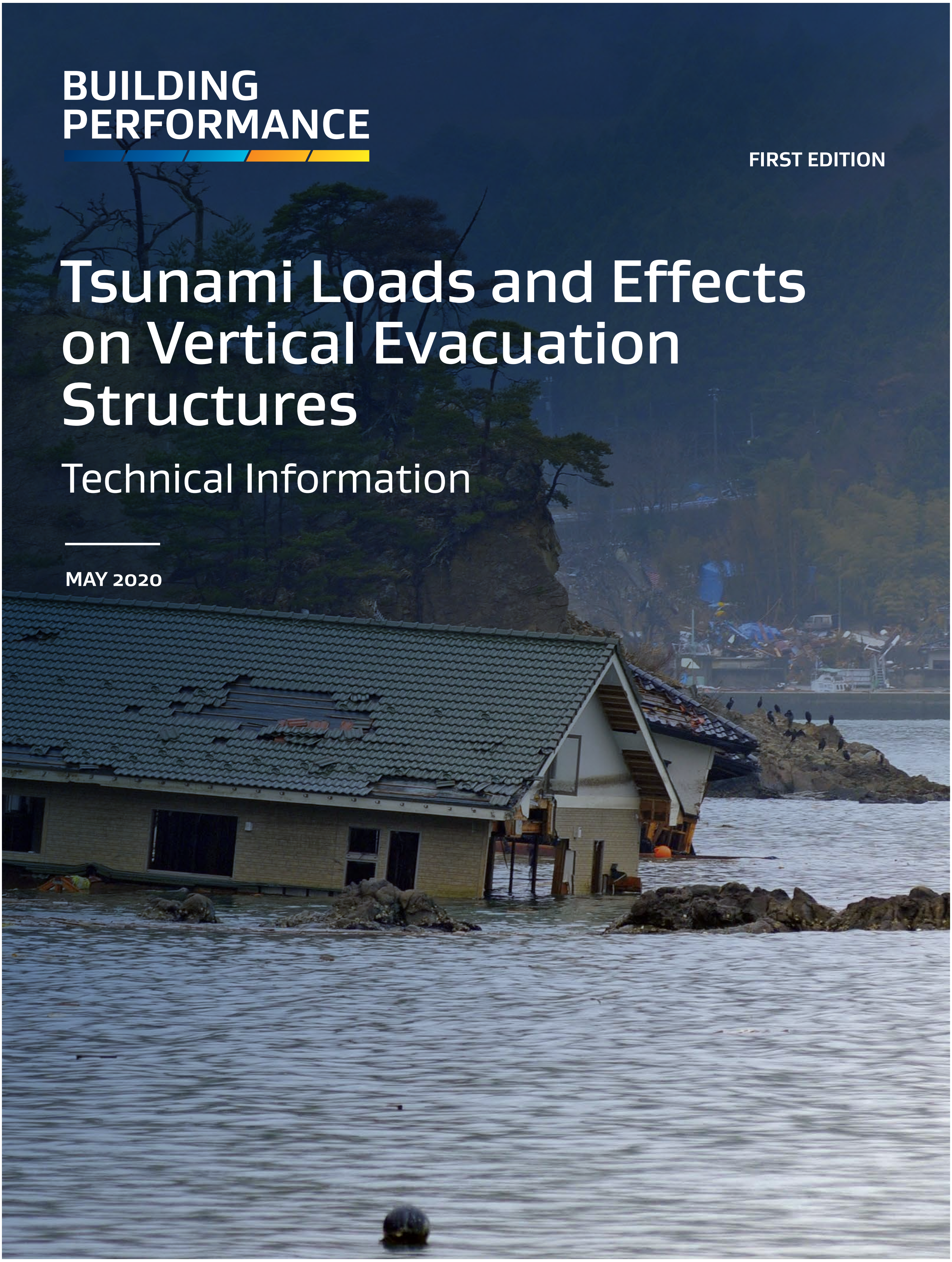
BUILDING
PERFORMANCE

FIRST EDITION

Tsunami Loads and Effects on Vertical Evacuation Structures

Technical Information

MAY 2020



MINISTRY OF BUSINESS,
INNOVATION & EMPLOYMENT
HĪKINA WHAKATUTUKI

New Zealand Government

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Contents

Foreword

The information in this document has been developed by Ministry of Business, Innovation, and Employment (MBIE) to support tsunami risk management across New Zealand. This guideline is a companion document to '*Assessment and Planning for Tsunami Vertical Evacuation*' Director's Guideline for Civil Defence Emergency Management Groups [DGL 21/18] developed by the National Emergency Management Agency (formerly the Ministry of Civil Defence & Emergency Management.

This technical information is intended to support Civil Defence Emergency Management (CDEM) Groups, Building Consent Authorities (BCAs) and technical organizations in determining the minimum tsunami loads and effects which need to be considered in designing a Vertical Evacuation Structure.

In developing this document, a working group was established to provide input on the material of document. The working group reviewed the current international standard and guidance documents such as '*Tsunami Loads and Effects Chapter in Minimum Design Loads and Associated Criteria for Buildings and Other Structures' (ASCE/SEI 7, 2016) and 'Guideline for Design of Structures for Vertical Evacuation from Tsunamis*' (FEMA 646, 2019, Third Edition). Hence, this technical information primarily provides contextualisation on the use of information in ASCE/SEI 7, 2016 standard in line with New Zealand building code and standards.

This publication was prepared under the supervision of the working group established by the MBIE. The working group members consisted of the following members:

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Acknowledgment

Permission from ASCE to reproduce and edit elements of ASCE/SEI 7, 2016 is greatly appreciated. Authors are also grateful to the ASCE 7, Tsunami Loads and Effects Subcommittee, for their support.

The MBIE project team would also like to thank everyone who has been involved in the development of this technical information and acknowledge that without their input this would not have been possible.

SECTION 1

Introduction

1.1 Purpose and scope

This document provides technical information on the tsunami loads and effects on vertical evacuation structures intended to provide short-term refuge during a tsunami event. A tsunami vertical evacuation structure is a building that has sufficient height to elevate evacuees above the level of tsunami inundation, and is designed and constructed with the adequate strength and resiliency needed to resist the effects of tsunami waves.

The information in this document is a resource for engineers, architects, building officials and building owners who are considering the design, construction and operation of tsunami vertical evacuation structures. It provides information on the design of structures able to be used as a refuge for vertical evacuation above the rising waters associated with tsunami inundation. As such, these structures are expected to provide enhanced performance[1] relative to typical buildings for normal occupancies. This technical information is not intended for application to other types of structures or hazards.

The document is intended for use in coastal areas of New Zealand that are exposed to tsunami hazard. However, tsunami vertical evacuation structures are a last-resort safety refuge for people in inundation zones. Timely evacuation outside of an inundation zone, known as horizontal evacuation, is always preferable. Vertical Evacuation Structures (VES) may be considered where local conditions such as short tsunami wave arrival times and poor access to safe areas means reliance on horizontal evacuation alone is not possible. This technical information must be read in conjunction with *Assessment and Planning for Tsunami Vertical Evacuation: Director's Guidelines for Civil Defence Emergency Management Groups* [DGL21/18] published in 2018 by the former Ministry of Civil Defence & Emergency Management, with assessment and planning conducted as per that guideline prior to the application of this document. The Director's Guideline is now maintained by the National Emergency Management Agency and available on its website.

1.2 Background

In some areas of New Zealand, namely coastal areas, a tsunami triggered by local earthquake events may not allow sufficient time for building occupants to evacuate (horizontally) to higher ground. In these cases, one option is to evacuate vertically to the upper-levels of a new or existing building specifically designed for this purpose. A building specifically designated for this purpose is a Vertical Evacuation Structure (VES).

