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Newcastle Ocean Baths Stage 2: Coastal inundation assessment

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1. Introduction

This technical note provides a coastal inundation assessment to support the design of Stage 2 of the Newcastle Ocean Baths renewal project. The works comprise renewal of the pavilion buildings and associated public-domain areas, including the revised northern pavilion roof and amenities, Canoe Pool terraces, and the interface between the Canoe Pool, pavilion forecourt and carpark.

The Stage 1 coastal inundation assessment established the site-specific XBeach-NH numerical model and assessed the adopted design wave and water-level conditions. It also developed a response matrix comprising 204 combinations of wave height, wave period and ocean water level, which was applied to approximately 13 years of concurrent observed wave and water-level data to quantify the frequency and duration of inundation across the facility.

For the Stage 2 assessment, the numerical model geometry was updated to represent the proposed pavilion buildings, raised floor levels and surrounding deck arrangement. Additional model simulations were then completed for the adopted design conditions using this updated geometry to quantify inundation depths, water-ingress rates and wave-induced loads at the Stage 2 buildings.

The current architectural and landscape designs have been reviewed against the updated model geometry and results. The principal hydraulic controls remain materially consistent with those represented in the Stage 2 model, including the general pavilion footprints and seaward alignment, the principal finished floor level of RL 4.800 m AHD and the governing levels across the pool and pavilion deck. The existing Stage 1 response matrix and the additional Stage 2 design-condition simulations therefore provide the quantitative basis for assessing the current design. No further numerical simulations were required.

2. Background

2.1 Stage 1 Inundation Assessment

The Stage 1 Coastal Inundation Assessment (Bluecoast, 2024) quantified the inundation risk to the Newcastle Ocean Baths precinct using a combination of deterministic design-event modelling and a probabilistic analysis of long-term wave and water-level records. While a summary of the key results is provided below, it is important that this technical note is read in conjunction with the Stage 1 inundation report for full details of modelling assumptions and limitation.

2.1.1 Deterministic results

Table 1 summarises the modelled inundation levels and depths for the existing pavilion buildings under representative design scenarios. The seaward edge of the pavilion deck is at RL 4.0 m AHD, and all inundation depths are referenced to this level.

Results indicate that inundation was greatest at the central and northern pavilion buildings, where the bleachers and connecting stairs between the pool and pavilion deck allowed waves to travel up the slope with less energy dissipation than at areas fronted by a vertical wall.

Table 1: Inundation level results for select scenarios.

Scenario	Offshore conditions			Inundation depths
	Hs (m)	Tp (s)	WL (m AHD)	1% ex. depth (m)
50% AEP 2023	**6.59**	**12.5**	**1.08**	**0.33**
1% AEP 2023	**8.50**	**12.5**	**1.30**	**0.78**
1% AEP 2050 (+0.23m SLR)			**1.53**	**0.83**
1% AEP 2120 (+0.93m SLR)			**2.23**	**1.27**

Note: Coastal inundation at the Ocean Baths occurs because of waves propagating across the facility. This means inundation depths vary rapidly in time with results presented herein as the one-percentile exceeded inundation depth (or 1% exceeded depth), representing the level exceeded one percent of the time during each simulation. This metric, derived from model time-series at discrete output points, provides a more reliable indication of wave-driven inundation than instantaneous maximum water levels.

2.1.2 Probabilistic assessment

To provide a more complete understanding of the likelihood and frequency of coastal inundation across the facility, a probabilistic inundation assessment was undertaken. A 13-year dataset of observed wave and tide conditions was interpolated across the model matrix to estimate annual exceedance probabilities of inundation at key output points. Key findings from this analysis are summarised in Table 2.

Table 2: Inundation likelihoods for certain events effecting future operations (Bluecoast, 2025).

Event	Threshold	Likelihood		
		%	Hours/ year	# events per year
Overwash into the pool	Depth > 0.3m	7.56	301	121
Small waves reaching inner pool deck	Depth > 0.2m	1.29	105	11
Waves reaching pavilion deck	Depth > 0.1m	0.35	25	2.5
Waves impacting southern pavilion building	Depth > 0.4m	0.01	1	0.5

Note: These likelihoods are based on 1 % exceeded water levels derived from the probabilistic interpolation of modelled inundation scenarios against long-term observed offshore wave and tide conditions.

