Collapse processes and associated loading of square light-frame timber structures due to bore-type waves

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Abstract
Extreme hydrodynamic events such as hurricanes or tsunamis threaten coastal regions in particular. Such hazards must be assessed and appropriately incorporated into building codes to mitigate casualties and damages to coastal structures. Guidelines are often developed through experimental investigations that assume buildings remain rigid during hydrodynamic loading. To challenge this ‘rigid building paradigm’, test specimens were designed to replicate the deformation characteristics of an idealized light-frame timber structure using Froude-Cauchy similarity. Subsequently, a large-scale experimental study was conducted at the Large Wave Flume of the Coastal Research Center in Hannover. Hydrodynamic loads and load gradients were investigated to describe both the influence of an elasto-plastically modeled test specimen compared to a rigid reference model and the effect of load history on the structural loads. Finally, the collapse sequences of elasto-plastic specimens were extracted from high-speed photographs and classified into three failure mechanisms. In this study, data analyses are presented with the intention to not only inform local authorities for future development of guidelines but also serve as calibration and validation data for improving numerical methods.

1 Introduction
The performance of buildings and civil infrastructure when exposed to environmental extremes (e.g. storm surge, flooding, wind, earthquake) and associated loading is a focal point of civil engineering design that may directly affect safety, hazard and evacuation assessments as well as disaster mitigation strategies for community resilience. Buildings may experience extreme external loads when exposed to storm surges with high wind waves (Robertson et al., 2007; Kennedy et al., 2011;...
This work focusses on the dynamics of extreme hydrodynamic events interacting with buildings. While previous works have already addressed loading conditions of indestructible buildings (primarily focused on maximum loading), very little has been done to explore the dynamics underway when collapse processes are considered. This perspective yet overlooks the fact that in many cases, as will be seen in the literature review, over-loading leads to failure, partial or full collapse of buildings. In turn, considerable amounts of debris being generated during these collapse processes is then transported downstream, increasing loading on neighboring buildings through impact (Naito et al., 2014; Stolle et al., 2019, 2020) or damming (Stolle, Takabatake, et al., 2018; Shekhar et al., 2020). This work hence has the overall objective to foster the scientific work on and lay a basis for future predictive skills, to enable accurate modelling of water-debris mixtures evolving where extreme hydrodynamic events take place.

Extreme hydrodynamic events are coastal as well as fluvial hazardous conditions characterized by fast, turbulent, debris-laden and transient flows. Buildings subjected to extreme hydrodynamic events experience immense loading through flow acting on building envelopes (Robertson et al., 2007, 2008; Chock et al., 2013). To add a layer of complexity, there is internal loading once water has broken into them. Buildings respond to those loads by deflecting, first elastically, but then plastically leading to progressive damage and ultimately collapse as well as dislodgement (Hatzikyriakou et al., 2016; Tomiczek et al., 2017; Duncan et al., 2021). The complexity of interactions between the anisotropic elasto-plasticity of building structures and hydrodynamics of extreme events has only recently gained attention in the scientific community (Lindt et al., 2009; Wilson et al., 2009; Duncan et al., 2021). The specific challenge lies within the combined modelling of the elasto-plastic properties and collapse processes of buildings while interacting with hazardous extreme hydrodynamic events. Effects of storm surge and tsunami on buildings (damage, collapse, dislodgement, scouring) are impressively shown in Figure 1; and it becomes vividly clear that the mixture of all sorts of solids and liquid is a characteristic that makes this research significant.
Figure 1. Effects of extreme hydrodynamic events: (a) Seaside Heights houses floated away [M. Tama/Getty Images] (b) Union beach house damaged, both New Jersey, USA after hurricane Sandy, October, 2012 [R. Sullivan, Woods Hole]. (c) Sendai plains with tsunami wave and entrained buildings [NHK World, Japan], (d) Kamaishi city, with a building afloat [unknown source], both in Japan on March 11, 2011 during the Tōhoku Earthquake and Tsunami (2011 TETaT).

Past research has most often focused on the water phase alone; this research in contrast assesses the total performance of a single building, including the collapse phase and evaluate the concurrent inundation extent as it is affected by the collapsing building. It also describes differences in the collapse processes and contrast them by their total loading that has led to the collapse. Prior to the presentation of the scientific results, the authors have opted to provide a more in-depth review of available literature on the subject of collapsing buildings in extreme hydrodynamic conditions, looking into neighboring fields where structural collapse has been addressed more widely by the communities.

2 Literature review

2.1 Extreme hydrodynamic events leading to building collapse

Ericson et al. (2006) found that combined effective sea-level rise and storm surges will have a considerable effect on millions of coastal inhabitants. Recent examples of devastating, deadly storm surges include typhoon Haiyan in the Philippines with local extreme water levels of 5-6 m (Mori et al., 2014) and in Bangladesh (Karim and Mimura, 2008). In the 2070s, major cities with important port facilities will be threatened by the combined effect of climate change, exposure to storm surges and subsidence, growing to about 150 million people at risk (Hanson et al., 2011). Induced by global
warming, changes of storm climatology in combination with sea level rise will, for the example of New York City, result in 100-yr flooding to occur every 3-20 years, and for the 500-yr flooding to occur every 25-240 years (Lin et al., 2012). Storm surges are frequently accompanied by strong waves (Ranasinghe, 2016), which – when impacting coastal development- cause severe damage while propagating further inland as waters rise. Recently, Roeber and Bricker (2015) found the generation of bore-type waves during Typhoon Haiyan washing away shed-like buildings as a result of enhanced energy in the infra-gravity frequency band (surf beat) leading to tsunami-like overland flow. Further, increasing wave heights of wind waves are predicted as a result of climate change (Chini et al., 2010; Mori et al., 2010) potentially causing more destructive hydrodynamic events in future.

Secondly, tsunamis are an arguably destructive geohazard occurring along the world’s coastlines. Recent events such as the 2004 Indian Ocean Tsunami and the 2011 TEaT have disastrously illustrated the destructive potential affecting large stretches of coastal land. According to Løvholt et al. (2012), about 20 million coastal residents are living in tsunami-prone regions. The 2011 TEaT led to the loss of life and buildings (190,000 damaged, 45,000 destroyed) and infrastructure damage (Norio et al., 2011), despite the level of preparation Japan had undertaken to prepare for such event. Tsunami waves, when propagating over shallow terrain, are assumed to be “dam break-like”. They exhibit a sharp and steep front followed by a quasi-steady tail (Chanson, 2006) with fast flow velocities and strong turbulence. On land, within the build environment, tsunami may have flow depth similar to building heights (1-, 2-, 3-storeys, Figure 1 (d)), and velocities between 3 -11 m/s, e.g. as observed in Kesennuma city, Japan (Fritz et al., 2012).

Thirdly, it is very likely that sub-daily extreme precipitation events will result in more severe flash floods (Westra et al., 2014). Flash floods are characterized by their short duration from onset to peak flow and their often steep front wave (Georgakakos, 1986) evolving when propagating along a river network. Flash floods interact with buildings near river courses when inundating river banks, and severe damage has been seen all over the globe as its result (~9% of insured losses, 1980-2002). A recent example of a large-scale flash flood occurred in the western part of Germany, in the Ahr valley (Fekete and Sandholz, 2021). Flash flood-like extreme hydrodynamic events could also evolve through breaching of aging dam or dike infrastructure, with significant numbers of events throughout the past decades (Gallegos et al., 2009; Pilotti et al., 2011), all of which can also be approximated by dam-break waves.

In summary, storm surges, tsunamis and flash floods owe the potential to destruct buildings, either by exerting excess loading on the building’s structural elements, or by undermining foundation components. Extreme hydrodynamic events are capable of severely affecting human settlements. It will be important to understand the processes leading to, during and after the collapse of individual builds in order to build tools to predict the overall flooding-destruction sequence eventually.

2.2 Building performance in light of extreme hydrodynamic events

Hydraulic loading exerted by extreme hydrodynamic events on buildings comprises impulsive forces (short-duration, transient) induced by wave breaking, hydrodynamic forces (quasi-static, strong turbulence) (Al-Faesly et al., 2012; Robertson et al., 2013; Krautwald et al., 2022), debris impact forces (short-duration, transient, very strong) (Nistor et al., 2017; Derschum et al., 2018; Stolle, Derschum, et al., 2018), debris damming forces (quasi-static once established, unknown probability, evolution process) (Stolle, Takabatake, et al., 2018; Shekhar et al., 2020), as well as buoyancy forces (Yeh et al., 2014), depending on building envelop state (Yeh et al., 2015). A forensic survey after
Hurricane Ike making landfall in Texas, US, found storm surge and wave related building damage, dislodgement, and entire collapse correlated with relative vertical distance between lower floor elevation to maximum surge level on structures (Robertson et al., 2008; Kennedy et al., 2011). The damage regime ranged from buoyancy and uplift forces dislodging homes to damaging, impulsive breaking wave forces associated with high waves propagating inland. Causes and effects of building damage could not however be analyzed with strong confidence, and the progressive collapse remains a mystery.

