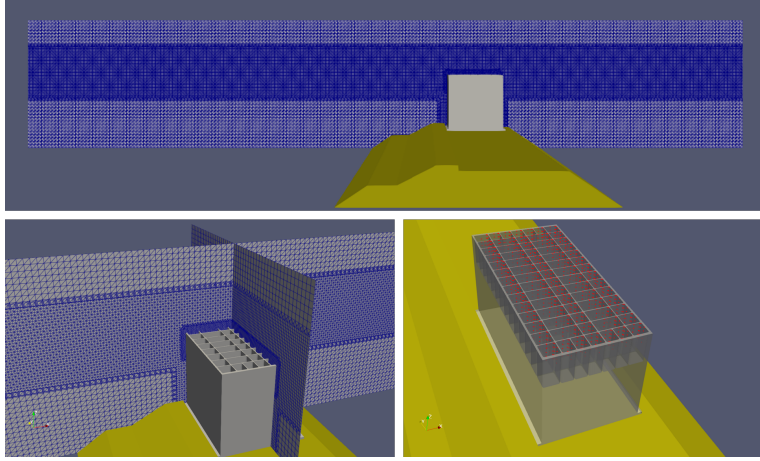


Figure 6.3: Panel a): 2D longitudinal view of the numerical wave flume. Panel b): 3D view of the computational mesh with details of local body-fitting and refinement. Panel c): 3D view of the caisson breakwater in transparency with details of the internal cells and position of the pressure sensors.

The numerical domain was designed to reproduce, at full prototype scale, the geometry of the caisson structure described in Section 6.2, which was tested experimentally at a 1:55 scale. The computational flume extended for 400.0 m in length, 75.0 m in height, and 40.0 m in width, see Panel a) of Figure 6.3. The geometry of the structure was exactly replicated, and the same regular wave conditions of the experimental campaign (Table 6.2) were imposed at the inlet boundary to ensure a one-to-one comparison between laboratory and numerical results. An active absorption boundary condition was applied on the wave-generation side of the flume, and along the solid, impermeable boundaries (the walls and the bottom), the numerical model imposes a no-slip velocity condition. The mesh consisted of structured cells with local refinement around the caisson internal chambers. Different mesh setups have been considered to assess the influence of the mesh resolution on the results. The chosen base grid spacing was $\Delta x = 1.9$ m in the longitudinal direction, $\Delta y = 1.9$ m in the transverse direction, and $\Delta z = 1.9$ m in the vertical one, reduced to $\Delta x = 0.5$ m and $\Delta z = 0.5$ m close to the structure. A variable time step was adopted to maintain a Courant number below 0.50, resulting in Δt between 0.01 s and 0.03 s.

6.4 Results and discussion

6.4.1 Qualitative description of the observed phenomena

All simulations, both physical and numerical, were initially analyzed from a qualitative point of view. During the laboratory tests on the 2Dv physical model and the CFD numerical simulations, the behavior of wave motion interacting with the structure in its temporary configuration, without the superstructure of the crown, was observed. Across all tested conditions, no wave breaking was observed either along the channel or near the structure. The post-overtopping flow consisted of compact, continuous water jets (“green water”) spilling over the caisson crest into the internal cells. No air entrainment was detected in the overtopping flow. From the observations, the post-overtopping process was articulated in several phases:

- **Initial phases:** the first overtopping waves spilled over the structure, impacting the exposed cells, which were initially only partially filled up to the mean sea level. The impact generated impulsive loads with localized high over-pressures on the internal walls immediately behind the seaward face.
- **Subsequent phases:** as the cells progressively filled, the level of internal water increased until it reached the top of the caisson trunk. In these conditions, subsequent impacts became milder, still maintaining high pressure values.
- **Bore propagation:** the complete filling of the cells favored the propagation of the water jet along the top of the caisson, allowing the water stream to reach even the rear cells. Although the energy gradually attenuated towards the port side, the hydrodynamic action remained significant even in the farthest cells.

The observed phenomenology highlights how the nature of the induced loads is partly correlated with the degree of filling of the cells, with a transition from impulsive impacts, characterized by high localized over-pressures, to quasi-impulsive impacts that still maintain significant pressures. To support this qualitative description, Figure 6.4 presents a sequence of frames extracted from the experimental videos, illustrating the overtopping process, the impact of the jets on the exposed internal walls, and the subsequent propagation of the flow along the top of the caisson.