2.1.3 Implications for Stage 2

The results indicate that the pavilion deck is subject to regular overtopping under present-day conditions, with shallow inundation (0.1–0.2 m) occurring several times per year and deeper events associated with less frequent storm conditions. The central and northern pavilion buildings are the most exposed and should be prioritised for less sensitive internal uses or more resilient design treatments.

The Stage 2 design should therefore minimise uncontrolled seaward openings and incorporate appropriately rated doors, glazing, gates, temporary barriers, drainage and water-resistant finishes at exposed locations.

2.2 Current Stage 2 renewal works

The current design retains the three-pavilion arrangement and a principal ground-floor finished floor level of RL 4.800 m AHD. The northern pavilion contains enlarged male and female amenities beneath a revised elevated and naturally ventilated canopy, with a principal seaward-facing entrance and a lower northern end entry at RL 4.600 m AHD. The central pavilion contains lifeguard and first-aid facilities, staff and storage areas, a ground-floor communications switch room and associated plant and services. The southern pavilion contains food-and-drink premises, kitchen and preparation areas, kiosk, cool room, waste and storage areas, bookable community space, accessible amenities and an internal air lock.

The Canoe Pool design retains the curved pool edge and heritage wall and includes new seating terraces, stairs, handrails, balustrades and open-sided shelters. The transition between the Canoe Pool, pavilion forecourt and carpark is also revised through an adjusted ramp, new stairs, integrated retaining and seating walls, roll-over kerbs, a realigned pedestrian crossing, revised parking, a rain garden, a new bin store and modified pavement treatments. These elements form part of the local overland flow pathway and influence how overtopping water is routed and drained through the precinct.

The principal matters considered in this assessment are the pavilion floor and threshold levels, seaward openings, wet-area configuration, electrical and communications infrastructure, Canoe Pool edge and terrace levels, and the grading and drainage of the pavilion forecourt and carpark interface. The current layout of the Stage 2 works is shown in Figure 1.