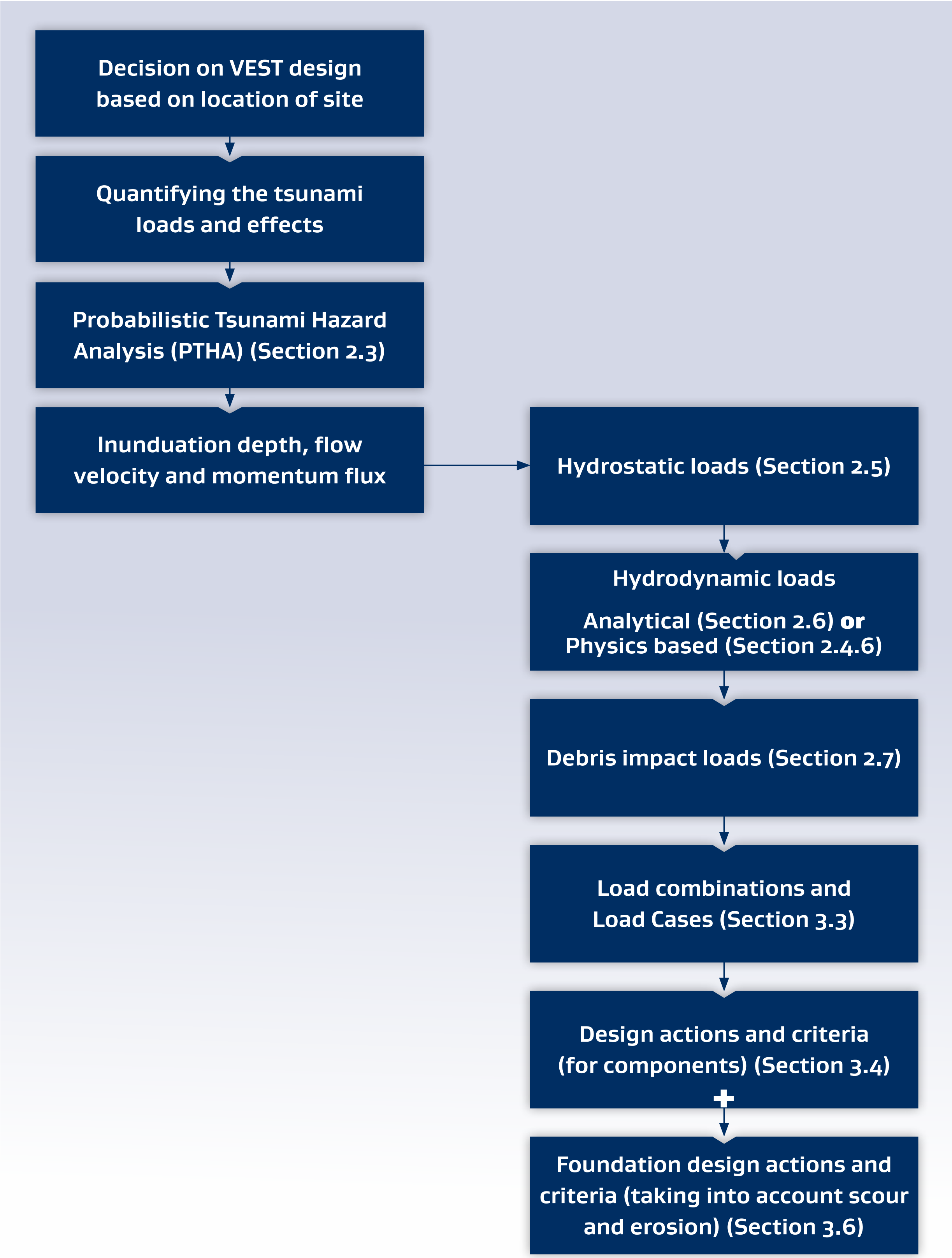
This information represents the second phase of a two-phase process for considering vertical evacuation. Phase-one of the process includes considerations such as understanding the hazard, assessing the risk, and evaluating different risk management measures, as prerequisites to deciding whether to develop a vertical evacuation structure (see Assessment and Planning for Tsunami Vertical Evacuation –Director's Guideline for Civil Defence Emergency Management Groups [DGL21/18] on the National Emergency Management Agency website).

The second phase of the project is to develop technical information for the design of Vertical Evacuation Structures (VES). Hence, the Ministry of Business, Innovation and Employment (MBIE) in collaboration with different stakeholders established a working group to develop this technical information. The focus of this guideline is on re-contextualising existing international guidelines and Standards for use in New Zealand.

1 Chock, G., Yu, G., Thio, H. K., & Lynett, P. J. (2016). Target structural reliability analysis for tsunami hydrodynamic loads of the ASCE 7 standard. Journal of Structural Engineering, 142(11), 04016092.

1.3 Overview of Document

The following flowchart presents an overview of the process and identifies the key variables in calculating tsunami loads and effects. It also refers to the relevant sections of document that explain specific loads and effects associated with a tsunami event.

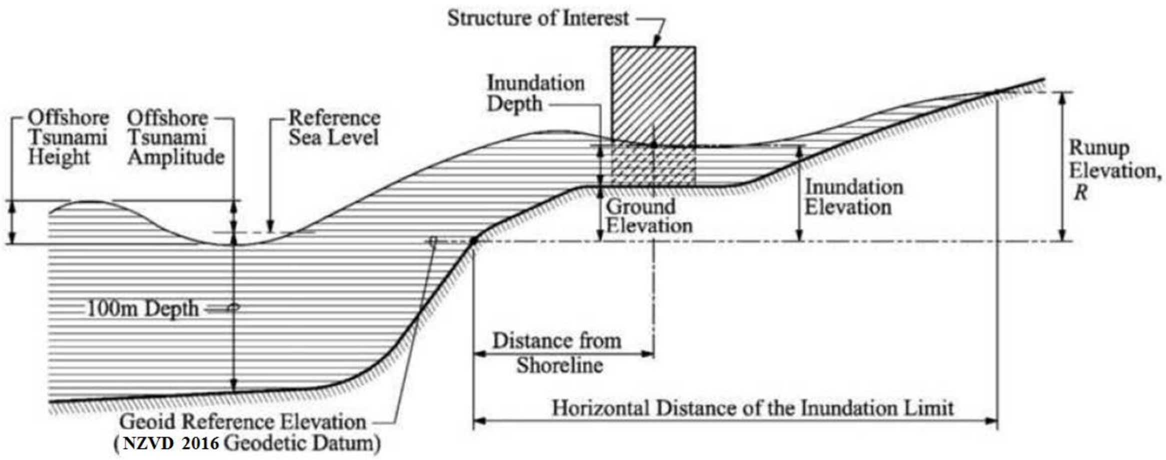


1.4 Definitions, Symbols and Notation

14.1 Definitions

The following definitions apply to the tsunami requirements of this Guideline. Figure 1-1 is also useful for an illustration of some key terms.

Figure 1-1 Illustration of key definitions along a flow transect (Reproduced with permission from ASCE 7-16 and a minor modification)



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A VERTICAL EVACUATION STRUCTURE: A structure that has sufficient height to elevate evacuees above the level of tsunami inundation, and is designed and constructed with the strength and resiliency necessary to resist the forces of tsunami waves, preceding earthquakes and aftershocks that may occur during the period in which the refuge is occupied.

BATHYMETRIC PROFILE: A cross section showing ocean depth plotted as a function of horizontal distance from a reference point (such as a coastline), in which the orientation of the cross section is perpendicular or at some specified orientation angle to the shoreline.

CHANNELIZED SCOUR: Scour that results from a broad flow that is diverted to a focused area such as return flow in a pre-existing stream channel or alongside a seawall.

CLOSURE RATIO (OF INUNDATED PROJECTED AREA): Ratio of the area of enclosure, not including glazing and openings, that is inundated to the total projected vertical plane area of the inundated enclosure surface exposed to flow pressure.

CRITICAL EQUIPMENT OR CRITICAL SYSTEMS: Non-structural components designated essential for the functionality of the vertical evacuation structure or that are necessary to maintain safe containment of hazardous materials.

DEADWEIGHT TONNAGE (DWT): Deadweight Tonnage (DWT) is a vessel's Displacement Tonnage (DT) minus its Lightship Weight (LWT). DWT is a classification used for the carrying capacity of a vessel that is equal to the sum of the weights of cargo, fuel, fresh water, ballast water, provisions, passengers, and crew; it does not include the weight of the vessel itself. Displacement Tonnage is the total weight of a fully loaded vessel. Lightship Weight is the weight of the vessel without cargo, crew, fuel, fresh water, ballast water, provisions, passengers, or crew.

DESIGN STRENGTH: Nominal strength multiplied by a strength reduction factor, ϕ, according to the relevant design material standards.

DESIGN INUNDATION DEPTH: Maximum inundation elevation multiplied by 1.3 minus the site ground elevation.

DESIGN INUNDATION ELEVATION: Maximum inundation elevation multiplied by 1.3.

DESIGN TSUNAMI PARAMETERS: The tsunami parameters used for design, consisting of the inundation depths and flow velocities at the stages of inflow and outflow and most critical to the structure and momentum flux.

DESIGNATED NON-STRUCTURAL COMPONENTS AND SYSTEMS: A non-structural component or system that is essential to the intended function of structure. Designated non-structural systems and their attachment to the structure shall be designed with sufficient strength and stiffness such that their behaviour would not prevent function immediately following any of the design level hazard events specified in this standard. Components of designated non-structural systems shall be designed, qualified, or otherwise protected such that they shall be capable of performing their critical function after the facility is subjected to any of the design level hazards specified in this document. Designated non-structural components shall be classified into categories per classification of parts shown in Table 8.1 of NZS 1170.5.

FROUDE NUMBER, F_r: A dimensionless number defined by $\frac{u}{\sqrt{gh}}$, where u is the flow velocity averaged over the cross section perpendicular to the flow, which is used to classify the tsunami flow velocity relative to the equivalent speed of a shallow water wave propagating in water depth h.