Charvet et al. (2014) used the extensive damage database of the Japanese Ministry of Land, Infrastructure, Transport and Tourism (MLIT, 2012) containing 178,448 individual buildings affected by the 2011 TEaT to improve empirical fragility assessment. A key finding was that besides tsunami flow depth, damaged building’s construction type, e.g., wood-/steel-frame, reinforced concrete (RC) was crucial for estimating damage states statistically. However, without process-based knowledge, identifying specific reasons for collapse is inherently difficult (see Figure 1 (c)), and empirical fragility functions are often tied to specific events. The use of satellite data prior and after extreme hydrodynamic events striking hazard-prone areas in conjunction with numerical simulations of the flow dynamics were useful resource for developing fragility functions, particularly when large areas as after the 2004 Indian Ocean Tsunami are affected (Suppasri et al., 2011). A key problem with empirical fragility functions is the intricate interplay of location, materials, flow complexity and building shape – which is difficult to replicate post-mortem.

Aside from empirical data, analytical fragility functions can be developed through the use of advanced structural modelling (McKenna, 2011). Petrone et al. (2017) displayed a method to apply tsunami loading on a RC building approximating loading from the 2011 TEaT. However, the set of fragility functions for this RC building did not reflect the full hydrodynamic breadth of load variations (e.g., no buoyancy, no debris impact) and its transient nature was approximated by triangular and trapezoidal force distributions. Flow through, and potential debris damming from deteriorated other buildings closer to the shoreline, was equally neglected (Petrone et al., 2017).

Without a break-through in understanding and more accurate representation of extreme hydrodynamic events when modelling building performance, progress on predicting progressive collapse of structures, along with adequate experimental facilities and numerical simulation tools remains very challenging.

### 2.3 Performance of individual buildings and building parts

Next, state-of-the-art knowledge pertaining to current building modelling methods and shortcomings are reviewed. The predominant portion of research that has addressed extreme hydrodynamic events acting on buildings has used rigid models, i.e., almost no elasticity or flexure under loading. This modelling approach can be termed the “rigid building paradigm”. Often, square or round, rigidly-mounted structures are used and subjected to extreme hydrodynamic events in small- or medium-scale dam-break flumes to investigate pressure and force characteristics (Yeh, 2006; Nouri et al., 2010). Large dynamic forces due to the violent interaction of the wave front reaching the structure or debris impact were observed that exceeded those forces prescribed by current design standards (ASCE 7-16, 2017). Past analysis confirmed that dam-break generated hydraulic bores resembled appropriately prototype tsunami conditions (Al-Faesly et al., 2012). Generally good agreement was found for tsunami-type loading on rigid building specimen when simulating the building performance with numerical models, although high computational effort and tuning was necessary (Tomita et al., 2007; Park et al., 2013; St-Germain et al., 2013). Some attempts have more recently been made in relation with debris impact onto vertical structures to account for structural flexibility/elasticity in...
building and debris stiffness known, and have successfully shown that debris impact forces can be predicted accurately, using data of a mid-scale experimental campaign for shipping container drift (Stolle et al., 2019).

Though rarely studied, one investigation found considerable effects of openings in buildings on total horizontal forces and overturning moments (Wüthrich et al., 2018). A small-scale study found that vertical forces strongly depend on the amount of opening in the model building. It yet remained unclear how dead weight of the additional water and buoyancy through trapped air inside may have affected the overall vertical forces. In pursuit of tsunami-resilient design, Thusyanthan and Madabhushi (2008) have modelled small-scale Sri Lankan buildings, for the first time resorting from simplified, box-type structures. These authors have used geometric scaling and unscaled materials (i.a., with prototype yield strength) to optimize architecture; however material properties and joint behavior were not considered and those buildings were still “rigid”. These authors also mention splash-up that hit the roof structure from inside during the tsunami action on the building. Wilson et al. (2009) tested a structurally compliant 1/6 scale wooden building subjected to wave loading and found that the building experienced about 60% lower forces in those cases where openings allowed the flow to pass through the building. It was also noted that specific forces changed drastically by modifying constructional details; for example, buoyancy forces (uplift) were dominant in cases where open crawl space vs. a slab/stem wall foundation was compared. It is noteworthy that unanticipated forces were detected that are ostensibly only architectural features, e.g., overhanging eaves above garages.

Realistic buildings with their intricate forms, shapes and architectural features are hence very likely to either withstand larger or fail at smaller total loading, as compared to those loads found in the above-mentioned studies. These loadings however often find their way into recommendations and standards, despite their original scope and validity. These uncertainties with respect to ultimate limit states of realistic building shapes necessitate further research, particularly looking into realistic loading that includes failure and collapse of buildings more specifically.

2.4 Towards damage and collapse of buildings exposed to extreme hydrodynamic events

Literature evidence of damage, collapse and dislodgement of buildings is only available through forensic, post-hazard surveys, yet no single reference could be found that documented progressive collapse as it happens. This is a major, fundamental shortcoming in the current knowledge, indicating a general lack of understanding at a process level. A survey after the 2011 TEaT showed previously unaddressed, potential failure causes of buildings: debris impact, severe scouring around buildings and liquefaction (Fraser et al., 2013). For timber buildings in Kamaishi, Japan, these authors note that soft-storey failure (lower floor collapse with roof and top storey remaining intact) may have happened due to tsunami action. Sheltering vs. exposure in building clusters played an important role in layouts where buildings did not collapse unexpectedly, for unknown reasons. The critical role, entrapped air played in the overturning of RC buildings, load reducing usage of break-away walls and the sheltering aspect of sturdy buildings protecting less well-built structures were noted by Yeh et al. (2013). Thus, raising the need to investigate failure modes and ultimate limit states of buildings exposed to extreme hydrodynamic events. These authors called attention to the importance of considering soil softening, scouring and erosion near buildings, an aspect overlooked in current research although these exposed foundations may lead to failure of building as a whole. Fraser et al. (2013) underpins this observation: buildings in Ofunato city whose windows were found unbroken,
and water marks inside the building indicated that air was trapped, potentially causing additional buoyancy forces resulting in up-lift.

Besides post-hazard observation, experimental or numerical insight into building collapse is very sporadic, and the entire challenge lacks sincere analysis, inappropriate to the global scale by which extreme hydrodynamic events threaten human dwellings. A full-scale study by Yeh et al. (1999) looks at wooden breakaway wall subjected to hurricane wave action; this study attempts to understand nail configurations and eventual failure due to the most damaging wave type, a broken storm wave attacking the wall as an approaching bore. A 1/6 scaled wood-frame house was tested destructively by Lindt et al. (2009) up to failure; in that case, this showed as soft-storey failure of the building model. The results were correlated with static load testing such that a simplified force equation could be proposed.

Numerical collapse modeling has evolved into a tool that allows removing single/multiple structural elements (e.g., columns, beams) and predicting a final post-collapse state. Although depicting a step forward, such technique was never applied where buildings interact with extreme hydrodynamic events. A review of progressive collapse and robustness of building structures mentions the current lack of experimental observation to calibrate and validate collapse simulations of timber and masonry structures (Adam et al., 2018), even without extreme hydrodynamic events. Salem and Helmy (2014) analysed the cause of the Minnesota I-35 bridge with good accuracy which failed in 1967 during construction work by an Applied Element Method (AEM), capable of predicting mode and time of failure, forces as well as velocity of flying debris. A one-way coupled, iterative approach was applied to study the collapse of the Tsuyagawa bridge subjected to the 2011 TEaT; the AEM applied hydrodynamic forces from simplified drag and buoyancy relations based on inundation depths and flow velocities provided by Fu et al. (2013) without resolving the fluid dynamics explicitly (Salem et al., 2016).