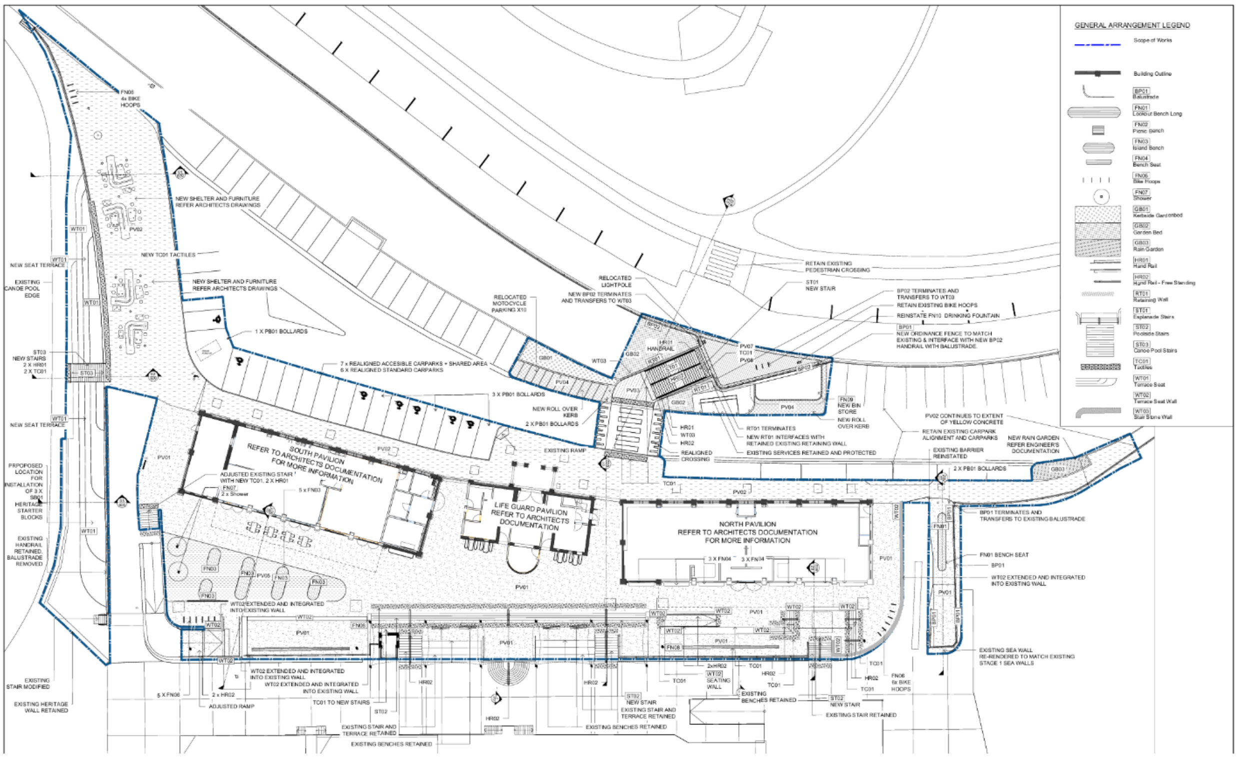

Figure 1: Current Stage 2 pavilion and public-domain arrangement (TCL, 2026).

3. Inundation assessment

3.1 Approach

The assessment applies the site-specific numerical model results to the current architectural, landscape and public-domain design. The modelling provides quantitative estimates of inundation depth, flow velocity, occurrence frequency, entrance discharge and wave-induced façade loading across the pavilion precinct.

The architectural drawings were reviewed to identify the pavilion footprints, floor and threshold levels, external openings, internal uses, wet areas, service rooms and drainage pathways. The landscape drawings were reviewed to assess the Canoe Pool edge and heritage wall, terraces, stairs, ramps, retaining and seating walls, roll-over kerbs, pedestrian crossing, rain garden, bin store and paved routes between the Canoe Pool, pavilion forecourt and carpark.

The principal external hydraulic controls represented in the model remain materially unchanged. The design amendments do not alter the broader wave transformation and overtopping mechanism but affect the local entry, routing and drainage of water once it reaches the pavilion precinct. The model-derived inundation levels, depths, velocities, discharges and loads have therefore been applied to the current design.

Table 3: Modelled scenarios and associated input conditions adopted for the Stage 2 assessment.

Scenario	Hs (m)	Tp (s)	Dp (º)	WL (m AHD)
50% AEP 2023	6.59	12.5	140	1.08

Scenario	Hs (m)	Tp (s)	Dp (º)	WL (m AHD)
1% AEP 2023	8.50	12.5	140	1.30
50% AEP 2050 (+0.23m SLR)	6.59	12.5	140	1.31
1% AEP 2050 (+0.23m SLR)	8.50	12.5	140	1.53
50% AEP 2120 (+0.93m SLR)	6.59	12.5	140	2.01
1% AEP 2120 (+0.93m SLR)	8.50	12.5	140	2.23

3.2 Results

3.2.1 Inundation depths

The numerical modelling indicates that the principal RL 4.800 m AHD finished floor substantially reduces inundation of the pavilion interiors relative to the adjoining pavilion deck.

Figure 2 presents the characteristic 1% exceeded inundation depths at the southern-central and northern pavilions. There is limited difference between the two locations for the present-day and 2050 1% AEP scenarios, with characteristic depths of approximately 0.23 to 0.25 m.

A 2075 sea-level-rise scenario of approximately 0.41 m was derived from the modelling results to represent the 1% AEP condition over an indicative 50-year design life. Under this planning horizon, the characteristic 1% exceeded inundation depths are approximately 0.27 m at the southern-central pavilion and 0.30 m at the northern pavilion.

These depths provide the design basis for doors, gates, glazing, temporary barriers, internal drainage, water-resistant finishes and protection of electrical and service infrastructure. The reduced exposure of the pavilion interiors is primarily attributable to the increased finished floor level and the arrangement of the pool edge, pavilion deck and surrounding public-domain levels represented in the numerical model.