GRADE PLANE: A horizontal reference plane at the site representing the average elevation of the finished ground level adjoining the structure at all exterior walls. Where the finished ground level slopes away from the exterior walls, the grade plane is established by the lowest points within the area between the structure and the property line or, where the property line is more than 1.80 m from the structure, between the structure and points 1.80 m from the structure.

HAZARD-CONSISTENT TSUNAMI SCENARIO: One or more surrogate tsunami scenarios generated from the principal disaggregated seismic source regions, taking into account the net effect of the probabilistic treatment of uncertainty into the offshore wave amplitude of the scenario(s).

INUNDATION DEPTH: The water depth under design tsunami inundation conditions, including relative sea level change, with respect to the grade plane at the structure.

INUNDATION ELEVATION: The elevation of the design tsunami water surface, including relative sea level change, with respect to vertical datum in New Zealand Vertical Datum, NZVD 2016.

INUNDATION DISTANCE: The maximum horizontal inland extent of tsunami flow for the Maximum Considered Tsunami, where the inundation depth above grade becomes zero; the horizontal distance that is flooded, relative to the shoreline defined where the local Mean Sea Level datum elevation is zero.

LIQUEFACTION SCOUR: The limiting case of pore pressure softening, where the effective stress drops to zero under strong flow conditions, causing significant scour. In non-cohesive soils, the shear stress required to initiate sediment transport also drops to zero during liquefaction scour.

MAXIMUM CONSIDERED TSUNAMI: A probabilistic tsunami having a 2% probability of being exceeded in a 50-year period corresponding approximately to a 2500 year return period, at the 84% confidence level.

MOMENTUM FLUX: The quantity $\rho_s hu^2$ for a unit width based on the depth-averaged flow speed u, over the inundation depth h, for equivalent fluid density ρ_s, having the units of force per unit width.

OPEN STRUCTURE: A structure in which the portion within the inundation depth has no greater than 20% closure ratio, and in which the closure does not include any tsunami breakaway walls, and which does not have interior partitions or contents that are unable to pass through and exit the structure as unimpeded waterborne debris.

PRIMARY STRUCTURAL COMPONENT: A structural component required to resist tsunami forces and actions and inundated structural components of the gravity-load-carrying system.

REFERENCE SEA LEVEL: The ambient sea level condition used in site-specific inundation modelling that is typically taken to be Mean High Water Springs (MHWS).

RUN-UP ELEVATION: Ground elevation at the maximum tsunami inundation limit, including relative sea level change, with respect to the local Mean Sea Level reference datum of New Zealand.

SECONDARY STRUCTURAL COMPONENT: A structural component that is not a primary component.

SHOALING: The increase in wave height and wave steepness caused by the decrease in water depth as a wave travels into shallower water.

SOLITON FISSION: Short-period waves generated on the front edge of a tsunami waveform under conditions of shoaling on a long and gentle seabed slope or having abrupt seabed discontinuities, such as fringing reefs.

STRUCTURAL COMPONENT: A component of a building that supports gravity loads or carries lateral forces as part of a continuous load path to the foundations, including beams, columns, slabs, braces, walls, wall piers, coupling beams, and connections.

SUSTAINED FLOW SCOUR: Enhanced local scour results from flow acceleration around a structure. The flow acceleration and associated vortices increase the bottom shear stress over the critical strength of the soil and scour out a localized depression.

TOPOGRAPHIC TRANSECT: Profile of vertical elevation data versus horizontal distance along a cross section of the terrain, in which the orientation of the cross section is perpendicular or at some specified orientation angle to the shoreline.

TSUNAMI AMPLITUDE: The absolute value of the difference between a particular peak or trough of the tsunami and the undisturbed sea level at the time.

TSUNAMI BORE: A steep and turbulent broken wave-front generated on the front edge of a long-period tsunami waveform when shoaling over mild seabed slopes or abrupt seabed discontinuities such as fringing reefs, or in a river estuary.

TSUNAMI BORE HEIGHT: The height of a broken tsunami wave above the water level in front of the bore or above the grade elevation if the bore arrives on nominally dry land.

1.4.2 Symbols and Notation

A_{beam} = vertical projected area of an individual beam element

A_{col} = vertical projected area of an individual column element.

A_{wall} = vertical projected area of an individual wall element

b= width (breadth) of a component or a building subjected to force

B = overall building width

C_{bs} = force coefficient with breakaway slab

C_{cx} = proportion of closure coefficient

C_d = drag coefficient based on quasi-steady forces

C_{dis} = discharge coefficient for overtopping

C_o = orientation coefficient (of debris)

c_{2V} = plunging scour coefficient

d_d = additional drop in grade to the base of wall on the side of a seawall or freestanding retaining wall subject to plunging scour

D_s = scour depth

DT = displacement Tonnage

DWT= deadweight Tonnage of vessel

E_s = earthquake load at Serviceability Limit State

E_u = earthquake load at Ultimate Limit State

F_d = drag force on an element or component

F_{dx} = drag force on the building or structure at each level

F_h = unbalanced hydrostatic lateral force

F_i = debris impact design force

F_{ni} = nominal maximum instantaneous debris impact force

F_{pw} = hydrodynamic force on a perforated wall

F_r = Froude number

F_{TSU} = tsunami load or effect

F_v = buoyancy effect

F_w = load on wall or pier

$F_{w\theta}$ = load on a wall oriented at an angle θ to the flow

G= permanent actions or dead load

g= acceleration caused by gravity

h= tsunami inundation depth above grade plane at the structure

H_B = barrier height of a levee, seawall, or freestanding retaining wall

h_{design} = design inundated depth above grade plane at the structure

h_e = inundated depth of an individual element

h_i = inundation depth at point i

h_o = offshore water depth

H_o = depth to which a barrier is overtopped above the barrier height

h_r = residual water height within a building

h_s = height of structural floor slab above grade plane at the structure

h_{ss} = height of the bottom of the structural floor slab, taken above grade plane at the structure

h_{sx} = story height of story x

H_{TSU} = load caused by tsunami-induced lateral earth pressure under submerged conditions

k = effective stiffness of the impacting debris or the lateral stiffness of the impacted structural element

k_s = fluid density factor to account for suspended soil and other smaller flow-embedded objects

L = live load (imposed actions) in non-refuge floor area

L_{refuge} = public assembly live load (imposed actions) in the tsunami refuge floor area

l_w = length of a structural wall

LWT= Lightship Weight of vessel

$m_{contents}$ = mass of contents in a shipping container

MCT= Maximum Considered Tsunami

m_d = mass of debris object

n= Manning's coefficient

p_u = uplift pressure on slab or building horizontal element

p_{ur} = reduced uplift pressure for slab with opening

p_{uw} = equivalent uniform lateral pressure

q= discharge per unit width over an overtopped structure

R_{max} = dynamic response ratio

R_s = net upward resistance from foundation elements

SLS= Serviceability Limit State according to AS/NZS 1170.0

t = time

t_d = duration of debris impact

TDZ = Tsunami Design Zone

u= tsunami flow velocity

U = jet velocity of plunging flow

ULS= Ultimate Limit State according to AS/NZS 1170.0

u_{max} = maximum tsunami flow velocity at the structure

u_v = vertical component of tsunami flow velocity

V_w = displaced water volume

w_g = width of opening gap in slab

W_s = weight of the structure

z = ground elevation above a local Mean Sea Level datum

γ_s = minimum fluid weight density for design hydrostatic loads

γ_{sw} = effective weight density of seawater

θ = angle between the longitudinal axis of a wall and the flow direction

ϕ = strength reduction factor

ρ_s = minimum fluid mass density for design hydrodynamic loads

ρ_{sw} = effective mass density of seawater

φ = average slope of grade at the structure

φ_i = average slope of grade at point i

Φ = mean slope angle of the nearshore profile

Ψ = angle between the plunging jet at the scour hole and the horizontal

ψ_E = earthquake combination factor as defined according to AS/NZS 1170.0

SECTION 2

Tsunami Load Determination

2.1 Performance Objectives

While specific performance objectives for various forms of rare loadings and building risk categories (or Importance Level) can vary, the minimum recommended structural performance for Vertical Evacuation Structures (VES) under the Maximum Considered Tsunami (MCT) generally follows a trend corresponding to:

- Little or no damage to the structural components in the occupied levels; those structural components are generally located above the design inundation depth
- Likely significant damage to the structural components and foundation in the un-occupied levels; those structural components are generally located below the design inundation depth. However, life safety requirements (equivalent to Ultimate Limit State (ULS) requirements per NZS 1170.5) should be strictly maintained.
- Little or no damage to critical equipment or critical systems. These must be located above the design inundation depth or protected from the waves of tsunami. If damage to the critical equipment/systems or the disruption to the utility network is envisaged during the design phase, the critical backup equipment/systems can also be utilized.

Although significant damage may occur to the structural components located below the design inundation depth, vertical evacuation structures should maintain a reliable and stable refuge above the inundation depth. Hence, the design philosophy for vertical evacuation structures under tsunami loads and effects places a greater emphasis on reserve strength and redundancy of vertical evacuation structures.