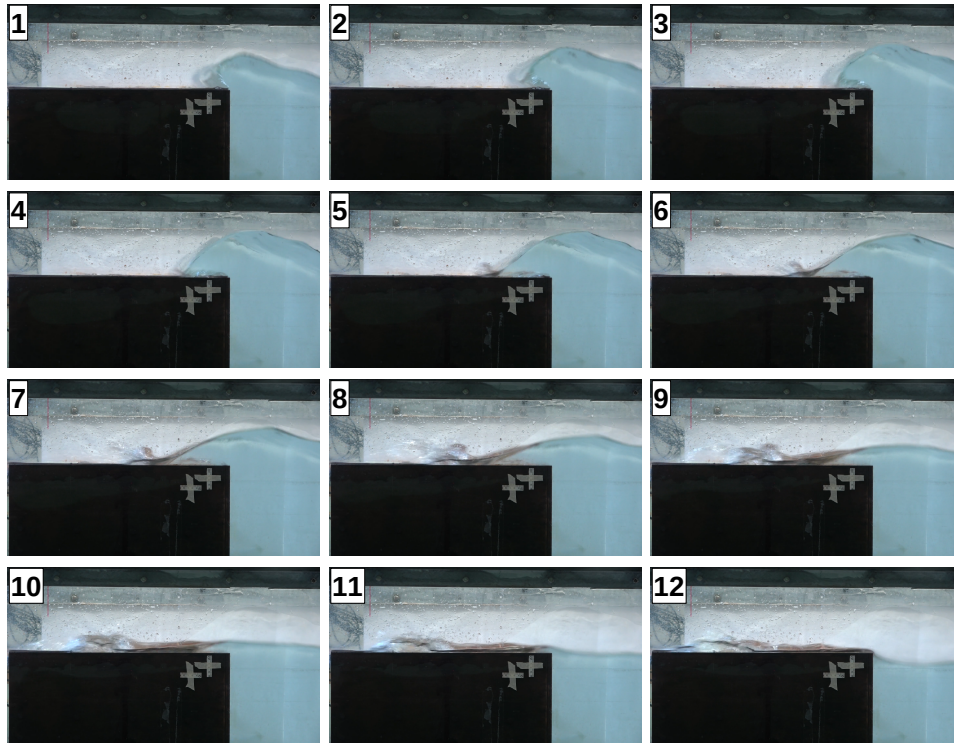


Figure 6.4: Frame sequence of the 2Dv experimental tests showing the impact of the post-overtopping flow on the cells and the propagation of the flow on the top of the caisson.

To further support this qualitative description, Figures 6.5 and 6.6 present two representative sequences from the numerical simulations, illustrating the interaction between the wave motion and the caisson along the middle section during one wave cycle. Figure 6.5 refers to the early stages of the process, when the internal cells are only partially filled up to the mean water level, while Figure 6.6 shows the subsequent steady state condition, when all cells are completely filled. In both cases, the CFD visualizations reproduce the same hydrodynamic patterns observed experimentally: the overtopping jets impacts the internal walls, generating localized pressure peaks that progressively evolve into smoother as the water level rises within the cells. The transition from impulsive to quasi-impulsive behavior is clearly visible in the velocity field and free-surface evolution, confirming the qualitative observations obtained from the physical model.

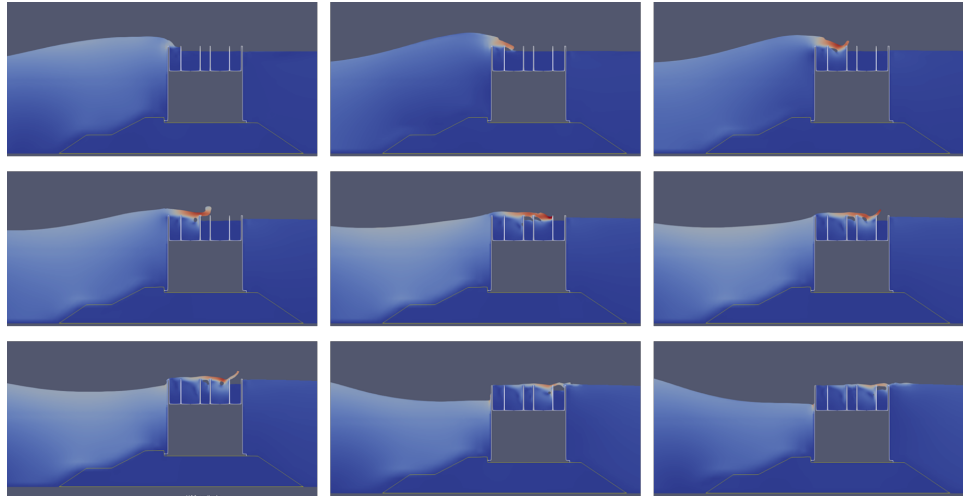


Figure 6.5: Frame sequence of the CFD simulation tests showing the impact of the post-overtopping flow on the partially filled cells.