Observations of damage, progressive collapse and dislodgement are challenging, given the powers that extreme hydrodynamic events hold. They are intrinsically difficult to study, as those waters are turbulent and clouded by suspended sediment, equally limiting observational access during the events. A preliminary review of the available eyewitness videos during the 2011 TEaT was conducted; this analysis lead to the following conclusion pertaining to failure modes and duration until failure:

- Pitched roof, grey wood-frame house in Kamaishi city (Video_1, 2011): Failure mode is dislodgement from foundation floating away with upper floor and roof intact, estimated time from onset of flow to failure about 40 s (the reader is also referred to Figure 1 (d) that depicts this scene)
- RC two-storey office building in Hachinohe port (Video_2, 2011): Failure mode is collapse of breakaway walls, with subsequent dislodgement of whole pitched roof, estimated duration of failure during second incoming tsunami wave is 12 s, the damaging bore propagating at a local water depth of about 1.5 m
- Cluster of one- and two-storey buildings in Miyako city (Video_3, 2011): Failure mode potentially push-off of foundation, or soft-storey failure, roofs propagation inland after collapse initiation, impact of floating roofs/upper building portions induce next-line failure, duration of failure 10 s

2.5 Urgent needs in the state-of-the-art and objectives
On the basis of this review, the following urgent needs are identified. Extreme hydrodynamic events, namely tsunami, storm surges with large waves, and dam break-like flash floods are calling for better preparation, and improved predictions prior to disasters. (1) Nowadays, warning and evacuation measures must rely on accurately forecasting site-specific hazard and its implications. Yet, we are currently unable to predict progressively collapsing buildings, one of the most uncontested problems in the engineering sciences to date. (2) Existing forecast tools represent buildings in the pathway of extreme hydrodynamic events as rigid, stationary obstacles (they do not move at all). This choice affects the momentum balance of realistic flows and causes inaccurate predictions where buildings would have collapsed, and debris would have been added downstream. (3) to the authors’ knowledge, the “rigid building paradigm” persists and has not been tackled before. One reason is our inability to control hydro-structural processes at appropriate length scale experimentally (duration- or magnitude-limitations), while numerical models have not been validated with respect to the process complexity.

The remainder of this study is hence aiming to answer the following questions experimentally, in pursuit of stipulating further research into the building collapse due to extreme hydrodynamic events:

- How does the elasticity of a building specimen, either elasto-plastic or fully rigid, affect the total force time-history of a building, when exposed to an extreme hydrodynamic event?
- Is the collapse process of identically constructed model specimen comparable or are there significant differences in the underlying failure mechanisms?
- What influence has the preceding load history on the forces and load transfer in the building specimen?

3 Materials and methods

3.1 Experimental test facility

In October 2019, an experimental test series was carried out in the Large Wave Flume (GWK) operated by the Costal Research Center (FZK) in Hannover, Germany. The GWK has dimensions of 307 m in length, 5 m in width and 7 m in depth and allows for close-to-prototype experimental tests. A schematic sketch of the experimental model setup is given in Figure 2. The coordinate system is zero for the position of the wave maker (horizontal), the southern wall (lateral) and the flume bottom (vertical). A composite slope was constructed at \( x = 193 \text{ m} \) starting with a 1:15 slope up to a height of \( z = 3.6 \text{ m} \) at \( x = 247 \text{ m} \) and a subsequent horizontal platform. To maintain a consistent bathymetry, the slope consisted of a sand core, geotextile layer and concrete slabs. The specimens were positioned in a distance of 8.0 m from the transition of the slope to the platform.
Figure 2. Experimental model setup in side view (a) and top view (b) including the position of all measurement devices. Two positions are shown in the top view (b) that are referred to in Figure 5. Following abbreviations are used consistently to Table 1: WG – wave gauge; ICM – inductive current meter; US – ultrasonic distance sensor; HS-Cam: High-speed-camera.

3.2 Instrumentation

Since the study of large-scale, non-rigid collapsing structures is an experimental novelty, a variety of different instrumentation were utilized to study both hydrodynamics and structural response. To acquire the water surface elevation during propagation over the flume bottom, two sets of wave gauge arrays (WG 1.1-1.4 and WG 2.1-2.4) were installed, each consisting of four wire-resistance wave gauges. When the wave arrived at the platform, six ultrasonic distance sensors (US 1-6) recorded the flow depth above the platform. Flow velocities were measured using inductive current meters (ICM 1-4) placed at a height of $z = 3.7 \text{ m}$, i.e. 10 cm above the bottom, and distributed along the horizontal platform. However, the inductive current meters must be submerged for reliable measurements and encountered problems with initially high flow velocities and entrained air during the period when the turbulent bore inundates the platform. Therefore, this data misses the initial high flow velocities and should be used for validation purposes only as has been described in von Häfen et al. (2022).

Four pressure transducers were installed in the centerline of the front panel (PS 1-4). These pressure sensors were mounted to the rigid structure only, since a structural collapse would potentially cause damage to these sensors. However, a multi-axis sensor was installed underneath the structures to acquire total forces and moments in all spatial directions. Four inertial measurement units (IMU) were installed on the timber studs supporting the front panels (IMU 1-4). Each inertial measurement unit recorded three-dimensional (3D) acceleration, turn rate as well as magnetic field intensity. Two high-speed cameras were mounted at a height of $z = 11.6 \text{ m}$ with their field of view (FOV) directed towards the horizontal platform. High-speed camera images allow to investigate the structural failure process in detail, however, the reader is referred to von Häfen et al. (2022) for a discussion on a more detailed analysis of the hydrodynamics using large-scale PIV methods.
Sensor types, brands, accuracies, ranges as well as sampling frequencies are given in Table 1 for a comprehensive overview of the technical specifications. Most of the data was recorded and synchronized by a central data acquisition system except for the inertial measurement units and the high-speed cameras. Both measurement systems were started synchronously using a trigger signal that was recorded by the central data acquisition system. Hence, fully synchronized data sets of the entire instrumentation are available. All other data series and images were synchronized in post-processing by their time stamp.

### Table 1. Instrumentation list providing an overview over the used sensors and technical details.

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Measuring device</th>
<th>Manufacturer</th>
<th>Location of company</th>
<th>Sensor</th>
<th>Unit</th>
<th>Accuracy</th>
<th>Range</th>
<th>Frequency [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>WG (1.1-1.4 &amp; 2.1-2.4)</td>
<td>Capacitance-type wave gauge</td>
<td>Coastal Research Center</td>
<td>Hannover, Germany</td>
<td>[no commercial product]</td>
<td>[m]</td>
<td>± 10 mm</td>
<td>± 2.25 m</td>
<td>100</td>
</tr>
<tr>
<td>US (1-4)</td>
<td>Ultrasonic distance sensor</td>
<td>Microsonic GmbH</td>
<td>Dortmund, Germany</td>
<td>mic+340/1U/TC</td>
<td>[m]</td>
<td>± 1 %</td>
<td>0.35 – 5.0 m</td>
<td>500</td>
</tr>
<tr>
<td>US (5-6)</td>
<td>Ultrasonic distance sensor</td>
<td>Microsonic GmbH</td>
<td>Dortmund, Germany</td>
<td>mic+130/1U/TC</td>
<td>[m]</td>
<td>± 1 %</td>
<td>0.20 – 2.0 m</td>
<td>500</td>
</tr>
<tr>
<td>ICM (1-4)</td>
<td>Inductive current meter</td>
<td>hs-engineers</td>
<td>Lichtenhagen, Germany</td>
<td>ISM-2001f</td>
<td>[m/s]</td>
<td>± 1 %</td>
<td>± 3 m/s</td>
<td>100</td>
</tr>
<tr>
<td>PS (1-4)</td>
<td>Pressure sensor</td>
<td>RS Hydro</td>
<td>Stoke Prior, United Kingdom</td>
<td>PDCR1830</td>
<td>[Pa]</td>
<td>± 0.1 %</td>
<td>± 35 kPa</td>
<td>1000</td>
</tr>
<tr>
<td>-</td>
<td>Multi-axis sensor</td>
<td>Interfaceforce e.K.</td>
<td>Tegernsee, Germany</td>
<td>IF6A175</td>
<td>[N]</td>
<td>± 0.5 %</td>
<td>± 20 kN ± 5 kNm</td>
<td>1000</td>
</tr>
<tr>
<td>HS (1-2)</td>
<td>High-speed camera</td>
<td>Basler</td>
<td>Ahrensburg, Germany</td>
<td>acA1300-200um</td>
<td>[-]</td>
<td>~ 6.6 mm</td>
<td>~ 14.0 m</td>
<td>100</td>
</tr>
<tr>
<td>IMU (1-4)</td>
<td>Inertia Measurement Unit</td>
<td>Inertia Technology</td>
<td>Enschede, The Netherlands</td>
<td>ProMove mini</td>
<td>[m/s²]</td>
<td>± 0.5 %</td>
<td>± 157 m/s² ± 2000 °/s</td>
<td>100</td>
</tr>
</tbody>
</table>

### 3.3 Model design of the elasto-plastic specimen

The fundamental objective of designing the elasto-plastic specimen was to demonstrate the feasibility of scaling selected structural properties (like stiffness) prior to investigating their effects when subjected to extreme hydrodynamic events. Despite having an influence on applied forces and collapse mechanisms, real architectural components such as doors, windows, overhangs were however neglected in here, as to limit this investigation on testing the rigid building paradigm by comparing elasto-plastic and fully rigid structures.