2.1.1 Tsunami Performance Objective

In this document, the design tsunami event is termed the Maximum Considered Tsunami (MCT). The method for determining the design inundation depth and flow velocity is described in Section 2.3. Vertical Evacuation Structures (VES) designed for actions in accordance with this document would be expected to provide a stable refuge when subjected to a design tsunami event consistent with the Maximum Considered Tsunami identified for the local area. The performance of vertical evacuation structures in this event would include the potential for significant damage under the Maximum Considered Tsunami in un-occupied floors while maintaining a reliable and stable refuge above the design inundation depth, although the economics of repair versus replacement will be uncertain.

2.1.2 Seismic Performance Objectives

The performance objective for vertical evacuation structures subjected to seismic hazards should be consistent with that of essential facilities such as hospitals, police and fire stations, and emergency operation centres. Vertical Evacuation Structures (VES) should be assigned Importance Level 4 (IL=4), triggering design requirements that provide an enhanced performance relative to typical buildings for normal occupancy. Table 2-1 shows the annual probability of exceedance for the different hazards, in line with AS/NZS 1170.0 for Importance Level 4 buildings.

Table 2-1: Annual probability of exceedance

Design working life	Importance level	Annual probability of exceedance under Ultimate Limit States (ULS)				SLS1	SLS2
		Wind	Snow	Earthquake	Tsunami		
50 years	4	1/2500	1/500	1/2500*	1/2500	1/25	1/500*

*If a site specific tsunami hazard study indicates that actions arising from the earthquakes associate with near-source generated tsunamis events are higher than earthquakes associated with the design response spectrum according to NZS 1170.5 or site specific seismic hazard analysis, the governing event should be used. In this case, the design should meet SLS2 requirements.

2.1.3 Near-Source-Generated Tsunamis

A vertical evacuation structure located in a region susceptible to near-source generated tsunamis is likely to experience strong ground shaking immediately prior to the tsunami. As a properly designed essential facility, it is expected that sufficient reserve capacity will be provided in the structure to resist the subsequent tsunami loading effects. To help ensure adequate strength and ductility in the structure for resisting tsunami load effects, a hazard factor (according to NZS 1170.5) equal to or greater than 0.15, should be used to design the structure. A properly designed VES is also expected to have improved performance of non-structural components during the seismic event including ceilings, walls, light fixtures, fire sprinklers, and other building systems.

For evacuees to feel comfortable entering a Vertical Evacuation Structure (VES) following an earthquake, and remain in the structure during potential aftershocks, it is also important that visible damage to both structural and non-structural components be limited. The critical equipment/systems required to be returned to a fully operational state within an acceptable short time frame (minutes or hours rather than days) in order for the structure to maintain those operations for which it is designated as critical. Particular attention should be focused on non-structural components in the stairwells, ramps and entrances that provide access to and vertical circulation within the structure. Identification of geo-hazards per section 2.1.6 is also essential to ensure the functional performance of vertical evacuation structures under pre-tsunami seismic events.

However, if a site-specific tsunami hazard study indicates that actions arising from the earthquake associated with near-source generated tsunamis are higher than from earthquakes associated with the design response spectrum according to NZS 1170.5 or site specific seismic hazard analysis, the governing seismic event should be used in design.

The residual capacity (where residual cracks or minor yielding are likely to occur at the structure during the pre-tsunami earthquake shaking) of the structure should be evaluated to verify its resistance to subsequent tsunami load effects. Since, it is challenging to assess the residual capacity of the structure if it undergoes incursion into the post-yield range, it is highly recommended to design the vertical evacuation structure to remain elastic under the earthquake associated with near-source generated tsunamis.

For earthquakes that generate near source tsunamis, design for enhanced performance is necessary to ensure that the structure is still usable for a tsunami following a local seismic event. At a minimum, the governing seismic event for designing a vertical evacuation structure for Serviceability Limit State (SLS2) is the earthquake with a 500 years return period. In some areas, the expected co-seismic shaking associated with the Maximum Considered Tsunami (MCT) may impose greater actions on the structure, in which case a higher seismic design threshold should be used.

2.1.4 Far-Source-Generated Tsunamis

Although a vertical evacuation structure is not likely to experience earthquake shaking directly associated with a far-source tsunami, seismic design must be independently included as dictated by the seismic hazard that is present at the site. Even in regions of low seismicity, however, it is recommended that the structure be designed as importance level 4 and a minimum hazard factor equal to 0.15 be adopted, to help ensure adequate strength, and ductility for resisting tsunami load effects.

For far-source generated tsunamis, it is recommended that the condition of the structure after a local earthquake with a 500 year return period (or any other governing seismic event for Serviceability Limit State (SLS2) is used to determine the adequacy of the structure for tsunami loadings and effects. It is recommended to design the vertical evacuation structures to perform in an elastic manner under this level of ground shaking. A site specific seismic hazard analysis is strongly recommended to ensure the appropriate site specific design spectrum.

2.1.5 Performance of Non-structural Elements

Designated non-structural components and systems in structures located in the Tsunami Design Zone should be either protected from tsunami inundation effects or positioned in the structure above the design inundation elevation of the Maximum Considered Tsunami, such that the designated non-structural components and systems will be capable of performing their critical functions during and after the Maximum Considered Tsunami. Hence, the parts and components in Vertical Evacuation Structures (VES) should be classified according to Table 8.1 of NZS 1170.5 depending on the functional and operational requirements needed for the buildings with importance level 4.

2.1.6 Geotechnical Considerations

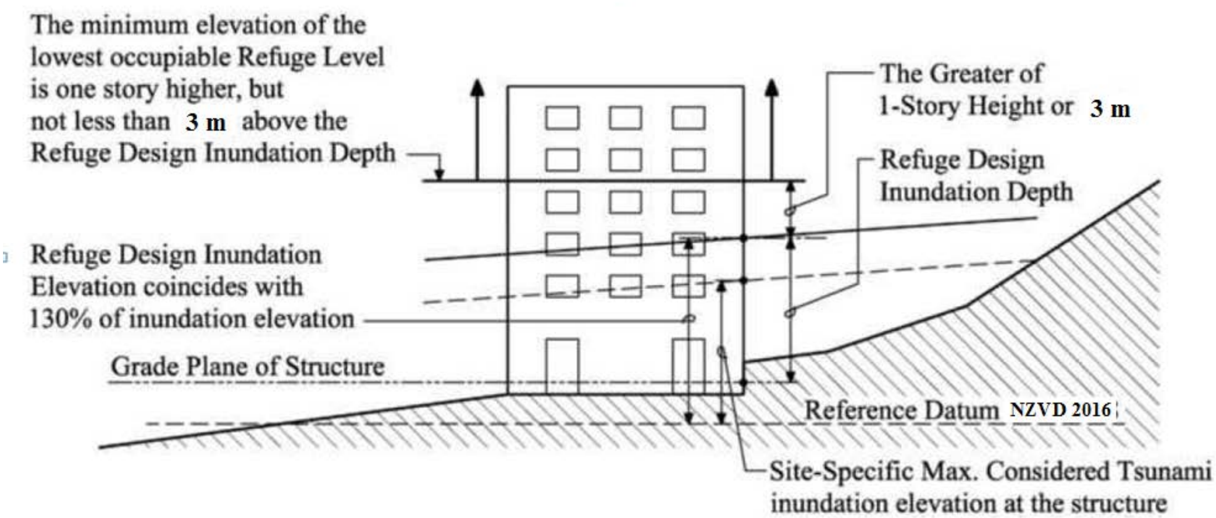
Desired performance objectives for the structural and non-structural systems could not be achieved without taking geotechnical hazards into consideration. Hence, identification and screening of geotechnical hazards related to seismic and tsunami events should be addressed thoroughly during the design process. Vertical evacuation structures should remain functional under pre-tsunami geotechnical hazards. In other words, people in the Tsunami Design Zone (TDZ) should have access to the refuge floors after the pre-tsunami (local) seismic event and be able to evacuate to the building before and from the building after the tsunami event. The pre-tsunami seismic event might trigger geo-hazards such as liquefaction, lateral spread, slope stability, debris flow, extreme elevated groundwater levels, and rapid groundwater draw down, which should not limit public access to the structure or its refuge floors.

2.2 General Requirements

2.2.1 Minimum Inundation Elevation and Depth

Tsunami refuge floors should be located at an elevation not less than the greater of 3.0 m or one-story height above 1.3 times the Maximum Considered Tsunami inundation elevation at the site as determined by a site-specific inundation analysis, as indicated in Figure 2-1.

Figure 2-1 Minimum Refuge Level Elevation (Reproduced with permission from ASCE 7-16 and a minor modification)



2.2.2 Refuge Live Load

A minimum assembly live load, L_{refuge}, of 5.0 kPa should be used in any designated evacuation floor area within a tsunami refuge floor level. The occupancy characteristic of this floor level is indicated as C5 occupancy category in Table 3.1 of AS/NZS 1170.1.