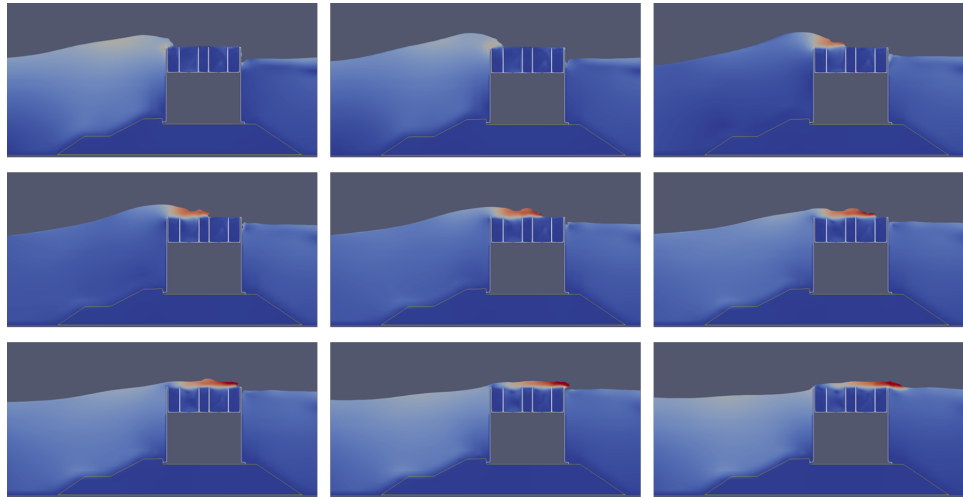


Figure 6.6: Frame sequence of the CFD simulation tests showing the impact of the post-overtopping flow on the completely filled cells.

These hydrodynamic aspects are further analyzed in Figure 6.7, which provides detailed views of the interaction between the overtopping flow and the internal walls for two example cells (one above and one below respectively), under two distinct conditions, partially filled (left panels) and completely filled (right panels). The middle panels show the corresponding pressure signals recorded at the impact areas, highlighting the time windows characterized by impulsive loads (red-shaded regions) and quasi-impulsive loads (blue-shaded regions). It can be observed that, during the initial

stages, the first overtopping waves produce highly impulsive loads associated with rapid jet impacts on the exposed cells (partially filled). Once the cells become fully filled, the flow transitions to a quasi-impulsive regime, characterized by reduced pressure peaks. Therefore, given the impulsive nature of the impacts during the early phase of the process, when air pockets are trapped by the overtopping jets (in conditions of partially filled cells), and considering the characteristics of the CFD model, in which air is treated as an incompressible fluid, making it impossible to accurately quantify such impulsive dynamics. For this reason, the numerical analysis focuses primarily on the quasi-impulsive regime, which is representative of the steady operational conditions of the system.