A simplified prototype building constructed in a wooden light-frame construction was used as reference for the model design. It is conceived with a square cross-section (5.0 m × 5.0 m) and a flat roof with a height of 3.5 m. The construction is representative of the common building standard of timber dwelling structures in Central Europe. Each side of the building had an external wall sheathing with four OSB (oriented strand board) panels measuring 1.25 m × 3.5 m × 0.012 m. The supporting wooden studs were assumed to have a cross section of 0.06 m × 0.18 m and a spacing of 1.25 m. Wall sheathing and supporting wooden studs are connected with steel nails with a diameter of 2.1 mm and a length of 50 mm at 0.10 m spacing. Using these specified dimensions, a scaling of the model specimen was performed to obtain scaled deformations in the experiment based on the geometric scaling factor. A systematic sketch of the scaled model specimen is provided in Figure 3 highlighting some construction details.
Figure 3. Systematic sketch of the elasto-plastic specimen where the upper interface of the wall sheathing and the roof joists are detailed in (a), the interface of wall sheathing and upper joists is detailed in (b) and the lower corner studs are depicted in (c).

To limit scale effects and achieve a feasible model design with a simultaneous consideration of both Froude and Cauchy similitude, the scale factor was chosen to be as large as possible (Heller, 2011). Therefore, a geometric scaling factor of 1:5 ($\lambda = 5$) was chosen as a compromise between a large scale and a reasonable blockage ratio (1/5) of the 5.0 m wide wave flume. However, scaling the structural stiffness is a more complex issue, depending on parameters such as load attack, material properties and the dimensions of nails, wooden studs and the shear walls. A full Cauchy model scaling based on the second moment of inertia and Young’s modulus usually results in either the need to select materials with different properties than wood or the required cross sections are not technically feasible. Hence, in a first step, the predominant load transfer processes for the individual structural components were identified and, based on experimental tests and scaling considerations, the required timber dimensions were determined.

Initially, the hydrodynamic bore exerts forces on the wave-directed wall. In this case, the wall sheathing transmits the entire forces to the vertical wooden studs, which subsequently deform and transfer the load to the roof and slab construction. Hence, the bending stiffness of the studs determine the maximum deformation and require a proper scaling. Following scaling considerations based on the Cauchy number are made for a proportionality of the bending stiffness between prototype and model (Chakrabarti, 2005):

$$ (EI)_p = \lambda^5 \cdot (EI)_m $$

(1)
where $E$ is Young’s modulus and $I$ is the second moment of inertia. Both the prototype as well as the elasto-plastic model are supposed to be constructed from the same wood species (pine wood), which consequently have the same elasticities and Young’s modulus. Therefore, Young’s modulus is the same for prototype and model scale and Equation (1) transforms into the following equation:

$$I_p = \lambda^5 \cdot I_m$$  \hspace{1cm} (2)

At prototype scale, the second moment of inertia, determined from the reference building described above, is equal to $I_p = 2.916 \cdot 10^7 \text{mm}^4$ and thus the dimensions of wooden studs at model scale are determined to be $b_{m, wall} = 14 \text{mm}; h_{m, wall} = 20 \text{mm}$ based on available materials and according to Equation (2).

However, it is assumed that the deformations of the lateral walls result mainly from the stiffness of the connection between the wooden studs and the sheathing when they are subjected to in-plane horizontal stress. To obtain geometrically scaled deformations, the following condition has to be fulfilled:

$$u_p = \lambda \cdot u_m$$  \hspace{1cm} (3)

where $u_p$ is the displacement in prototype and $u_m$ is the displacement at model scale. Considering a hydraulic Froude scale model, the applied forces at model scale are related to prototype scale by the scaling factor to the power of three (Chakrabarti, 2005):

$$F_p = \lambda^3 \cdot F_m$$  \hspace{1cm} (4)

where $F_p$ is the force at prototype scale and $F_m$ is the force at model scale. Bearing these scaling considerations in mind, the horizontal displacement ($u$) of the upper joist can be calculated based on Castigliano’s theorem (Castigliano, 1879) with the following equation (Kessel, 2002):

$$u = 2 \cdot \frac{a_v}{K_{ser}} \cdot \frac{h}{l} \cdot \left( \frac{n_h}{h} + \frac{n_l}{l} \right) \cdot F$$  \hspace{1cm} (5)

where $a_v$ is the fastener spacing, $K_{ser}$ is the slip modulus of the fasteners connecting the wall sheathing and the wooden studs, $h$ and $l$ are the total height and width of the wall, respectively and $n_h$ and $n_l$ are the number of sheathing panels over the wall height/length.

Next, Equation (4) and Equation (5) are inserted into Equation (3), yielding the following equation:

$$2 \cdot \frac{a_{v,p}}{K_{ser,p}} \cdot \frac{h_p}{l_p} \cdot \left( \frac{n_h}{h_p} + \frac{n_l}{l_p} \right) \cdot F_p = \lambda \cdot 2 \cdot \frac{a_{v,m}}{K_{ser,m}} \cdot \frac{\lambda \cdot h_p}{\lambda \cdot l_p} \cdot \left( \frac{\lambda \cdot n_h}{h_p} + \frac{\lambda \cdot n_l}{l_p} \right) \cdot \frac{F_p}{\lambda^3}$$  \hspace{1cm} (6)

For this particular model, the number of sheathing panels over the wall height equals to one and the number of sheathing panels over the wall length equals to four ($n_h = 1; n_l = 4$). Using this
information and rearranging Equation (6) gives the following equation to determine the fastener spacing based on the scaling factor and the slip modules:

\[ a_{v,m} = a_{v,p} \cdot \frac{K_{ser,m}}{K_{ser,p}} \cdot \lambda \]  

(7)

The slip modulus of the sheathing connection in prototype and model scale was determined in an experimental study at the Institute of Building Construction and Timber Structures at the TU Braunschweig. Based on available and selected nails and dimensions of structural timbers, the fastener spacing acts as parameter to “calibrate” the displacement of the model specimen proportionally to the displacement of the prototype. In the experimental tests, the mean slip modulus of a single fastener in the compound of wooden sheathing (12 mm thickness) and the wooden studs was determined to be \( K_{ser,p} = 700 \text{ N/mm} \) for the 2.1 mm steel nails in prototype scale. At model scale, brass nails with a diameter of 1.3 mm and a length of 13 mm were chosen and a mean slip modulus of \( K_{ser,m} = 86 \text{ N/mm} \) per fastener were determined experimentally. Available pine wood plates with a thickness of 3 mm were used as wall sheathing. With a prototype nail spacing of \( a_{v,p} = 100 \text{ mm} \), the required nail spacing at model scale equals to \( a_{v,m} = 61.4 \text{ mm} \) and was set to \( a_{v,m} \approx 60 \text{ mm} \) due to construction constraints.

In contrast to the longitudinal walls, it cannot be assumed that the roof deformations result exclusively from the flexibility of the connection between the horizontal wooden joists and the wall sheathing. In the structural compound of the roof panel, the load-bearing rim joist should transmit the forces to the panels via the fasteners. However, the risk exists that an unscaled high stiffness in the rim joist will result in a secondary load path that transfers loads through the end joists on the lateral walls. This highlights the need to also scale the stiffness of the rim joist. Requiring a width of at least \( b_{m,roof} = 10 \text{ mm} \) for adequate manufacturing quality, the height of the rim joist could be determined based on Equation (2) and the prototype dimensions to be \( h_{m,roof} = 12 \text{ mm} \).

Overall, the most relevant quasi-static load paths could be identified and appropriately scaled for studying the effect of loads on buildings with a scaled stiffness. Selected wood components with low defects were used (e.g. no bark inclusions, no cracks, no knots, etc., see Wei et al. (2011)), holes were pre-drilled and about 600 nails were used per structure. All these efforts were directed towards the goal of realizing and maintaining a high manufacturing quality for the test specimens. A total of nine test specimens with the same structural design were produced and tested in the Large Wave Flume.