2.2.3 Laydown Impacts

Where the design inundation depth exceeds 1.80 m, the laydown impact of adjacent structures collapsing onto the occupied portions of the building should be considered.

2.2.4 Information on Construction Documents

Construction documents should include tsunami design criteria and the occupancy capacity of each of the tsunami refuge areas. Floor plans should indicate all refuge areas of the facility and exiting routes from each area. The latitude and longitude coordinates of the building should be recorded on the construction documents.

2.2.5 Peer Review

Design should be subjected to the independent peer review by an appropriately licensed design professional. Tsunami and seismic hazard modelling assumptions, model inputs, and results should be also independently peer reviewed by individuals or groups with demonstrated expertise in tsunami modelling and design.

2.3 Inundation Depth and Flow Velocity

2.3.1 Tsunami Hazard Modelling Framework

A site specific Probabilistic Tsunami Hazard Analysis (PTHA) should be performed as the main component of the analysis used to define the inundation elevation and flow-velocities of the Maximum Considered Tsunami (MCT). The MCT will not necessarily represent a single tsunami event but may typically be a composite of multiple events. The PTHA will typically be conducted by modelling scenarios disaggregated from the National Tsunami Hazard Model.

A one-dimensional cross-check model, similar to the Surf Similarity Parameter and Energy Grade Line Analysis in Section 6.5.5.1 of the ASCE/SEI 7-16, is a desirable feature but requires further work to be applicable to New Zealand conditions. At the current time, a stringent peer-review requirement (Section 2.3.1.1 and Section 2.3.2) is used instead of a one-dimensional cross-check.

The MCT should at minimum encompass the 2500 year return period tsunami inundation event at the 84% level of confidence. This is the same return period and confidence level requirement that is used to define the yellow tsunami evacuation zone in DGL 08/16 guideline published by the National Emergency Management Agency. In many situations it will be appropriate to use the same tsunami modelling that was used to define the yellow zone to set the MCT for vertical evacuation (this assumes that the modelling was conducted at Level 3 or 4 as defined in DGL 08/16 guideline). It is acceptable to use a more stringent requirement (longer return period and/or higher confidence level) for the vertical evacuation structure MCT than the yellow zone, but it is not acceptable to use a less stringent one.

The probabilistic methodology used to estimate the MCT should be consistent with the National Tsunami Hazard Model and the tsunami-source data underlying the NTHM. Any deviations from the NTHM need to be discussed and approved by the peer-reviewers (see Section 2.3.2).

Unless a variation is approved by the peer-reviewers:

1. The analysis should include the disaggregation of the seismic sources and associated moment magnitudes that together contribute at least 80% to the at-coast tsunami hazard at the site under consideration.
2. The predominant local sources (<1 hour travel time, more than 20% of disaggregation) should be explicitly modelled to take non-uniform slip distributions into account.

2.3.1.1 Reviewing Requirements

The tsunami hazard modelling used to define the inundation depth of the MCT should be subject to a (minimum) three-step independent peer-review process:

- At the initial project design stage
- At the halfway point of the project
- At the end of the project

Peer reviews should be by experts in tsunami modelling as used for tsunami hazard assessment, and the peer reviewers should have in-depth knowledge of the National Tsunami Hazard Model (NTHM).

2.3.1.2 Alternative Probabilistic Tsunami Hazard Analysis (PTHA) Framework

It is expected that the National Tsunami Hazard Model (NTHM) will be used as the primary basis for meeting the return period and confidence level requirements of the tsunami modelling. The confidence level requirements account for epistemic uncertainties.

Deviations from the NTHM should be discussed and agreed with the peer-reviewers (Section 2.3.1.1) and should take into account the following:

A statistically weighted logic tree approach or equivalent Monte Carlo approach should be used to account for epistemic uncertainties in the model parameters and should provide a sample of tsunamigenic earthquakes and their occurrence probabilities from tectonic, geodetic, historical, and paleo-tsunami data, and estimated plate convergence rates, as follows:

1. Subdivide the occurrence probability systematically to account for variations in the parameters of magnitude, fault depth and geometry, and location, slip distribution, and rupture extent of events consistent with maximum magnitudes, and tidal variation considering at least the Reference Sea Level.
2. To the extent practical and quantifiable, follow a similar logic tree approach to determine samples of tsunami sources such as non-subduction zone earthquakes, landslides, and volcanic eruptions.

2.3.2. Modelling Requirements

The tsunami modelling must be conducted using a well-established and benchmarked tsunami modelling code based on suitable equations for the physics of the scenarios being modelled (for example the shallow-water wave equations), capable of integrated tsunami generation, propagation and inundation modelling. Well-validated tsunami modelling systems that use a unit-source database to represent tsunami generation and propagation (e.g. COMMIT, 2019) are acceptable provided they can adequately represent the physics of the scenarios being modelled.

2.3.2.1 Benchmarking

The tsunami modelling software should have been validated against well-established benchmarking criteria. For example, the certification criteria of the National Tsunami Hazard Mitigation Program (NTHMP) for the USA can be used. Satisfactory performance against the benchmarks should be demonstrated to the peer-reviewers or in the form of peer-reviewed papers/technical reports.

2.3.2.2 Modelling Grid Requirements

A Digital Elevation Model (DEM) from global, regional, and coastal data sets should be used to cover the computational domain from the tsunami sources to the site. The resolution of the model grid (or mesh, or set of nested grids) must be sufficient to capture the tsunami propagation and inundation with non-physical attenuation of tsunami amplitudes being minimized to an acceptable level.

The bathymetry grid of the deep ocean should have a DEM resolution finer than 7000 m, and the offshore model regime with depths 1000-200 m should have a DEM resolution finer than 1000 m.

A Digital Elevation Model (DEM) for the nearshore bathymetry depth of less than 200 m should have a resolution not coarser than 90 m. At bathymetric depths of less than 10 m and on land, the DEM should have a resolution not coarser than 20 m and have topographic elevation accuracy better than 0.3 m in the site area. If a nested grid approach is used, the reduction in grid-spacing between consecutive grids should not be more than a factor of 5.

If buildings and other structures are included for the purposes of more detailed flow analysis, the Digital Elevation Model resolution should have a minimum resolution of 3.0 m and be able to resolve building footprints to capture flow deceleration and acceleration in the built environment.

2.3.2.3 Terrain Roughness

It should be permitted to perform inundation analysis assuming bare-earth conditions with equivalent macro roughness. Bed roughness should be prescribed using the Manning's coefficient n or equivalent values if a roughness model other than Manning's formula is used. It should be permitted to use the values listed in Table 2-2 or other values based on terrain analysis in the recognized literature or as specifically validated for the inundation model used.

Where values other than the defaults are used, the effects of degradation of roughness because of damaging flow characteristics should be considered in the choice of Manning coefficient.

Table 2-2 Manning's Roughness, n

Description of Frictional Surface	n
Coastal water nearshore bottom friction	0.025 to 0.03
Open land or field	0.025
All other cases	0.03
Buildings of at least urban density	0.04

Modelling based on explicit representation of buildings and other obstacles to the tsunami flow can be advantageous, but the approach should first be discussed and approved with the peer-reviewers (Section 2.3.1.1).

2.3.2.4 Sea Level Change and Tides

The direct physical effects of potential relative sea level change should be considered in determining the maximum inundation depth during the project lifecycle. A project lifecycle of not less than 50 years should be used. The rate of potential relative sea level change should be either the historically recorded sea level change rate for the site, or the rate based on the Intergovernmental Panel on Climate Change (IPCC) analysis, whichever is the larger. The potential increase in relative sea level during the project lifecycle of the structure should be added to the Reference Sea Level and incorporated directly into the initial conditions of the tsunami modelling used to estimate the inundation elevation of the MCT. Where there is no site-specific sea level rise study, latest guidelines published by relevant authorities might be used.

Modelling should be conducted assuming a tide-level of Mean High Water Spring.

2.3.2.5 Historical or Paleo-tsunami Inundation Data

Performance of the modelling setup should be validated with available historical and/or paleo-tsunami records at the site of interest.

2.3.3 Tsunami source models

For seismic sources, the earth surface deformation should be determined from the seismic source parameters using a planar fault model (including, where appropriate, planar sub-faults to allow for curvature of the source region and non-uniform slip) accounting for vertical changes to the seafloor.

2.3.3.1 Seismic Subsidence before Tsunami Arrival

Where the seismic source is a local earthquake event, the Maximum Considered Tsunami inundation should consider elevation subsidence, directly computed from the seismic source mechanism.

2.3.3.2 Non-seismic Tsunami Sources

The following tsunami sources should be considered in addition to the seismic sources in the National Tsunami Hazard Model in places where the hazards posed by these sources are documented in the recognized literature:

1. Local coastal and submarine landslide sources documented in the recognized literature as being tsunamigenic and capable of similar (or greater) run-up while also having estimated probabilities of occurrence within an order of magnitude of the principal seismic fault sources.
2. Offshore or near-shore volcanic sources documented in the recognized literature as being tsunamigenic and capable of similar (or greater) run-up while also having estimated probabilities of occurrence within an order of magnitude of the principal seismic fault sources.