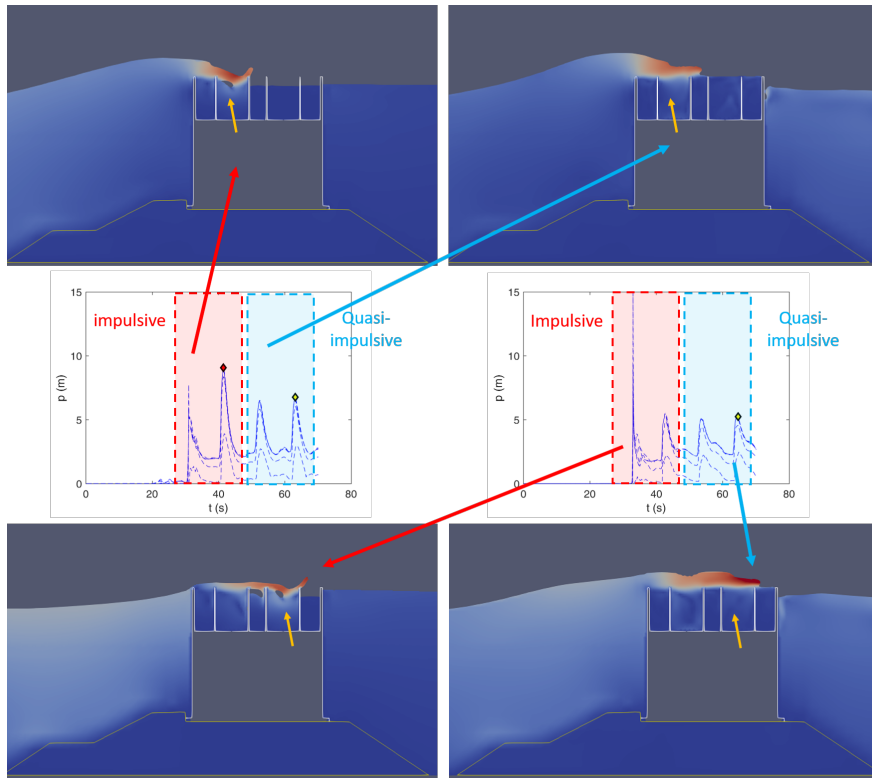


Figure 6.7: Top and bottom panels: details of the hydrodynamic interaction between overtopping waves and internal walls for two example cells (Cell 2 and 4) under partially (left) and fully (right) filled conditions. Middle panel: pressure signals measured on the internal wall of the cells during the events shown in the top and bottom panels. The shaded regions indicate time windows characterized by impulsive (in red; partially filled cells) and quasi-impulsive (in light blue; fully filled cells) loadings.

6.4.2 Validation of the numerical simulation

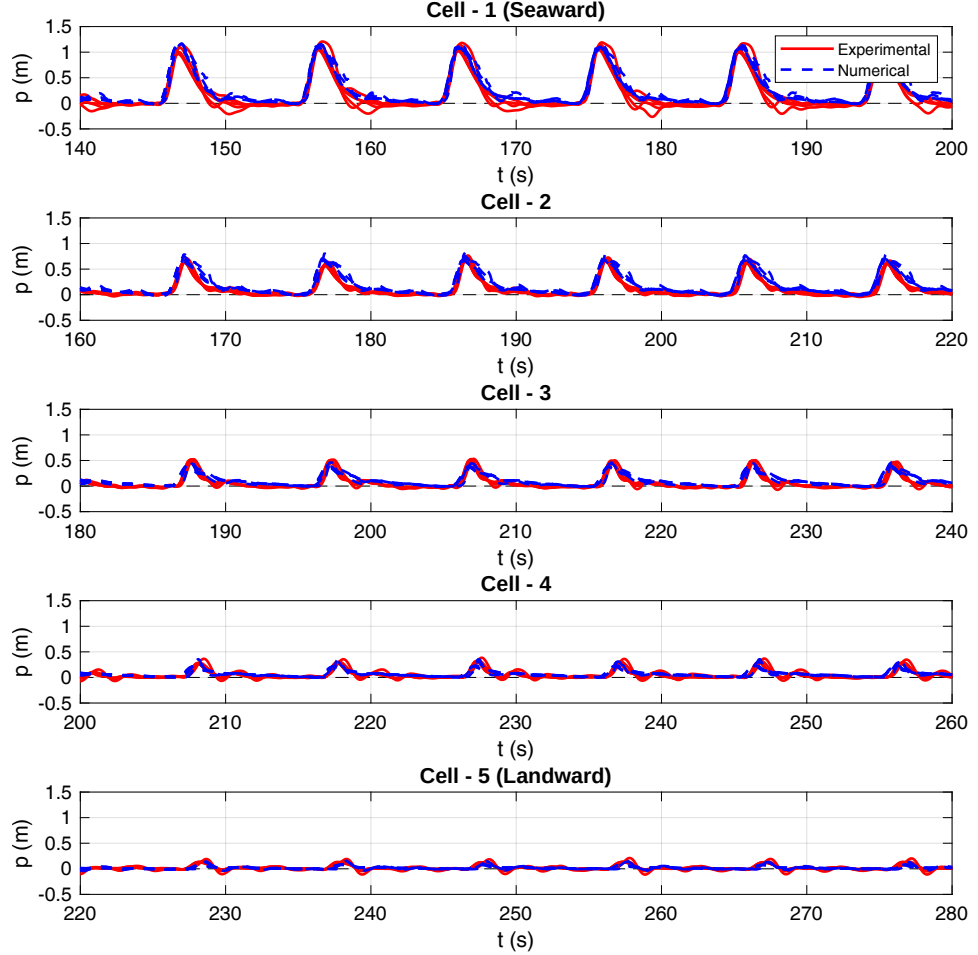


Figure 6.8: An example of the comparison between experimental (red) and numerical (blue) pressure time series, showing only the dynamic pressure component, recorded at the locations of the pressure sensors (P1-P5) for the five internal cells (Cells 1-5). The test shown is number 7 in the Table 6.2 ($H=0.07$ m , $T=1.3$ s at laboratory scale).