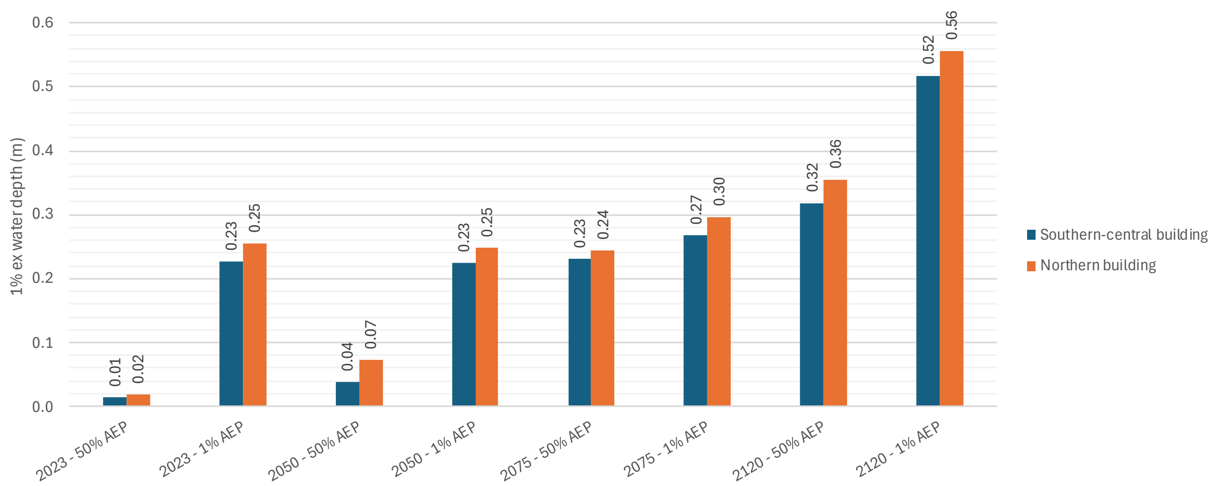

Figure 2: Characteristic 1% exceeded inundation depths at the seaward face of the pavilion buildings.

The main switchboard (MSB) is located on the landward side of the carpark near the opening between the northern and central buildings. The MSB sits on a concrete plinth approximately 0.3 m above ground level. Sensitivity testing undertaken in Stage 1 was used to determine 1 % exceedance water levels at this location, as the inclusion of the modelled buildings is essential to capture accurate inundation behaviour. As such, the results from the Stage 1 modelling were used to derive these inundation depths. The results (Figure 3) indicate that even under the 2120 planning horizon, inundation depths remain below the base of the MSB enclosure. However, vertical splashing and local run-up may still occur during extreme events, and the MSB housing should therefore be constructed to an appropriate weathertight standard.

The current design also includes a ground-floor communications switch room and associated plant within the central pavilion. Water-sensitive equipment should be elevated above the adopted design inundation level, installed in suitably rated enclosures and separated from adjoining wet-area drainage paths. Cable and service penetrations should be sealed, and the services design should include appropriate isolation and post-event inspection procedures.