### 3.4 Experimental program

At the time this experimental study was conducted, the Large Wave Flume at the Coastal Research Center was operated with a piston-type wave maker characterized by a maximum stroke of 4.0 m, velocities of the paddle up to 1.7 m/s and accelerations up to 2.1 m/s². These limitations were utilized to generate solitary waves in a constant water depth of \( d_0 = 3.4 \text{ m} \) and resulted in nominal wave heights between \( H_{nom} = 0.50 \text{ m} \) and \( H_{nom} = 1.10 \text{ m} \) at the wave maker. An enhanced overview of the parameters and shapes of solitary waves generated in the Large Wave Flume can be found in Schimmels et al. (2016), Sriram et al. (2016) and Schimmels et al. (2014).

In all tests, the shoaling over the slope causes the wave to break and thereafter transforms into a turbulent bore propagating over the adjacent horizontal platform. In experimental tests without any...
structure, overland flow depths of between \( h_{s,US4} = 0.17 - 0.35 \) m were measured at the position of the structure. von Häfen et al. (2022) have analyzed and illustrated the hydrodynamics of broken solitary waves in detail when travelling above a composite slope with an adjacent horizontal platform. For further information on the specific hydrodynamics of the undisturbed bore propagation, the interested reader is hence referred to Section 4.1 and von Häfen et al. (2022).

For the sake of completeness, Table 2 provides the sequence of experimental tests for each of the nine (9) model structures to illustrate the generated wave heights and loading conditions. Within each row, the first tests are those in which the structure was not preloaded, while the last tests are those in which the structure collapsed. This work uses specimen multiple times and subjects them to varying waves, in order to reflect that tsunami often exhibit multiple wave crests (Choowong et al., 2008), which can impact the built environment.

Table 2. Experimental test program with collapsing models. The matrix on the right shows the sequence of experimental tests for the different model structures and the respective nominal wave height.

<table>
<thead>
<tr>
<th>Structure</th>
<th>Failure mechanism (Section 4.3)</th>
<th>Water depth at the wave maker [m]</th>
<th>Sequence of experimental tests with respective wave height ( H_{nom} [m] )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>3.4</td>
<td>No preload</td>
</tr>
<tr>
<td>EP1</td>
<td>(ii)</td>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>EP2</td>
<td>(ii)</td>
<td></td>
<td>0.6</td>
</tr>
<tr>
<td>EP3</td>
<td>(i)</td>
<td></td>
<td>0.7</td>
</tr>
<tr>
<td>EP4</td>
<td>(iii)</td>
<td></td>
<td>0.9</td>
</tr>
<tr>
<td>EP5</td>
<td>(ii)</td>
<td></td>
<td>0.8</td>
</tr>
<tr>
<td>EP6</td>
<td>(ii)</td>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>EP7</td>
<td>(i)/(iii)</td>
<td></td>
<td>1.1</td>
</tr>
<tr>
<td>EP8</td>
<td>(ii)</td>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>EP9</td>
<td>(iii)</td>
<td></td>
<td>0.8</td>
</tr>
</tbody>
</table>

4 Results

4.1 Wave characteristics without structure

Three exemplary wave heights \( H_{nom} = 0.7 \) m, 0.9 m and 1.1 m were compared for a constant water depth of \( d_0 = 3.4 \) m. In these tests, no model specimen was installed on the platform allowing reference flow depths and velocities to be examined.

Figure 4 shows the time-history of the wave maker’s stroke and the wave heights at two positions in the offshore section. The hydraulic properties shown in here are referred to a zero time when the flow depth at US1 exceeds a threshold of 0.01 m. Stroke, velocity and acceleration of the piston type wave maker are set to generate the desired wave properties, a solitary wave based on Goring’s method (Goring, 1979; Schimmels et al., 2014). It is evident from the stroke time-histories (Figure 4 (a)) that higher waves are not only generated by an increasing maximum stroke but also due to a steeper gradient – hence a higher traversing speed. Waves are shown directly in front of the wave paddle at \( x = 3.6 \) m (Figure 4 (b)) and in a distance of \( x = 60 \) m (Figure 4 (c)). Comparing the signals at both positions, it can be inferred that the solitary waves deform slightly during propagation and a trailing wave train occurs. However, the trailing wave is considered to have negligible impact on the structural loads.
Figure 4. Time-histories of the stroke (a) and the wave propagation for two positions along the horizontal flume bottom (b) – (c) for three wave heights of $H_{\text{nom}} = 0.7$ m, 0.9 m and 1.1 m.

Next, the characteristics of the bore-like flow over the horizontal platform are examined. Characteristics considered in this study are, on the one hand, the measured flow depths and flow velocities and, on the other hand, the inferred parameters of the momentum flux and the Froude number ($Fr$). The momentum flux ($hu^2$), a commonly used parameter for estimating structural loading, is determined by the squared flow velocity multiplied by the flow depth (ASCE 7-16, 2017). Further, the Froude number ($Fr$) is given as ratio of inertia to gravity forces, which were observed in previously occurred tsunami to be between 0.6 and 2.0 (Iizuka and Matsutomi, 2000; Fritz et al., 2006, 2012). The Froude number is calculated as follows (Chakrabarti, 2005):

$$Fr = \frac{u}{\sqrt{gh}}$$

where $u$ is the flow velocity, $g$ is the acceleration of gravity and $h$ is the flow depth.

For two positions surrounding the later installed specimen ($x = 255$ m), these parameters are shown in Figure 5. Position 1 refers to US 2 and ICM 1 ($x = 249.0$ m), whereas position 2 refers to US 4 and ICM 2 at the wave-directed side of the specimen ($x = 254.5$ m) (cf. Figure 2). Expectedly, higher wave heights correlate with an increase of flow depths at both positions and flow depth generally decrease with increasing distance from the breaking point of the wave. At position 1, the flow depth of the $H_{\text{nom}} = 1.1$ m wave is initially superimposed by a large splash-up that occurs due to the wave breaking close by, whereas the flow depths at position 2 increase more uniformly. For wave heights of $H_{\text{nom}} = 0.9$ m and 1.1 m, a high surface friction of the platform causes the wave tip region of the bore to steepen initially, which has already been discussed in previous literature on bore-like flow (Chanson, 2006, 2009).
Flow velocity measurements, shown in Figure 5 (c) – (d), are provided in this study for some sample instants in time, when the ICM sensors are fully submerged. Therefore, the flow velocities during the impulsive or transient phases were not recorded, yet, a comparison of flow velocities during the quasi-steady phase may still be conducted later. Measured flow velocities were generally higher with increasing wave heights. Further, the flow velocity decreases linearly at position 1 with approximately the same gradient for the contrasted wave height. At the second position further downstream, fluctuations of the flow velocities occur during the decreasing phase that might be related to the measurement technique (air entrainment, insufficient water level).

Significant different momentum fluxes are found in Figure 5 (e) – (f) for the respective wave heights at position 1 and 2. The momentum flux of the highest wave ($H_{nom} = 1.1$ m) surpasses the momentum flux of the $H_{nom} = 0.7$ m wave by up to 60% for position 1 and up to 50% at position 2. On the other hand, the Froude numbers are very similar between the wave heights (Figure 5 (g) – (h)), but increase from position 1 ($F_{rmax} = 2.2$) to position 2 ($F_{rmax} = 2.9$).

Figure 5. Time-histories of flow depths (a) – (b), flow velocities (c) – (d), momentum fluxes (e) – (f) and Froude numbers (g) – (h) for wave heights of $H_{nom} = 0.7$ m, 0.9 m and 1.1 m are shown for tests without the presence of a structure. Position 1 refers to the ICM1 and US2 sensors at $x = 249$ m and position 2 refers to ICM2 and US4 at $x = 254.5$ m.

The hydrodynamic conditions presented in this study are selected to resemble prototype tsunami on-land flow conditions as best as possible in the available experimental facility. Velocity and Froude numbers are close to those prototype values found by Fritz et al. (2006), Fritz et al. (2012) and Iizuka and Matsutomi (2000). The conditions used for testing loads and progressive collapse reside on the lower end of durations necessary for collapse processes to fully finish (Madsen et al., 2008), however, video-based analysis of observed collapse processes has yield durations between 10 s – 40 s (see Section 2.4, see also Figure 1 in Nistor et al. (2017)), indicating that the experimental duration for the flow-structures interaction last sufficiently long to represent the governing processes.
4.2 Hydrodynamic loads on structures

A long-standing discussion has been attributed to whether elasto-plasticity of structures has a considerable effect on the total loading as compared to rigid structures. To that end, this work has – for the first time – compared total horizontal loads on rigid and non-rigid structures when exposed to extreme flow conditions. For a start, the generated nominal wave heights cause varying flow depths and flow velocities that strongly affect the structural loads. Figure 6 illustrates the horizontal forces and overturning moments for wave heights between $H_{nom} = 0.5 - 1.1$ m measured for two tests with rigid structures and one test of the elasto-plastic structure. All tests with the elasto-plastic structures shown herein are tests without any preloading.