The numerical model was validated by reproducing the same regular wave conditions tested experimentally and comparing the resulting dynamic pressure time series. Numerical pressure sensors were placed at the same locations as the experimental sensors (P1-P5), allowing a direct, sensor-by-sensor comparison. It is important to note that the validation was carried out only on the stationary part of the phenomenon. This choice stems from the fact that the physical and numerical models exhibit different ramping times, which leads to a mismatch during the very early stages of wave generation,

and the first wave impacts are impulsive. An example of the comparison is shown in Figure 6.8, where experimental (red lines) and numerical (blue lines) dynamic pressure time series are reported for the five internal cells of the caisson (Cells 1-5). For each cell, the curves correspond to the individual pressure sensors installed on the internal wall. The numerical results closely follow the experimental measurements, reproducing both the timing and amplitude of the pressure peaks across all cells and capturing the main pressure dynamics observed in the laboratory.

6.4.3 Analysis of pressure signals

Experimental and numerical analyses were carried out in a complementary manner, reproducing identical regular wave conditions, with the specific aim of exploring the phenomenology of the wave-induced loading processes acting on the internal caisson walls rather than deriving design-oriented metrics. The analysis of pressure time histories was therefore conceived to identify the characteristic phases of the post-overtopping process and to relate them to the observed hydrodynamic mechanisms. The recorded signals exhibit a repeatable sequence of phases within the wave cycle, reflecting the progressive evolution of the interaction between overtopping flows and the internal cells.

As an illustrative example, Figure 6.9 reports pressure signals recorded by five sensors installed on the same internal wall during an experimental test, which clearly highlight the main stages of the post-overtopping process, consistently with the qualitative observations discussed in the previous section:

- **Initial filling phase:** a rapid increase in pressure is observed, attributable to the hydrostatic loading associated with the rise of the internal water level in the cells.
- **First impacts:** impulsive pressure peaks are recorded, generated by the direct impact of the post-overtopping flow on the partially filled cells.
- **Steady-state impacts:** with the complete filling of the cells, impacts become quasi-impulsive, with a more uniform pressure distribution along the water column, although reaching high pressure values.
- **Load transfer:** the wave-induced load is transmitted uniformly along the entire water column contained in the cells.

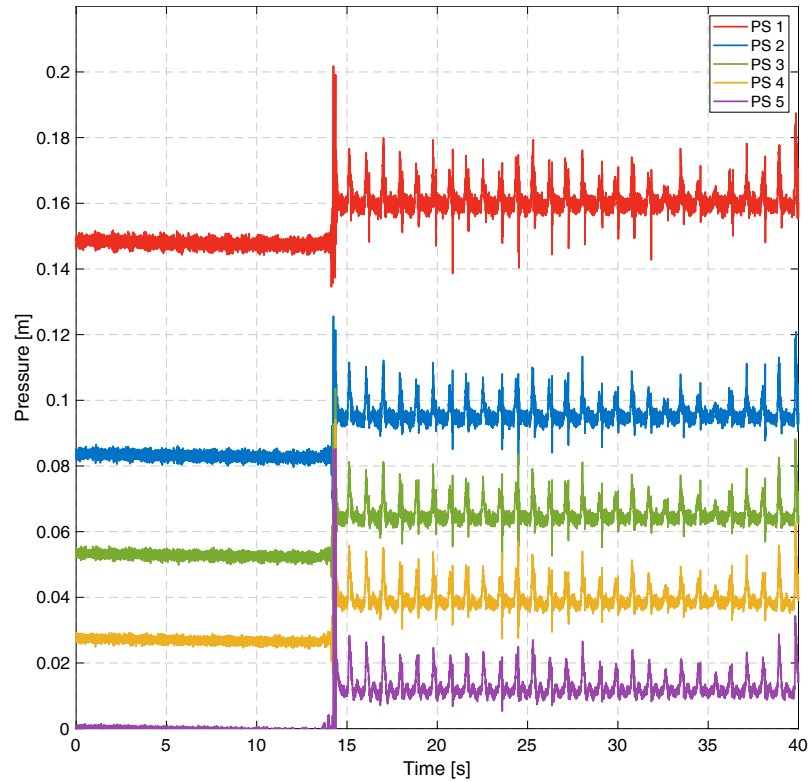


Figure 6.9: Example of pressure signals recorded by five sensors installed on the same internal wall (Cell 2) during a 2Dv physical model test at laboratory scale. The signals are superimposed in a single graph and show the main phases of the overtopping process. The test shown is number 2 in Table 6.2.