2.3.4 Flow Parameters

The inundation parameters required from a study are run-up, inundation depth, flow velocity, and/or specific momentum flux. Maxima of these parameters should be taken over the region of interest as well as site-specific time series. These parameters should be collected for all the disaggregated scenarios. From these values the process of computation of the probabilistic parameters of the MCT from the set of scenarios should be discussed and agreed with the peer reviewers. Typically, a percentile from the weighted distribution of peak water-surface elevation, flow-speed, and/or momentum flux estimates from the set of scenarios will be used. The choice of percentile will be made in consultation with the peer reviewers and may be different for the disaggregated scenarios and for any local sources modelled with non-uniform slip (see 2.3.1). The guiding principle will be adherence to the requirement to encompass the 2500 year return period tsunami inundation event at the 84% level of confidence.

Tsunami inundation depth and velocity should be evaluated from the time series for the site at the stages of inundation defined by the Load Cases in Section 3.3. If the maximum momentum flux is found to occur at an inundation depth different than Load Case 2, the flow conditions corresponding to the maximum momentum flux should be considered in addition to the Load Cases defined in Section 3.3.

2.3.3.3 Amplified Flow Velocities

If the modelling is done assuming bare-earth roughness conditions, the flow velocities should be adjusted for flow amplification in accordance with Section 2.4.2. If the modelling has been conducted using explicit representation of buildings and other obstacles the need for additional flow-amplification may be reduced or waived if this is agreed to by the peer reviewers (Section 2.3.1.1).

2.4 Tsunami Loads and Effects

The following key tsunami loads and effects should be considered for the design of Vertical Evacuation Structures: (1) hydrostatic loads; (2) hydrodynamic loads; (3) debris impact loads; (4) additional gravity loads from retained water on elevated floors. The effect of floating debris that accumulates against the exterior of a building, referred to as debris damming, is included in the evaluation of the degree of closure of the overall building and the tributary width for individual structural components. The tsunami effects associated with these loads such as buoyancy and uplift effects should also be taken into account.

In this document, wave-breaking forces are not considered in the design of Vertical Evacuation Structures (VES). In general, tsunami waves break offshore, and Vertical Evacuation Structures (VES) should be located some distance inland from the shoreline. The term 'wave-breaking' is defined here as a plunging-type breaker in which the entire wave front overturns. When waves break in a plunging mode, the wave front becomes almost vertical, generating an extremely high pressure over an extremely short duration. Once a tsunami wave has broken, it can be considered as a bore because of its very long wavelength. Wave-breaking forces could be critical for Vertical Evacuation Structures (VES) located in the wave-breaking zone, which is beyond the scope of this document. If it is determined that a structure must be located in the wave-breaking zone, further recognized guidelines should be consulted for additional information on wave-breaking forces.

2.4.1 Minimum Fluid Density for Tsunami Loads

Seawater specific weight density γ_{sw} should be taken as 10 kN/m³. Seawater mass density ρ_{sw} should be taken as 1,025 kg/m³. The minimum fluid specific weight density γ_s for determining tsunami hydrostatic loads accounting for suspended solids and debris flow-embedded smaller objects should be:

$$\gamma_s = k_s \gamma_{sw} \tag{2.4-1}$$

The minimum fluid mass density, ρ_s, for determining tsunami hydrodynamic loads accounting for suspended solids and debris flow-embedded smaller objects should be:

$$\rho_s = k_s \rho_{sw} \tag{2.4-2}$$

where k_s, fluid density factor, should be taken as 1.1.

2.4.2 Tsunami Flow Velocity Amplification

The effect of upstream obstructing buildings and structures should be permitted to be considered at a site that is exposed to the flow diffracting conditions given in Section 2.4.2.1 by any of the following:

1. A site-specific inundation analysis that includes modelling of the built environment in accordance with Section 2.3.1.2, or
2. Site-specific physical or numerical modelling in accordance with Section 2.4.2.2 or Section 2.4.6, as applicable.

2.4.2.1 Upstream Obstructing Structures

The effect of upstream obstructions on flow should be considered where the obstructions are enclosed structures of concrete, masonry, or structural steel construction located within 150 m of the site, and both of the following apply:

1. Structures have plan width greater than 30.0 m or 50% of the width of the downstream structure, whichever is greater.
2. The structures exist within the sector between 10 and 55 degrees to either side of the flow vector aligned with the centre third of the width of the downstream structure.

2.4.2.1 Tsunami Flow Velocity Amplification by Physical or Numerical Modelling

The effect of upstream structures on the flow velocity at a downstream site should be permitted to be evaluated using site-specific numerical or physical modelling, as described in Section 2.3.1.2 or 2.4.6.

The velocity determined for a "bare-earth" inundation should be amplified for upstream obstructions as per Section 2.4.2.1. The analysis to Section 2.4.2.1 is not permitted to reduce the flow velocity, except for structural countermeasures designed in accordance with Section 3.5.5.

2.4.3 Directionality of Flow

2.4.3.1 Flow Direction

The design of structures for tsunami loads and effects should consider both incoming and outgoing flow conditions. The principal inflow direction should be assumed to vary by ±22.5 degrees from the transect perpendicular to the orientation of the shoreline, averaged over 150 m to either side of the site. The centre of rotation of the variation of transects should be located at the geometric centre of the structure in plan at the grade plane. However, if the direction of the tsunami can be influenced by preferential flow paths such as rivers, such that the principal inflow direction may be more than 22.5 degrees from the shoreline transect, other appropriate values may be used.

2.4.3.2 Site-Specific Directionality

A site-specific inundation analysis performed in accordance with Section 2.3.1.2 should be permitted to be used to determine the direction of the tsunami flow, provided that the directions determined are assumed to vary by at least ±10 degrees.

2.4.4 Minimum Closure Ratio for Load Determination

Loads on buildings should be calculated assuming a minimum closure ratio of 70% of the inundated projected area along the perimeter of the structure, unless it is an Open Structure as defined in Section 1.4.1.

The load effect of debris accumulation against or within an Open Structure should be considered by using a minimum closure ratio of 50% of the inundated projected area along the perimeter of the Open Structure. Open Structures need not be subject to Load Case 1 of Section 3.3.

2.4.5 Minimum Number of Tsunami Flow Cycles

Design should consider a minimum of two tsunami inflow and outflow cycles; the first tsunami cycle should be based on a design inundation depth at 80% of the Maximum Considered Tsunami (MCT). The second tsunami cycle should be assumed to occur with the Maximum Considered Tsunami design inundation depth at the site.

Local scour effects determined in accordance with Section 3.6, caused by the first tsunami cycle, should be considered as the initial condition when calculating scour from the second tsunami cycle.

2.4.6 Physical Modelling of Tsunami Flow, Loads, and Effects

Physical modelling of tsunami loads and effects for a specific situation of interest should be permitted as an alternative to the prescriptive procedures in Sections 2.4.2 (flow velocity amplification), 2.6 (hydrodynamic loads), 2.7 (debris impact loads), provided that the physical modelling meets all the following criteria:

1. Physical model studies should only be undertaken by groups with demonstrated expertise in hydraulic modelling, with appropriate facilities and measurement equipment to provide robust and repeatable results at an acceptable scale.
2. The facility or facilities used for physical modelling should be capable of generating appropriately scaled flows and inundation depths as specified for Load Cases in Section 3.3. The model geometric scale should be selected to ensure that scale effects are minimised when interpreting the physical model results at prototype scale. This is particularly important if the model study includes the effects of debris impacts or scour.
3. Given the relatively long period of a tsunami wave, flow conditions of interest should be tested for a sufficient duration to ensure that measurements are taken under quasi-static conditions following the impact of a tsunami bore. The quasi-static measurement period should not be affected by reflections from the end or sidewalls of the test facility, nor by a reduction in wave height (e.g. due to the finite reservoir volume used to generate a dam-break bore nor the limited stroke length of a mechanical wave maker).
4. The report of test results should include a discussion of the accuracy of load condition generation and scale effects caused by dynamic and kinematic considerations, including dynamic response of test structures and materials.
5. Test results should be adjusted to account for effective density, as calculated in Section 2.4.1.
6. Test results should be adjusted by a factor equal to 1.25 to account for significant uncertainties in variables.
7. Test results should include the effects of flow directionality in accordance with Section 2.4.3. This inclusion can be accomplished either by direct testing of flow at varying angles of incidence or by a combination of numerical and physical modelling that takes into account directionality of flow.

All physical model studies should be independently peer reviewed by experts in tsunami impacts and hydraulic model studies. This peer review should include careful consideration of the overall model goals, the justification for the geometric, kinematic and dynamic scaling used, the test facility, the model materials and fabrication, the measurement equipment, test method, observations during testing, data processing, and (if applicable) the application of results to the design process.