![Figure 6. Time-histories of horizontal forces and overturning moments for wave heights between $H_{nom} = 0.5 - 1.1$ m (Panels (a) – (n)). Two tests of the rigid and one test of the elasto-plastic structures without a preload are presented. Please note that the scale of the y-axes changes for panels (g) – (n).](https://doi.org/10.24355/dbbs.084-202208291215-0)
Generally, both tests of rigid structures shown in Figure 6 demonstrate a good repeatability of structural loads in the experimental test facility. The horizontal forces on rigid structures by an impulsive bore-like flow are characterized by a nearly vertical increase during the impulsive phase, settling to a plateau during the quasi-steady phase, and lastly decrease (Arnason et al., 2009; Ko and Yeh, 2018). The maximum horizontal forces and moments on rigid structures occur during the impulsive phase for wave heights larger than 0.6 m (Figure 6 (e) – (n)) and during the quasi-steady phase for lower wave heights up to 0.6 m (Figure 6 (a) – (d)). In particular, the force peak during the impulsive phase increases with larger wave heights for the rigid structures. Most noteworthy and in contrast to rigid structures, it is observed that the horizontal force during the impulsive phase is both less steep and less pronounced for the elasto-plastic structures. Consequently, the elasto-plastic structures may not transmit the impulsive horizontal forces entirely but convert these forces into deformation and vibrational energy depending on the structural vibration modes. This was observed for cases where the elasto-plastic structures were not preloaded and were therefore able to withstand the impulsive loads exceeding their long-term load-bearing capacity.

During the quasi-steady phase, the forces on the elasto-plastic structures are mostly identical to the forces on the rigid structures, however, some exceptions persist. Figure 6 (c) and (k) show a slightly higher horizontal force of the elasto-plastic structure compared to the rigid structure. A conclusive explanation of these increased forces does not appear to be due to the structural rigidity but rather to minor differences in wave generation, which were otherwise demonstrated to be very replicable. In summary, it is deduced that the shape-induced drag coefficient linking the quasi-steady hydrodynamic conditions to the structural loads is identical for elasto-plastic structures without preload and for rigid structures.

Overturning moments in this study are mainly driven by the horizontal force and the corresponding lever arm. Therefore, the general trends of their time-histories are comparable to those of the horizontal force. It is noteworthy that the peak moment during the impulsive phase was generally higher for the rigid than for the elasto-plastic structures. Furthermore, the impulsive forces of rigid and elasto-plastic structures are less different for larger wave heights compared to the maximum overturning moments (Figure 6 (i) – (n)). This suggests that the wall sheathing and the load-bearing system of the elasto-plastic structure transfers the horizontal force to the bottom plate, while the peak overturning moments are not transferred entirely. Hence, the load transfer path of the overturning moment might be interrupted locally due to structural deformation or even partial failure.

4.3 Damage classification and sequences of collapse

After looking into the global force and momentum conditions on rigid and elasto-plastic specimen without preloading, it will be important to understand the collapse process the elasto-plastic structures undergo. In the presented experimental test series, it was not observed that any structure failed without preloading, thus all collapsing structures were already preloaded (cf. Table 2). For an in-depth analysis of the failure mechanisms in this study, the failure mechanisms are categorized in this section using sequences of photographs from the high-speed cameras. In general, three types of failure mechanisms were identified, namely: (i) rapid collapse of the front studs, (ii) tensile forces get transferred insufficiently at the roof’s rear joist and (iii) asymmetric failure caused by an interrupted load transfer between the longitudinal walls and the roof’s joist (cf. Table 2).

Failure mechanism (i) is initiated by a rapid collapse of the front studs. To break down this failure mechanism, Figure 7 shows camera images of the collapse, whereas schematic sketches of the consecutive processes are given in Figure 8. During the highly impulsive initial impact, the force acts
primarily on the front panel of the building and is transferred to the rear panel via the roof. The front panel experiences a deflection to the inside of the building, while the rear panels are deflected towards the outside of the structure. Subsequently, the bore is directed upwards in front of the structure, causing the water level and the hydrostatic force to increase (Figure 7 (b)). During the increasing structural load, the studs at the front panels deflect and break as soon as the load-bearing capacity is surpassed. As a consequence, the uprising water level in front of the structure attacks the front section of the roof and applies a force directed upwards and rearwards (Figure 7 (c)). Thus, the roof’s joists become detached from the longitudinal and rear walls, resulting in the loss of structural support provided by the roof (Figure 7 (d)). There is a minor time offset in the load increase of both structures that indicates a pronounced slippage of the elasto-plastic structure at the beginning of load application (Figure 7 (a)), which was also seen in the structural tests during model design. Besides, the impulsiveness of the force increase is comparable between both structures, in contrast to the findings for the structure without preload. Therefore, these load curves suggest that preloaded structures can no longer convert the wave energy and react more like rigid structures. Additionally, the maximum forces occur simultaneously and with the same magnitude. Briefly after reaching the maximum force, the force time-history of the elasto-plastic structure falls below that of the rigid structure at \( t = 1.21 \) s (Figure 7 (a)). As of this time, it can be assumed that the front studs are fractured and insufficiently transfer the load to the bottom plate. Hence, the rapid collapse of the structural front initiated failure mechanism (i) and paved the way for detrimental load application to the roof, resulting in the overall collapse.

**Figure 7.** Time-history of horizontal force and water depths (a) as well as a sequence of high-speed camera photographs (b) – (e) depicting the collapse process of EP3 classified as failure
mechanism (i). The time-history of EP3 and a rigid structure (a) provides the horizontal force on the left axis and the water depth on the right axis along with the time stamps of the photographs.

Figure 8. Schematic sketch of the underlying processes causing failure mechanism (i).

A second failure mechanism (ii) is associated with the insufficient transfer of tensile forces at the roof’s rear joist, visualized by high-speed camera photographs in Figure 9 and illustrated with a schematic sketch in Figure 10. Initially, the structural front is bent towards the inside comparable to failure mechanism (i) but in this case the studs remain sturdy (Figure 9 (b)). Instead, the roof sheathing gets detached from the rear joist, interrupting the shear transfer from the sheathing to the rear joist and causing a sideward rotation of the sheathing. Therefore, the collapse is initiated at the rear side of the structure and causes the roof to lose its structural integrity. Further, the structural front loses support from the roof so the front studs have to transfer increased loads towards the bottom plate and get ultimately damaged. This failure mechanism is accompanied by a sudden upward movement of the roof that is partially overtopped as the entire failure occurs at a later stage (Figure 9 (d)). Furthermore, the collapse process is supported by the load time-histories (Figure 9 (a)), where the load curves are comparable between the two structures for the entire impulsive phase up to the quasi-steady phase at \( t = 1.6 \) s. This indicates that the high forces during the impulsive phase could be transferred via the front studs, while subsequently the structural deformation became so severe that the bore separated individual fastener connections – in this case by dislodging the roof from the lateral walls.

https://doi.org/10.24355/dbbs.084-202208291215-0
Figure 9. Time-history of horizontal force and water depths (a) as well as a sequence of high-speed camera photographs (b) – (d) depicting the collapse process of EP6 classified as failure mechanism (ii). The time-history of EP6 and a rigid structure (a) provides the horizontal force on the left axis and the water depth on the right axis along with the time stamps of the photographs.

Figure 10. Schematic sketch of the underlying processes causing failure mechanism (ii).
Failure mechanism (iii) is related to an interruption of the load transfer (shear flow) around the roof joists above the lateral walls (cf. Figure 11, Figure 12). The initial impact of the bore separates the lateral wall sheathings from the roof joist, which originally transfer the shear loads from the roof to the lateral studs. As this load transfer path gets disrupted, the load-bearing system shifts so that the rear wall has to carry loads that were originally transferred by the lateral wall. Since the load-bearing system shifts, the shear centre is relocated causing the entire structure to start rotating in three dimensions (roll, pitch and yaw) until the front studs ultimately fail (Figure 11 (c)). As a result of the rotation, the rear wall experiences an upward force that ultimately separates the rear stud and causes the global failure (Figure 11 (d)). Linking these deductions to the force time-histories in Figure 11 (a), it is apparent that the loads during the impulsive phase are comparable or even higher for the collapsed structure until $t = 1.5$ s. Therefore, the front studs remained sturdy during the impulsive phase until the rotational movement of the structure and the shift of the load-bearing system causes an uplift of the rear stud and the total collapse.