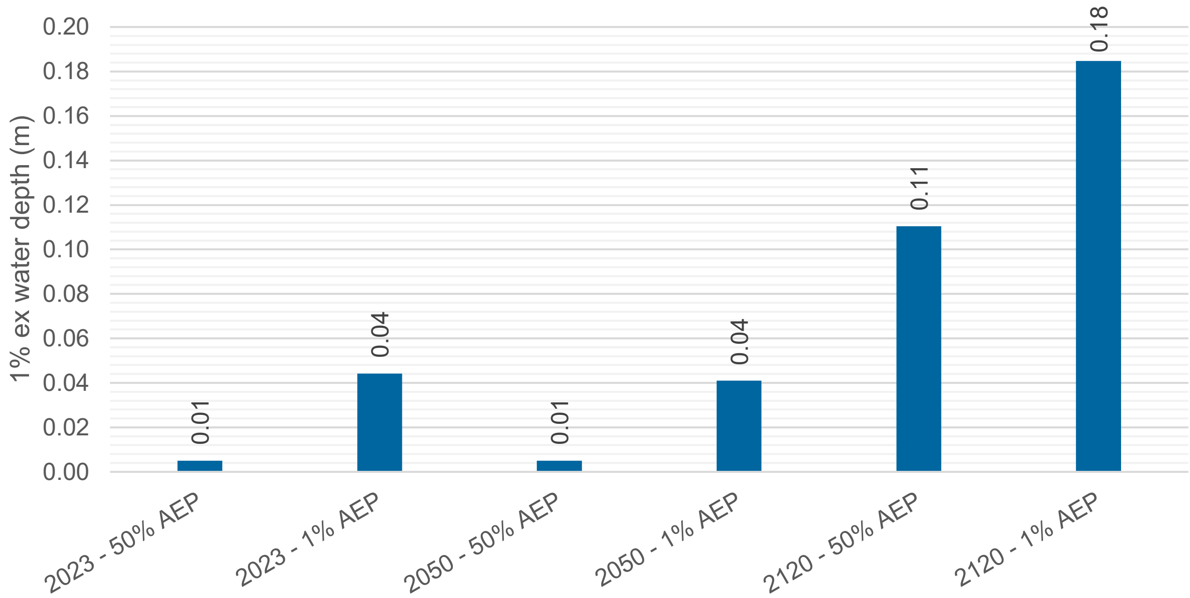

Figure 3: Inundation depths at the Bath's main switch board.

3.2.2 Frequency of inundation

Results from the probabilistic analysis are summarised in Table 4. The assessment indicates that wave overtopping of the pavilion deck occurs relatively frequently, with shallow inundation (greater than 0.05 m) expected approximately 1.5 % of the time, equivalent to 129 hours per year or around three distinct inundation events annually.

Direct wave impacts on the pavilion buildings are far less frequent. The southern building is not expected to experience inundation under the modelled conditions, while the central and northern buildings may be affected on rare occasions, with depths exceeding 0.15 m in front of the façades occurring roughly 1.15 % of the time (averaging about 100 hours per year, or one event every two years).

Overall, these results confirm that wave overtopping of the pavilion deck will remain a regular operational consideration during large swell events, while direct impacts to the pavilion buildings are infrequent and limited in extent.

Table 4: Inundation likelihoods affecting the pavilion buildings.

Event	Threshold	Likelihood		
		% of occurrence	Hours/year	# events per year
Waves reaching the pavilion deck	1% exceedance depth exceeding 0.05m on crest of the pavilion deck	1.47	129	3
Waves impacting the pavilion building	1% exceedance depth exceeding 0.15m in front of the building	1.15	100	0.5
Water ingress reaching side entrances	1% exceedance depth exceeding 0.1m in between buildings	1.43	125	2.5

3.2.3 Water ingress into northern pavilion amenities

The northern pavilion contains a principal seaward-facing entrance to the male and female amenities. The security gate or grille at this entrance will not prevent water ingress during overtopping events.

Model outputs were used to calculate potential water-ingress rates from the time-varying water depth and flow velocity immediately in front of the entrance. The calculated mean unit ingress rates range from approximately 5.5 L/s/m for the present-day 50% AEP condition to 60.3 L/s/m for the 2120 1% AEP condition. The applicable unit rate should be multiplied by the final clear opening width to establish the design inflow rate.

The northern amenities should be treated as wet and semi-open marine spaces. The drainage design should account for inflow through the principal seaward entrance and lower northern end entry, wind-driven rain and salt spray beneath the canopy, distributed floor falls, local ponding behind cubicles, sump and pump capacity and prevention of backflow from external drainage systems.

Internal walls, thresholds and drainage should prevent water entering the amenities from migrating into the central pavilion, communications switch room, plant rooms or other water-sensitive spaces.