The pressure signals recorded on the internal walls of the caisson cells were analyzed focusing on the extreme values associated with individual overtopping events. Given the impulsive and highly intermittent nature of the pressure time histories, a Peak Over Threshold (POT) approach was adopted. For each pressure sensor, peaks exceeding a predefined threshold were identified, and the maximum pressure value associated with each overtopping event was extracted and retained for subsequent analysis.

The pressure values considered in the analysis derive only from experimental results and exclude the first recorded peak, which is often characterized by impulsive response. This initial peak was discarded since, under laboratory conditions, the first incoming wave propagates and interacts with the structure under still-water conditions, a scenario that is unlikely to occur in real sea states, where wave-structure interaction typically takes place

under already disturbed hydrodynamic conditions.

To allow comparison across different wave conditions and test configurations, the extracted peak pressures were normalized by the incident wave height, as $P_{max}/(\rho g H)$. where P_{max} is the maximum pressure measured at a given sensor during an event, ρ is the water density, g the gravitational acceleration, and H the incident wave height.

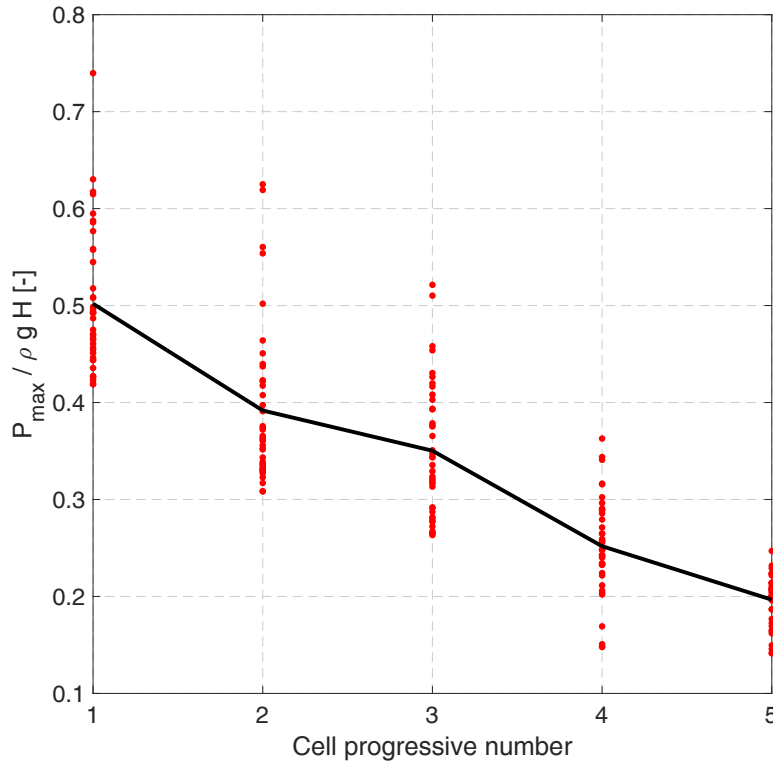


Figure 6.10: Normalized maximum pressures, $P_{max}/(\rho g H)$, across the internal cells. The x-axis reports the progressive cell number from the seaward side (Cell 1) to the landward side (Cell 5). Red markers indicate the dimensionless peak pressures extracted for each sensor and test. The black line represents the mean value for each cell.

The spatial evolution of the normalized maximum pressures across the caisson was investigated by considering the progressive number of the internal cells, from the seaward side (Cell 1) toward the harbor side (Cell 5). For each test condition, the dimensionless peak pressures recorded at all sensors within the same cell are reported as individual realizations. Figure 6.10 presents a scatter plot in which the red markers represent the normalized maximum pressures measured during each test, while the black line denotes

the average value computed for each cell. Despite the considerable scatter inherent to impacts, a clear decreasing trend of the pressure magnitude can be observed moving from the seaward side of the caisson toward the port side.

In terms of absolute values, the measured pressures, expressed in meters of water column, are, on average, comparable to approximately half of the incident wave height for the first internal wall, progressively decreasing to about 20% of the incident wave height for the last cell. This trend highlights the progressive dissipation of wave energy as the overtopping flow propagates across the caisson top and interacts sequentially with the internal walls. Note that the crest freeboard elevation was kept constant throughout the experimental campaign and set equal to $R_c = 0.035$ m.