**Figure 11.** Time-history of horizontal force and water depths (a) as well as a sequence of high-speed camera photographs (b) – (e) depicting the collapse process of EP9 classified as failure mechanism (iii). The time-history of EP9 and a rigid structure (a) provides the horizontal force on the left axis and the water depth on the right axis along with the time stamps of the photographs.
All in all, three fundamental failure mechanisms have been identified and characterized by their mechanisms to reduce the load-bearing capacity for this specific structure. The failure mechanisms yield different destruction patterns on the structures, which can result in either complete or partial failure. Likewise, the initial movement patterns of the debris parts differ, which are e.g. predominantly directed upwards (i) or result in a rotational movement (iii). Thus, modelling collapsing structures realistically may be used for producing relevant input parameters of subsequent or even simultaneous experimental modelling of debris flows.

4.4 Effect of load history

As seen in the preceding section, after repeated exposure to broken solitary waves, the buildings yielded and ultimately collapsed in one of three distinct failure modes. In the following, it will be investigated to what extent the various structure types have absorbed the maximum loads (horizontal force and overturning moment) and transmitted them to the force sensor. At the same time, it can be shown whether the instantaneous occurrence of the maximum force can be supported by the elasto-plastic structures, for which the respective load gradients serve as an indicator.

Figure 13 and Figure 14 represent typical time-histories, gradients and maximum gradients of horizontal forces and overturning moments for wave heights of 0.9 m and 1.1 m for three types of performed tests – namely, tests with a rigid structure, an elasto-plastic structure without preload and an elasto-plastic structure that collapsed during the test. In the case of the elasto-plastic structure without preload, both maximum loads are not transmitted entirely to the force sensor, instead they are attenuated by the structure through elastic or plastic deformations. This is indicated by the maximum horizontal force (Figure 13 (a)) being about 35% lower and the maximum overturning moment (Figure 13 (d)) about 50% lower than the respective tests with rigid specimens for a wave height of 0.9 m. Besides, the time-histories of forces and overturning moments are comparable during the quasi-steady phase for the rigid and elasto-plastic structure, although the latter shows higher fluctuations. Remarkably, the collapsing structure did accurately replicate the maximum horizontal force while having a considerably lower maximum overturning moment. Furthermore, the large gradient during the impulsive phase was observed for the horizontal force, yet it was not recorded for the overturning moment – this indicates a structural failure in the load path of the overturning moment.
Figure 13. Time-histories of horizontal forces and overturning moments, respective load gradients and the maximum load gradients as bar chart for a wave height of $H_{nom} = 0.9$ m. Data series of the rigid structure with a black solid line, an elasto-plastic structure without preload with a blue dashed line and a collapsed structure (EP3) with a red dash-dotted line are given.

Figure 14. Time-histories of horizontal forces and overturning moments, respective load gradients and the maximum load gradients as bar chart for a wave height of $H_{nom} = 1.1$ m. Data series of the rigid structure with a black solid line, an elasto-plastic structure without preload with a blue dashed line and a collapsed structure (EP7) with a red dash-dotted line are given.
A comprehensive overview of the normalized maximum loads \( F_{h,\text{max}}^*, M_{h,\text{max}}^* \) and the normalized maximum load gradients \( \Delta F_{h,\text{max}}^*/\Delta t, \Delta M_{h,\text{max}}^*/\Delta t \) is given in Figure 15 related to the nominal wave height. Figure 15 is a normalized chart where the respective maximum loads and load gradients are normalized using the ensemble averaged maximum loads and load gradients from the rigid reference tests for each nominal wave height (two to three repetitions). Thus, the following equation is used to determine the normalized maximum horizontal force \( F_{h,\text{max}}^* \) for an experimental test:

\[
F_{h,\text{max}}^*(H_{\text{nom}}) = \frac{F_{h,\text{max}}(H_{\text{nom}})}{F_{h,\text{max, rigid}}(H_{\text{nom}})}
\]  

(9)

where \( F_{h,\text{max, rigid}}(H_{\text{nom}}) \) is the ensemble averaged maximum horizontal force of the rigid reference tests for a specific nominal wave height. The same calculation procedure using the average of the rigid reference tests is used for the normalized overturning moments and load gradients. The normalized loads indicate the share of the force measured in the elasto-plastic structures to the rigid comparison cases and thereby give an indication of the load transfer within the individual structures is consistent or disrupted. Further, the normalized gradients allow comparing the impulsiveness of the applied horizontal forces and overturning moments.

**Figure 15.** Normalized maximum horizontal forces (a), overturning moments (b), normalized maximum gradients of horizontal forces (c) and of overturning moments (d). It is distinguished between test series with elasto-plastic structures without preload, elasto-plastic structures with preload and the preloaded elasto-plastic structures that collapsed during the test. A horizontal black line indicates unity, that is, the average of the rigid tests that were used for normalization.
Maximum horizontal forces and overturning moments were seen in Figure 6 to occur predominantly in the quasi-steady phase for lower wave heights of $H_{\text{nom}} = 0.5$ m – 0.7 m, while larger wave heights are associated with a predominant load during the impulsive phase. Figure 15 (a) and (b) show that the normalized horizontal forces and overturning moments of the elasto-plastic structures without preload (cf. Table 2, column 4) are around or even above unity for wave heights up to 0.7 m. Therefore, the elasto-plastic structures indicate a comparable load transfer for the horizontal forces and the overturning moment with minor deviations. Further, the maximum load gradients are around unity, which indicates a comparable steepness of the load curves for the quasi-steady phase dominated load conditions.

For higher wave heights ($H_{\text{nom}} \geq 0.8$ m) with a more pronounced impulsive phase, the normalized loads on the elasto-plastic structures show a decreasing trend with loads far below the rigid reference case. This trend is seen not only for the horizontal forces and overturning moments but also for the normalized gradients of both loads. Therefore, especially the strong increase during the impulsive phase is attenuated by deformations in the structural load-bearing system. Hence, it can be concluded that the elasto-plastic structures without a preload are considerably restricted in their ability to transfer loads of higher wave heights, especially across the load path of the overturning moment. Instead, these structures transform the applied wave energy potentially into a structural response and deformations for impulsive phase-driven conditions.

The elasto-plastic structures with a preload, which had already been exposed to one or multiple waves (cf. Table 2, columns 5-8), show higher loads and load gradients compared to the non-preloaded structures. However, there is a similar decreasing trend over increasing wave heights with normalized values below unity for wave heights larger than 0.9 m. As the preload increases, the structures lose capabilities to dampen the wave energy during the impulsive phase and convert it into deformations. Therefore, the elasto-plastic structures with preload react more stiffly and may even exceed the loads on rigid structures as a result of their structural response. The assumption of a stiffer structural response is supported by the load gradients that are generally higher compared to the elasto-plastic structures without preload. However, the scatter is significantly larger in these measurements, which may be explained by the preloading having been different for all structures, which in turn caused different deformations and structural responses.

Starting with a wave height of 0.8 m, the elasto-plastic structures with a significant preload collapsed during the experimental tests. Figure 15 shows that the normalized loads and load gradients of the collapsing structures are mostly higher than those of the other elasto-plastic structures. Likewise, the maximum horizontal forces and load gradients for $H_{\text{nom}} = 0.8$ m and 0.9 m are mostly higher than those of the rigid structure, while they become lower for even higher wave heights. However, the normalized overturning moments are generally lower than the normalized horizontal forces, implying a predominant failure in the load transfer system of the overturning moment. Due to the limited number of experiments, the results are scattered; however, a general decreasing trend can be observed for increasing wave heights. This strengthens the hypothesis that the preloaded structures are no longer able to attenuate the maximum load conditions for $H_{\text{nom}} = 0.8$ m and 0.9 m and that the loads occurring at $H_{\text{nom}} = 1.0$ m and 1.1 m cause significant plastic deformations or even a rapid collapse. In cases of a rapid collapse, the preloaded structure then fails even before reaching the maximum horizontal force, possibly related to a failure of the front studs (failure mechanism (i)).
5 Discussion

Results of the presented experiments indicate that scaled elasto-plastic structures with and without preloading show different structural characteristics compared to rigid structures during bore-like hydrodynamic loads. Additionally, when structures reach their load-bearing capacity, the elasto-plastic structures may not fully transfer the loads anymore but rather transform them into elastic and plastic deformations potentially leading to a collapse. A fundamental study by Linton et al. (2013) provided valuable experimental insights into tsunami wave loading on three different light-frame wood walls. The unanchored walls failed during the tests by bending of the bottom plate. However, this failure mechanism was due to the idealized experimental setup, where no realistic load-bearing system (e.g., of a residential house) was reproduced. Hence, these authors suggested that a simplified but realistic representation of the load-bearing system appears to be necessary for investigations of collapsing structures. The present study identified weak spots of the load-bearing system of a simplified but reasonable model design in case of extraordinary loads and exhibits a great variability even for idealized constructions with one type of light-frame wood wall. In a subsequent step, the construction of light-frame wood walls could be varied depending on local building codes with the objective to evaluate their performance and failure mechanisms for realistic extreme hydrodynamic events as suggested by Linton et al. (2013). However, this requires not only diligent model design but also a sufficient quantity of experimental tests in a large-scale facility. Further, it should be noted that the reasons for different failure mechanisms might be attributed to major quality variations in wood as construction material and (Ross and Bergman, 2010), which underlines the importance of repetitive testing. In addition, wood as an anisotropic material should be considered specifically during model design and its effects could be subject to further investigation in future studies.