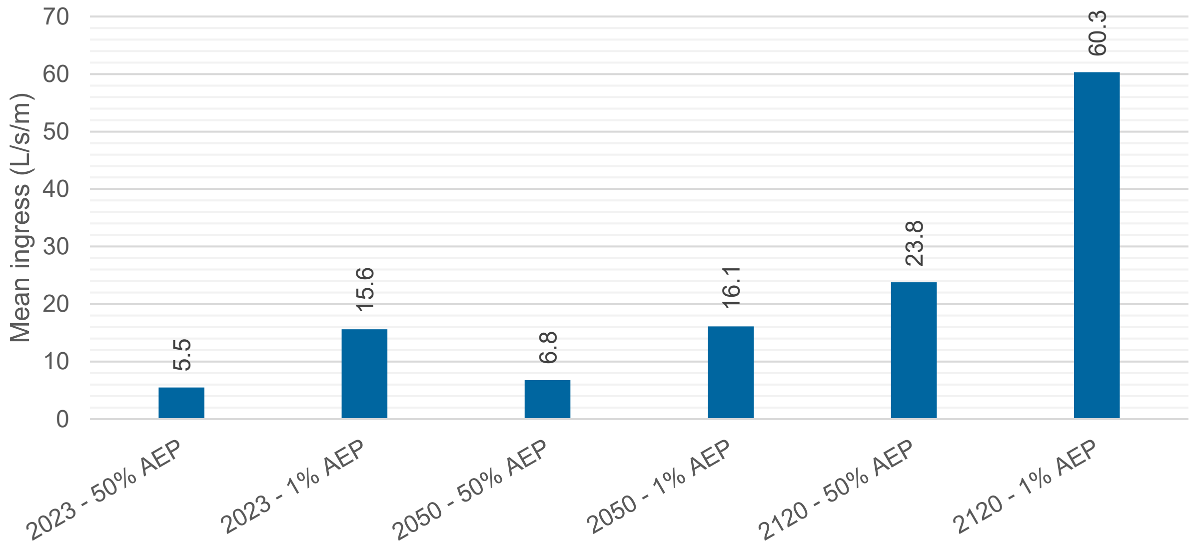

Figure 4: Mean water ingress rates (L/s/m) for the northern pavilion change door entrance for modelled scenarios.

4. Wave loads

4.1 Approach

The numerical model was used to estimate wave-induced loads at the seaward pavilion façades. Dynamic line loads were calculated from the modelled water depth and horizontal flow velocity immediately in front of the building alignment using the empirical relationship proposed by Streicher et al. (2018). The current façade arrangement, including revised doors, gates, glazing, screens and canopy elements, was reviewed against the modelled load envelope. This method captures the transient loads associated with the impact of overtopping bores on the structure. While it does not explicitly resolve short-duration impulsive forces from localised wave breaking directly on the façade, it provides a robust upper-bound estimate of the sustained dynamic pressures generated by overtopping bores at this site.

4.2 Results

The Stage 2 model results show that the raised pavilion floor levels and stepped deck configuration substantially reduce both overtopping depths and near-structure flow velocities relative to the Stage 1 outcomes.

Table 5 presents the calculated maximum and 1 % exceeded (1 % exceedance) wave loads for each design scenario. Because the empirical force equation is sensitive to instantaneous velocity (which is squared in the formulation), the maximum value represents a conservative upper bound, whereas the 1 % exceedance value provides a more realistic indication of sustained loading over the 30-minute model duration.

Table 5: Calculated wave line loads at the pavilion façades.

Scenario	1% exceedance wave load (kN/m)	Max. wave load (kN/m)
50% AEP 2023	0.01	0.86

Scenario	1% exceedance wave load (kN/m)	Max. wave load (kN/m)
1% AEP 2023	4.88	46.12
50% AEP 2050 (+0.23m SLR)	0.08	9.33
1% AEP 2050 (+0.23m SLR)	3.73	23.94
50% AEP 2075 (+0.41m SLR)	6.39	37.02
1% AEP 2075 (+0.41m SLR)	9.66	42.83
50% AEP 2120 (+0.93m SLR)	8.12	70.14
1% AEP 2120 (+0.93m SLR)	22.03	91.10

The 1% exceeded line load represents characteristic sustained loading during each model simulation, while the maximum line load provides a conservative indication of short-duration loading associated with individual overtopping bores.

For the present-day and 2050 1% AEP scenarios, the calculated 1% exceeded line loads are approximately 3.7 to 4.9 kN/m. Larger characteristic and maximum loads are calculated for the 2075 and 2120 planning horizons.

Doors, glazing, gates, screens, solid panels, columns and their connections should be designed for the applicable coastal line loads with appropriate factors of safety. Particular attention should be given to large operable openings, hinges and locking points, glazing edges and mullions, façade returns, lightweight screens, canopy columns and shelter elements.

The calculation method does not fully resolve highly localised impulsive loading from splash, rebound or a wave locally reforming and breaking against an individual element. The structural and façade design should include appropriate allowance for these effects.

5. Assessment of mitigation options

The principal RL 4.8 m AHD pavilion floor materially reduces the exposure of internal building areas. However, shallow overtopping and temporary ponding will continue during large swell events, particularly around exposed entrances, façade openings and the lower northern entry.