2.5 Hydrostatic Loads

Hydrostatic loads occur when standing or slowly moving water encounters a structure or structural component. This force always acts perpendicular to the surface of the component of interest. It is caused by an imbalance of pressure due to a differential water depth on opposite sides of a structure or component.

Hydrostatic loads may not be relevant to a structure with a finite (i.e., relatively short) breadth, around which the water can quickly flow and fill in on all sides. Hydrostatic forces are usually important for long structures such as walls or for evaluation of an individual wall panel where the water level on one side differs substantially from the water level on the other side.

2.5.1 Buoyancy Effects

Reduced net weight caused by buoyancy should be evaluated for all inundated structural and designated non-structural elements of the building in accordance with Eq. (2.5-1). Uplift caused by buoyancy should include enclosed spaces without tsunami breakaway walls that have an opening area of less than 25% of the inundated exterior wall area. Buoyancy should also include the effect of air trapped below floors, including integral structural slabs, and in enclosed spaces where the walls are not designed to break away.

All windows, except those designed for large wind-borne missile debris impact or blast loading, should be permitted to be considered openings when the inundation depth reaches the top of the windows or the expected strength of the glazing, whichever is less. The volumetric displacement of foundation elements, excluding deep foundations, should be included in this calculation of uplift.

$$F_v = \gamma_s V_w \tag{2.5-1}$$

where γ_s is the minimum fluid specific weight density for determining tsunami hydrostatic loads and V_w , is the volume of water displaced by the building or part of the building.

2.5.2 Unbalanced Hydrostatic Loads

Inundated structural walls with openings less than 10% of the wall area and either longer than 9.0 m without adjacent tsunami breakaway walls or having a two- or three-sided perimeter structural wall configuration regardless of length should be designed to resist an unbalanced hydrostatic lateral force given by Eq. (2.5-2), occurring during Load Case 1 and Load Case 2 inflow cases defined in Section 3.3.

$$F_h = \tfrac{1}{2}\, \gamma_s b h^2_{design} \tag{2.5-2}$$

where F_h is the hydrostatic load, b is the breadth (width) of the wall or component subjected to load and γ_s, the minimum fluid specific weight density for determining tsunami hydrostatic loads.

2.5.3 Residual Water Surcharge Loads on Floors and Walls

All horizontal floors below the design inundation depth should be designed for dead load plus a residual water surcharge pressure, pr, given by Eq. (2.5-3). Structural walls that have the potential to retain water during drawdown should also be designed for residual water hydrostatic pressure.

$$P_r = \gamma_s h_r \qquad\qquad h_r = h_{design} - h_s \tag{2.5-3}$$

Where h_s = top of floor system (slab) elevation. However, h_r need not exceed the height of the continuous portion of any perimeter structural element at the floor.

2.5.4 Hydrostatic Surcharge Pressure on Foundation

Hydrostatic surcharge pressure caused by tsunami inundation should be calculated as

$$P_s = \gamma_s h_{design} \tag{2.5-4}$$

2.6 Hydrodynamic Loads

When water flows around a structure, hydrodynamic loads are applied to the structure as a whole as well as to individual structural components. These loads are induced by the flow of water moving at moderate to high velocity, and are a function of fluid density, flow velocity and structure geometry.

Hydrodynamic loads should be determined in accordance with this section. The structure's lateral-force-resisting system and all structural components below the inundation elevation at the site should be designed for the hydrodynamic loads given in either Section 2.6.1 or 2.6.2. In addition to the loads calculated in 2.6.1 or 2.6.2, all wall and slab components should also be designed for all applicable loads given in Section 2.6.3.

2.6.1. Simplified Hydrodynamic Loads as Equivalent Uniform Static Pressure

It should be permitted to account for the combination of any unbalanced lateral hydrostatic and hydrodynamic loads by applying an equivalent maximum uniform pressure, p_{uw}, determined in accordance with Eq. (2.6-1), applied over the calculated design inundation depth h_{design} at the site, in each direction of flow.

$$P_{uw} = 1.56\gamma_s h_{design} \tag{2.6-1}$$

2.6.2 Detailed Hydrodynamic Loads

Hydrodynamic loads also known as drag forces, are a combination of the loads caused by the pressure loads from the moving mass of water and friction forces generated as the water flows around the structure or component.

2.6.2.1 Overall Drag Loads on Buildings

The building lateral-force-resisting system should be designed to resist overall drag forces at each level caused either by incoming or outgoing flow at Load Case 2 given by Equations (2.6-2) and (2.6-3).

$$F_{dx} = ½\, \rho_s B(hu^2)C_d C_{cx} x\ 1.25 \tag{2.6-2}$$

where;

› B is the breadth of the building in the plane normal to direction of flow which subjected to flow loads or is the component width perpendicular to the flow,

› h is flow depth at Load Case 2,

› u is tsunami flow velocity at Load Case 2,

› C_d is the drag coefficient (based on quasi-steady loads) for the building as given in Table 2-3 (for buildings) or Table 2-4 (for components), and

› C_{cx} is proportion of closure coefficient or blockage determined in accordance with Eq.(2.6-3)

$$C_{cx} = \frac{\Sigma\,(A_{col} + A_{wall}) + 1.5A_{beam}}{Bh_{sx}} \tag{2.6-3}$$

where:

› A_{col} and A_{wall} are the vertical projected areas of all individual column and wall elements,

› A_{beam} is the combined vertical projected area of the slab edge facing the flow and the deepest beam laterally exposed to the flow, and

› h_{sx} is the story height of x.

The drag forces on the buildings or components of buildings (and coefficient C_{cx}), are calculated for each story below the tsunami design inundation depth for each of the three Load Cases specified in Section 3.3. Any structural or non-structural wall that is not a tsunami breakaway wall should be included in A_{wall}. C_{cx} should not be taken as less than the closure ratio value given in Section 2.4.4 but need not be taken as greater than 1.0.

Table 2-3 Drag Coefficients for Rectilinear Structures (Buildings)

Width to Inundation Depth[a] Ratio (B/h_{sx})	Drag Coefficient C_d
<12	1.25
16	1.3
26	1.4
36	1.5
60	1.75
100	1.8
≥120	2.0

[a] Inundation depth for each of the three Load Cases of inundation specified in Section 3.3. Interpolation should be used for intermediate values of width to inundation depth ratio B/h_{sx}.

The hydrodynamic loads on the individual components per Eq. (2.6-3) should be applied as a pressure resultant (loads) on the projected inundated height of all structural components and exterior wall assemblies below the design inundation depth. The following parameters are used once the hydrodynamic load on the individual components is calculated by Eq. (2.6-3):

For interior components;

› C_d is given in Table 2-4, and
› B is the component width perpendicular to the flow.

For exterior components,

› a C_d value of 2.0 should be used, and
› width dimension B should be taken as the tributary width multiplied by the closure ratio value given in Section 2.4.4.

The drag force on component elements (interior or exterior) should not be additive to the overall drag force computed in Section 2.6.2.1.

Table 2-4 Drag Coefficients for Structural Components

Structural Element Section	Structural Element Section Drag Coefficient C_d
Round column or equilateral polygon with six sides or more	1.2
Rectangular column of at least 2:1 aspect ratio with longer face oriented parallel to flow	1.6
Triangular pointing into flow	1.6
Freestanding wall submerged in flow	1.6
Square or rectangular column with longer face oriented perpendicular to flow	2.0
Triangular column pointing away from flow	2.0
Wall or flat plate, normal to flow	2.0
Diamond-shape column, pointed into the flow (based on face width, not projected width)	2.5
Rectangular beam, normal to flow	2.0
I, L, and channel shapes	2.0

2.6.2.2 Impulsive Hydrodynamic Loads on Vertical Structural Components

Impulsive hydrodynamic loads are caused by the leading edge of a surge of tsunami impacting a structure. Since the tsunami bore is always expected to occur, the impulsive hydrodynamic loads on the structural components (F_w) should be determined by Eq. (2.6-4). Load (F_w) is only applied to all vertical structural components that are wider than 3 times the design inundation depth corresponding to Load Case 2 during inflow as defined in Section 3.3.

$$F_w = \tfrac{3}{4}\, \rho_s C_d b (h_e u^2)_{bore} \times 1.25 \qquad (2.6\text{-}4)$$

The impulsive hydrodynamic force is taken as 1.5 times the hydrodynamic force for the same element, and acts on members at the leading edge of the tsunami bore.

It is also required bore heights corresponding to 1/3 of a particular structural wall width also be considered as potentially the controlling load for that wall. Hence, it is necessary to find the worst loading condition by checking the wall for drag loads per Eq. (2.6-3) for a depth equal to the design inundation depth and impulsive hydrodynamic load per Eq. (2.6-4) for a depth equal to 1/3 of the wall width.