Duncan et al. (2021) conducted experimental tests with scaled wood-frame residential structures and observed the progressive damage and failure under regular wave conditions based on Hurricane Sandy. These authors found that the collapse was initiated at the first floor and lead to a disconnection from the foundation structure. It should be emphasized that not only the different hydrodynamic conditions (hurricane-like vs. solitary waves) but also the model design (idealized elasto-plastic building without windows/doors vs. residential structure with several architectural features) have a significant impact on the observed load cases and failure mechanisms. Since these authors compared a slab-on-grade and an elevated structure, the focus of this study was on the evaluation of soft-storey failures, which can occur in hurricane but also in tsunami conditions.

Vertical hydrostatic and hydrodynamic loads become a significant concern in the design of elevated structures (Moon et al., 2019; Alam et al., 2020; Winter et al., 2020; Krautwald et al., 2022), with additional failure mechanisms depending on their architectonical features such as girders or floor joists (Duncan et al., 2021; Krautwald et al., 2021). In contrast, the present study considers other failure mechanisms, which are associated with a rapid, impulse-type loading that is present when large tsunami waves with considerable duration strike. Thereby, this study identified three failure mechanisms for the rapid collapse of an elasto-plastic structure that are: (i) rapid collapse of the front studs, (ii) tensile forces being transferred insufficiently at the roof’s rear joist and (iii) asymmetric failure caused by an interrupted load transfer between the longitudinal walls and the roof’s joist. Overall, this shows the need for further systematic model tests of a variety of structures with and without architectonical features under different hydrodynamic load conditions.

In this study, a distinction has been made between structures with and without preload, for which higher loads and load gradients were detected. Additionally, it has been demonstrated that no structure collapsed while being subjected to a single solitary wave alone, thus generally requiring a higher force regime and a preload for rapid collapse. However, it could not yet be clarified whether
and to what extent the load history also has an influence on possible failure mechanisms. The occurrence of the distinct failure mechanisms could be explained but not ultimately attributed to their underlying causes in this study. Inside the building – fairly airtight after construction – it is suspected that overpressure develops due to the hydrodynamic loads and structural deformation at the front and causes the sheathings to detach from the joists, which forms the basis of failure mechanisms (ii) and (iii). Concerning real tsunami events with multiple waves hitting the structures (Choowong et al., 2008) and increased load durations, the influence of moisture penetration, overpressure of air within the structure, preloading, malfunctioning components, and progressive damage is even more pronounced. In addition, Duncan et al. (2021) has found that the natural frequencies of structures decrease with increasing number of tests similarly to the increased preloading. Therefore, the effect of preloading on structures should be further investigated and quantified to be transferred to real world tsunami conditions. This may be done based on an energy-balance approach that quantifies the energy absorbed. Also, from a timber engineering perspective, the effects of tsunami-like load durations and moisture penetration on the interfaces between structural components and fasteners should be investigated further with the aim to combine the findings of both disciplines and improve the design guidelines.

As a consequence of the high complexity of collapse processes, which are equally dependent on the hydrodynamic boundary conditions as well as the structural design and the contact mechanics, it will be necessary in the future to be able to represent such processes numerically. First efforts exist in this area to simulate the wave-induced failure of e.g. a reinforced concrete wall in Oñate et al. (2022). These authors combined a Particle Finite Element Method (PFEM) for replicating the water surface elevation and coupled the Finite Element Method (FEM) and Discrete Element Method (DEM) to predict the structural behaviour. Those numerical methods rely on very basic benchmark testing of collapsing structures in a reasonably sized experimental test facility. Oñate et al. (2022) used the representation of a collapsing reinforced concrete wall of Arikawa (2009) without adding the complexity of a realistically shaped residential structure. Numerical methods are yet to be developed and validated under consideration of large-scale experimental test series, potentially including both water and air phase simultaneously. Therefore, it seems reasonable to conduct further fundamental experimental test series in which timber engineers, coastal engineers and computational engineers work together on idealized model types and subject them to extreme hydrodynamic loads. Versatile instrumentation (e.g. force transducers, strain gauges, accelerometers and laser displacement sensors) is urgently needed to record and facilitate a more detailed interpretation of the structural reactions, but also to provide the necessary calibration data for numerical analysis.

6 Summary and conclusions

For this study, experimental tests on a 1:5 scale with rigid and elasto-plastic model structures were conducted in the Large Wave Flume in Hannover, Germany. Solitary waves were generated using a piston-type wave maker and the wave characteristics in the offshore section and above a slope with an adjacent horizontal platform were examined. An analysis of the experimental data led to the following findings:

(1) A new methodology to replicate wooden light-frame constructions in hydrodynamic experiments was provided and experimentally analyzed. Elasto-plastic structures were modeled by simultaneous consideration of the Froude and Cauchy scaling laws. By scaling the structural stiffness, the structural deformations caused by the extreme hydrodynamic loads should be preserved allowing for a realistic replication of collapse processes.
(2) Different hydrodynamic conditions were tested by means of increased offshore wave heights that broke over a slope and subsequently propagated over a horizontal platform. Increasing offshore wave heights have been shown to be related to increasing overland flow depths, flow velocities and momentum fluxes. As the momentum fluxes increase, so do the loads on the structures.

(3) Horizontal loads on rigid and elasto-plastic structures have been demonstrated to be comparable for small offshore wave heights, where the quasi-steady phase dominates the horizontal loads. With increasing wave heights, the maximum loads occur during the impulsive phase, where the loads differ significantly between the rigid and elasto-plastic structures. Under impulsive force dominated wave conditions, the elasto-plastic structures without preload deform during the impulsive phase, i.e. forces are only partially transmitted to the force transducer while parts of the wave energy are transformed into deformations and structural response. For the same boundary conditions, preloaded elasto-plastic structures transmit higher forces and the impulsiveness of the applied force to the force transducer and therefore behave gradually more like a rigid structure. This reduced possibility to transform loads into elastic or plastic deformations finally caused the destruction of the preloaded structures.

(4) As soon as the loads and preloads on the elasto-plastic structures became too severe, the structures started to collapse. This collapse could be classified into three failure mechanisms: (i) rapid collapse of the front studs, (ii) tensile forces get transferred insufficiently at the roof’s rear joist and (iii) asymmetric failure caused by an interrupted load transfer between the longitudinal walls and the roof’s joist.

In the future, more types of coastal structures and buildings should be tested, ideally with variations of material as well as design; these should be investigated in the context of different wave boundary conditions, i.e. additional long wave conditions as well as storm surge-related wind wave conditions. Moreover, questions regarding the load history can be conceived further in the presence of tsunamis with multiple waves or even longer inundation periods. Furthermore, as the amount of experimental data on structural collapse processes grows, advanced numerical methods of highly-resolved models should also be utilized to represent and investigate collapse processes of coastal buildings in the real world with the ultimate purpose of improving resilience against tsunami or storm surges of coastal communities.
Conflict of Interest

The authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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Supplementary Material

Videos showing the collapse of the nine model specimens will be available via the following doi as soon as the manuscript got accepted: https://doi.org/10.24355/dbbs.084-202202110737-0

Until then, reviewers are referred to the following link: https://publikationsserver.tu-braunschweig.de/receive/dbbs_mods_00070273

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Data Availability Statement

Raw data were generated at the Coastal Research Center, Hannover, Germany. Derived data supporting the findings of this study are available from the corresponding author, C. Krautwald, on reasonable request.

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