The measures summarised in Table 6 address the remaining water-ingress pathways, drainage requirements, water-sensitive services and detailed public-domain grading.

Table 6: Ingress pathways and recommended options.

Area / pathway	Issue	Mitigation options	Notes / expected benefit
Northern pavilion seaward entrance	Primary seaward entry subject to direct wave-driven inundation.	Fit an appropriately rated gate or partially sealed door and consider a temporary wave barrier across the opening.	Reduces direct water ingress to the northern amenities during larger overtopping events.
		Confirm the final clear opening width and use the modelled unit ingress rates to size internal drains, sumps and pumps.	Provides capacity to manage residual ingress where the entrance remains partially open.
Southern pavillion entrances	Sliding and bifold doors may allow ingress of water.	Fit low-profile removable wave barriers at vulnerable openings, with dedicated mounting and restraint points.	Maintains normal access and visibility outside storm events while reducing ingress during forecast conditions.
		Provide suitably rated solid lower panels, doors, glazing, frames and connections where permanent protection is preferred.	Reduces operational reliance on temporary barriers and improves resistance to wave loading.
Entrances not facing seaward	Secondary access doors may admit water from propagating wave bores.	Specify appropriately rated doors and thresholds and consider temporary side barriers or deflectors. Provide local drainage within the air lock.	Limits transfer of water between internal uses and reduces loading and maintenance of door seals.
Communications switch room, electrical equipment and plant	Water-sensitive equipment is located at ground-floor level.	Elevate critical equipment above the adopted design inundation level and provide suitably rated enclosures, sealed penetrations and isolation procedures.	Reduces the likelihood of damage, loss of service and unsafe electrical conditions following inundation.
Grated drains and service penetrations	Water pooling on pavilion deck from overtopping	Design deck drainage to accommodate expected overtopping volumes and connect to sumps with backflow prevention.	Enables rapid dewatering following events and prevents internal backflow.
Canoe Pool edge, terraces and access gaps	Edge levels, walls, stairs and terrace breaks influence the overtopping pathway toward the pavilions.	Maintain the controlling edge, heritage-wall and terrace levels and avoid lower or more direct flow paths toward pavilion entrances.	Retains the principal hydraulic controls represented in the numerical model.

Area / pathway	Issue	Mitigation options	Notes / expected benefit
Canoe Pool, forecourt and carpark interface	Revised stairs, ramp, walls, roll-over kerbs and crossing may redistribute or locally retain overtopping water.	Coordinate final surface levels and falls to provide a defined overland flow route away from pavilion entrances, the lower northern threshold and electrical infrastructure.	Reduces local ponding and prevents landscape elements from directing water toward vulnerable areas.
Bin store, waste areas and grease arrestor	Potential flow obstruction, contamination, flotation, saltwater entry and backflow.	Maintain the overland flow path around the bin store and provide secure containment, suitable thresholds and drainage. Provide anti-flotation design, watertight covers and backflow prevention for the grease arrestor.	Reduces obstruction and contamination and protects below-ground infrastructure during inundation.
Picnic shelters, furniture and balustrades	Elements within the public domain may obstruct flow, trap debris or be exposed to wave and debris loading.	Maintain open flow paths and design columns, foundations, furniture and fixings for marine exposure and applicable flow and debris loads.	Minimises flow concentration and debris accumulation while improving durability.

The Canoe Pool, pavilion forecourt and carpark interface should function as part of the overland flow system. Final surface levels should provide a continuous drainage route through or around the stairs, ramps, retaining walls, seating walls, roll-over kerbs and landscape elements. The drainage design should allow for grated drains becoming temporarily overwhelmed or blocked and for downstream water levels restricting discharge.

Temporary barriers are recommended at vulnerable seaward-facing openings in the southern pavilion and may also be appropriate at the lower northern entry.

The selected system should be designed for the adopted water depths and wave-induced flow conditions. The detailed design should identify the barrier extent and height, restraint and fixing arrangement, storage location, deployment access, inspection requirements and compatibility with accessibility and normal door operation.

Proprietary barriers intended for low-velocity floodwater should not be assumed to be suitable without verification. Additional restraint or a project-specific mounting system may be required.