6.5 Key insights of this chapter

This chapter presented an exploratory investigation of wave-induced pressures acting on the internal diaphragms of a caisson breakwater during the construction phase, i.e., in the temporary configuration without the cast-in-situ superstructure and with open compartments. Based on combined 2D physical model tests and CFD simulations, the results show that the interaction between overtopping waves and the internal cells evolves through distinct loading regimes governed by the filling level of the compartments. In the initial transient stage, when cells are partially filled, overtopping jets generate impulsive responses on the internal walls. As the cells progressively fill up to the crest level, the system reaches a stationary condition in which the impacts become quasi-impulsive and the pressure time histories are smoother and more repeatable.

The analysis of pressure extremes reveals a clear spatial trend across the caisson: the dimensionless pressure indicator $P_{\max}/(\rho g H)$ decreases systematically from the seaward cell (Cell 1) toward the most landward cell (Cell 5). Despite the considerable scatter typical of impact-driven processes, the average behavior indicates a progressive dissipation of overtopping-flow along the caisson. In terms of order of magnitude, the measured pressures expressed in meters of water column are, on average, of the order of $\sim 0.5H$ at the first internal diaphragm and decrease to about $\sim 0.2H$ at the last cell, under the tested regular-wave conditions and for a fixed crest freeboard ($R_c = 0.035$ m).

The numerical model was able to reproduce the main features of the experimental pressure time series and the spatial evolution of peak pressures when focusing on the stationary part of the phenomenon. Conversely, the early transient stage may involve highly impulsive dynamics influenced by air entrapment and compressibility effects, which are not fully captured by the adopted incompressible two-phase formulation. Accordingly, the results should be interpreted as an exploratory contribution aimed at improving the physical understanding of wave-induced loads on internal caisson cells, rather than as design-oriented outcomes, and further investigations will be required to extend these findings to realistic construction scenarios.

Conclusions

Overview and scope of the thesis

This thesis investigated the engineering optimization and adaptation of vertical breakwaters in relatively deep-water conditions, with a specific focus on composite caisson breakwaters equipped with either flushed or retreated wave walls. Through a sequence of complementary laboratory campaigns, ranging from broad parametric small-scale testing to more detailed mid/large-scale experiments and, in one case, coupled physical/numerical analyses, the work aimed to improve the physical understanding of wave-structure interaction processes that govern overtopping, reflection, and wave-induced loads, including post-overtopping impact phenomena and construction-phase vulnerabilities. Chapters 3-6 address RQ1-RQ4 through, respectively, (i) a broad parametric small-scale campaign on retreat effects, (ii) targeted experiments on post-overtopping event dynamics and impacts on retreated walls, (iii) irregular-wave testing focused on random seeding variability, and (iv) coupled physical/CFD analyses of construction-phase loads on internal caisson cells. Although specific conclusions are provided at the end of each individual chapter, the present chapter revisits the main findings in a more concise and integrated form, synthesizing the overall outcomes of the research and explicitly linking them to the formulated research questions.

Integrated answers to the research questions

RQ1: *How does the retreat distance of the crown wave wall influence wave-induced forces, reflection, and overtopping performance of composite vertical breakwaters under varying geometrical and wave conditions?*

The retreat distance significantly alters the hydraulic behaviour of composite vertical breakwaters and fundamentally modifies wave-structure in-

teraction mechanisms. Overall, retreated walls tend to increase impulsive loads on the wall itself, while generally reducing trunk and global forces, except in some unexpected configurations when the wall is located too close to the seaward edge. Wave reflection also decreases for sufficiently large retreats. At the same time, overtopping discharges increase with respect to the flushed configuration, particularly under “threshold conditions”, where little or no overtopping would occur for a flushed wall. These results highlight the sometimes antithetical effects induced by wall retreat, which are largely governed by post-overtopping flow processes. While retreated configurations may offer advantages in terms of selected load components and wave reflection, they can also introduce severe impulsive impacts on the wall and increased overtopping, requiring careful, process-based evaluation and, where possible, dedicated modelling. In this sense, the findings provide the answer to *RQ1*, showing that the retreat can be beneficial only within a trade-off space where overtopping performance and impulsive wall impacts are explicitly accounted for.