A suitably designed barrier system can substantially reduce direct water ingress. Residual splash, seepage and water entering through adjoining areas should be managed through internal drainage and resilient finishes.

Temporary barriers, pumps and drainage infrastructure should be supported by an operational Trigger Action Response Plan. The plan should address forecast triggers, responsibility for area closure and barrier deployment, barrier storage and inspection, electrical isolation, post-event cleaning and dewatering, and inspection of doors, seals, drains and pumps.

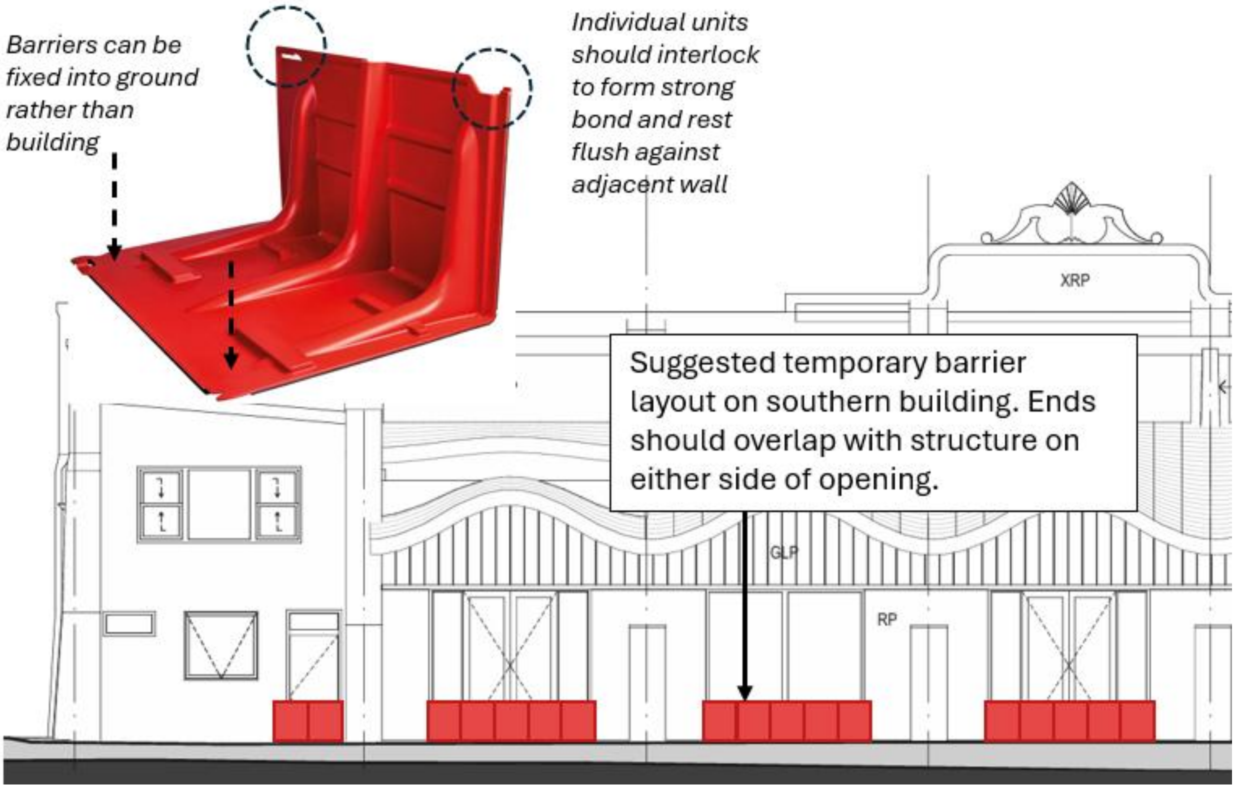

Figure 5: Possible temporary wave barrier layout on seaward façade of Southern building (indicative only).

Under the 1 % AEP (2050 + SLR) scenario, the modelled overtopping depth at the pavilion deck is approximately 0.25–0.30 m. Installation of modular barriers would prevent direct ingress through doors and openings for all events up to this design condition. Residual splash and seepage can be managed through internal drainage and sump pumps, providing effective operational protection without the need for permanent structural modification.

All internal electrical fittings and services should be located above potential inundation levels or designed to tolerate temporary exposure to water ingress.

6. References

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