RQ2: *How do different types of post-overtopping flows interact with retreated crown wave walls, and which hydrodynamic features govern the magnitude, timing, and impulsive nature of the resulting loads on the wall and on the superstructure?*

Post-overtopping flows on composite vertical breakwaters can be categorized into distinct event types, dam-break (DB), plunging-dam-break (PDB), and hammer-fist (HF), building on classifications developed for green-water events on ship decks (Greco et al., 2005, 2007). The experiments confirmed that these regimes also occur on breakwater superstructures and provided criteria to predict their onset based on wave and structural parameters. Post-overtopping loads were found to depend strongly on both flow regime and wall position. The most critical scenarios were consistently associated with PDB and HF events combined with small retreats, where impulsive forces on the wall and trunk may act in phase, leading to severe global loading conditions. The combined use of force measurements and image analysis supported a process-level interpretation and enabled the definition of synthetic indicators for air content and impact loads, culminating in a parametric map, useful for design purposes, inspired by the PROVERBS framework (Oumeraci et al., 2001). These outcomes address *RQ2*, clarifying how event type and retreat position govern the impulsive nature and magnitude of post-overtopping impacts, and providing a first step toward design-oriented classification and estimation tools.

RQ3: *To what extent does random phase seeding in irregular-wave generation affect the estimation of mean and extreme overtopping indicators and wave-induced loads on a vertical breakwater, and what are the implications for the robustness of experimental/design metrics?*

Random phase seeding was shown to significantly affect overtopping estimates under irregular sea states, especially when only a limited number of realizations is available. Variability depends strongly on the relative free-board, remaining low under frequent-overtopping conditions and increasing markedly when overtopping is controlled by few waves. Wave-induced loads were found to be less sensitive to random seeding in comparison: global horizontal forces and synchronous front-face pressures generally exhibit limited variability, while post-overtopping downfall pressures and associated vertical forces display higher scatter. Overall, these findings answer *RQ3* and indicate that robust overtopping assessments require explicit consideration of stochastic uncertainty, motivating the use of multiple realizations and a cautious interpretation of extremes when sampling is limited. It is worth noting that the analysis of random phase seeding presented here refers exclusively to the flushed-wall configuration. The extension of this assessment to retreated-wall layouts is identified as a relevant topic for future investigation.

RQ4: *What hydrodynamic loads are induced by post-overtopping flows on internal caisson cells left temporarily uncovered during construction phase, and how do these loads evolve across different filling regimes and along the caisson?*

Open internal compartments of caisson breakwaters during construction can experience distinct loading regimes governed by the progressive filling level of the cells. The results indicate impulsive responses during the initial transient stage and quasi-impulsive, more repeatable loads under stationary conditions. A clear spatial attenuation of pressure peaks was observed along the caisson, with peak pressures decreasing from the seaward to the landward cells, reflecting progressive dissipation of overtopping-flow energy. The CFD-physical modelling reproduced the main features of the stationary loading regime. These findings address *RQ4*, improving the physical understanding of construction-phase vulnerability and supporting future developments toward design-oriented guidance.

Future developments

Future research can build upon the advancements made by the findings of this thesis. Specifically:

- Extend the assessment of hydraulic performance (loads, reflection and overtopping) of vertical breakwaters with retreated wave walls to **irregular-wave conditions**, including a systematic comparison with flushed-wall configurations.
- Expand the analysis of retreated-wall configurations through **numerical modelling** (e.g., CFD and/or hybrid approaches), using solvers capable of accounting for air compressibility effects, with dedicated validation datasets and sensitivity analyses for highly impulsive, air-influenced impacts.
- Investigate the **effect of random seeding** also for **retreated-wall breakwaters**, quantifying how seed variability propagates to overtopping, wall impacts, downfall pressures, and global force combinations.
- Develop **probabilistic/statistical characterizations** of seed-number effects on key response variables (means, extremes, tails), consistently across physical and numerical models, to support uncertainty-aware interpretation and design margins.
- Study **individual overtopping volumes and extreme events** on vertical breakwaters, linking them to impact regimes and enabling first-order **risk-oriented assessments** (e.g., exceedance probabilities of critical thresholds).
- Deepen the investigation of **construction-phase vulnerabilities** of caisson internal cells, extending toward **irregular waves** and **3D effects**, including oblique wave attack, which may substantially modify internal hydrodynamics and pressure distributions.
- Broaden the explored parameter space (geometry, water depth, foundation/berm configurations) and refine classification/mapping tools for post-overtopping events to improve transferability toward practical applications.

Expand the analysis of retreated-wall configurations through numerical modelling (e.g. CFD and/or hybrid approaches), using solvers capable of accounting for air compressibility effects, with dedicated validation datasets and sensitivity analyses for highly impulsive, air-influenced impacts.

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