2.6.2.3 Hydrodynamic Load on Perforated Walls, Fpw

For walls with openings that allow flow to pass between wall piers, the force on the elements of the perforated wall F_{pw} should be determined using Eq. (2.6-5), but the value of F_{pw} determined should not be less than F_d per Eq. (2.6-2):

$$F_{pw} = (0.4 C_{cx} + 0.6) F_w \qquad (2.6\text{-}5)$$

2.6.2.4 Hydrodynamic Loads on Walls Angled to the Flow

For walls oriented at an angle less than 90° to the flow directions considered in Section 3.2, the transient hydrodynamic load per unit width, $F_{w\theta}$, should be determined in accordance with Eq. (2.6-6).

$$F_{w\theta} = F_w \sin^2\theta \qquad (2.6\text{-}6)$$

where θ is the inclined angle between the wall and the direction of the flow.

2.6.3 Hydrodynamic Pressures Associated with Floor Systems

2.6.3.1 Flow Stagnation Pressure

The walls and floor systems of buildings that are subject to flow stagnation pressurization should be designed to resist the pressure determined in accordance with Eq. (2.6-7).

$$P_p = \tfrac{1}{2}\, \rho_s u^2) \times 1.25 \qquad (2.6\text{-}7)$$

where u is the maximum free flow velocity for Load Case 2 at that location.

2.6.3.2 Hydrodynamic Surge Uplift at Floor Systems and other Horizontal Components

Floor systems and other horizontal components should be designed to resist the applicable uplift pressures given in this section.

2.6.3.2.1 Floor Systems Submerged during Tsunami Inflow

The floor systems that have a zero grade (e.g. horizontal) and become submerged during a tsunami inundation inflow should be designed for a minimum hydrodynamic uplift pressure of 1.0 kPa applied to the soffit of the slab or floor system. This uplift is an additional Load Case to any hydrostatic buoyancy effects required by Section 2.5.1.

2.6.3.2.2 Sloping Floor Systems

Sloping floor systems with a grade slope, φ, that is greater than 10 degrees should be designed for a redirected uplift pressure applied to the soffit of the slab (floor systems), given by Eq. (2.6-8), but not less than 1.0 kPa.

$$P_u = 1.5\rho_s u_v^{\,2} \times 1.25 \tag{2.6-8}$$

where;

- $u_v = u \tan \varphi$,
- u is the horizontal flow velocity corresponding to a water depth equal to or greater than h_{ss}, the elevation of the soffit of the floor system, and
- φ = average slope of grade plane beneath the floor system.

2.6.3.3 Tsunami Bore Flow Entrapped in Structural Wall-Slab Recesses

Hydrodynamic loads for bore flows entrapped in structural wall-slab recesses should be determined in accordance with this section. The reductions of load given in Sections 2.6.3.3.2 to 2.6.3.3.5 may be combined, however, the net load reduction should not exceed the maximum individual reduction given by any one of these sections.

2.6.3.3.1 Pressure Load in Structural Wall-Slab Recesses

Where flow of a tsunami bore beneath an elevated slab is prevented by a structural wall located downstream of the upstream edge of the slab, the wall and the slab within hs of the wall should be designed for the outward pressure, P_u, of 16.8 kPa.

Beyond h_s, but within a distance of $h_s + l_w$ from the wall, the slab should be designed for an upward pressure of half of P_u e.g., 8.4 kPa.

The slab outside a distance of $h_s + l_w$ from the wall should be designed for an upward pressure of 1.5 kPa.

2.6.3.3.2 Reduction of Load with Inundation Depth

Where the design inundation depth is less than two-thirds of the clear story height, the uplift pressures specified in Section 2.6.3.3.1 should be permitted to be reduced in accordance with Eq. (2.6-9) but should not be taken as less than 1.5 kPa.

$$P_u = \left(28.25 - 7.66\frac{h_s}{h_{design}}\right) \times 1.25 \tag{2.6-9}$$

where h_s/h_{design} is the ratio of slab height to the design inundation depth.

2.6.3.3.3 Reduction of Load for Wall Openings

A reduced pressure on the wall and slab can be determined in accordance with Eq. (2.6-10) where the wall blocking the bore below the slab has openings through which the flow can pass.

$$P_{ur} = C_{cx} P_u \tag{2.6-10}$$

where C_{cx} is the ratio of the solid area of the wall to the total inundated area of the vertical plane of the inundated portion of the wall at that level.

2.6.3.3.4 Reduction in Load for Slab Openings

Where the slab is provided with an opening gap or breakaway panel designed to create a gap of width wg, adjacent to the wall, then the uplift pressure on the remaining slab should be determined in accordance with Eq. (2.6-11).

$$P_{ur} = C_{bs} P_u \tag{2.6-11}$$

$$\text{where } w_g < 0.5h_s,\ C_{bs} = 1 - \frac{w_g}{h_s} \tag{2.6-12}$$

$$\text{where } w_g \geq 0.5h_s,\ C_{bs} = 0.56 - 0.12\frac{w_g}{h_s} \tag{2.6-13}$$

The value of C_{bs} should not be taken as less than zero.

2.6.3.3.5 Reduction in Load for Tsunami Breakaway Wall

If the wall restricting the flow is designed as a tsunami breakaway wall, then the uplift on the slab should be permitted to be determined in accordance with Section 2.6.3.1, but it need not exceed the pressure equivalent to the total nominal shear force necessary to cause disengagement of the breakaway wall from the slab.

2.7 Debris Impact Loads

Debris impact loads should be determined in accordance with this section. Where the design inundation depth is approximately 1 m or greater, a design should include the effects of debris impact forces. The most severe effect of impact loads within the design inundation depth should be applied to the perimeter gravity-load-carrying structural components located on the principal structural axes perpendicular to the range of inflow or outflow directions defined in Section 2.4.4. Except as specified below, debris impact loads should be applied at points which are critical for flexure and shear on all members in the design inundation depth being evaluated.

Inundation depths and velocities corresponding to Load Cases 1, 2, and 3 defined in Section 3.3 should be used in the estimation of debris impact loads. Impact loads need not be applied simultaneously to all affected structural components. Additionally, debris impact loads need not be combined with other tsunami related loads as determined in other sections of this guideline.

All buildings and other structures meeting the above requirement should be designed for impact by floating wood poles, logs, and vehicles, and for tumbling boulders and concrete debris, per Sections 2.7.2 to 2.7.4.

Where a site is located close to a port or container yard, the potential for strikes from shipping containers and ships and barges should be determined by the procedure in Section 2.7.6. Vertical Evacuation Structures (VES) in the hazard zone for strikes by shipping containers should be designed for impact loads in accordance with Section 2.7.7.

The alternative simplified static method set out in Section 2.7.1 are just used to cross-check the detailed considerations of Sections 2.7.2–2.7.7 in evaluating the impact of poles, logs, vehicles, tumbling boulders, concrete debris, and shipping containers.

Vertical evacuation Structures (VES) determined to be in the hazard zone for strikes by ships and barges in excess of 88,000 lb (40,000 kg) Deadweight Tonnage (DWT), as determined by the procedure of Section 2.7.6, should be designed for impact by these vessels in accordance with Section 2.7.8.

2.7.1 Alternative Simplified Debris Impact Static Load

It should be permitted to account for debris impact by applying the force given by Eq. (2.7-1) as a maximum static load, in lieu of the detailed loads defined in Sections 2.7.2 to 2.7.7. This force should be applied at points critical for flexure and shear and on all such members in the inundation depth corresponding to Load Case 3 defined in Section 3.3.

$$F_i = 1{,}470 C_o \times 1.25 \; [kN] \tag{2.7-1}$$

where C_o is the orientation coefficient, equal to 0.65.

Where it is determined by the site hazard assessment procedure of Section 2.7.6 that the site is not in an impact zone for shipping containers, ships, and barges, then it should be permitted to reduce the simplified debris impact force to 50% of the value given by Eq. (2.7-1).

2.7.2 Wood Logs and Poles

The nominal maximum instantaneous debris impact force caused by the impact of wood logs and poles, F_{ni}, should be determined in accordance with Eq. (2.7-2).

$$F_{ni} = u_{max} \sqrt{k m_d} \tag{2.7-2}$$

The design instantaneous debris impact force caused by the impact of wood logs and poles, F_i, should be determined in accordance with Eq. (2.7-3).

$$F_i = C_o F_{ni} \times 1.25 \tag{2.7-3}$$

Where;

- C_o = Orientation coefficient, equal to 0.65 for logs and poles;
- u_{max} = Maximum flow velocity at the site occurring at depths sufficient to float the debris;
- k = Effective stiffness of the impacting debris or the lateral stiffness of the impacted structural element(s) deformed by the impact, whichever is less; and
- m_d = Mass W_d/g of the debris.

Logs and poles are assumed to strike longitudinally for calculation of debris stiffness in Eq. (2.7-2). The stiffness of the log or pole should be calculated as k =EA/L, in which E is the longitudinal modulus of elasticity of the log, A is its cross sectional area, and L is its length. A minimum weight of 450 kg and minimum log stiffness of 61,300 kN/m should be assumed. The impulse duration for elastic impact should be calculated from Eq. (2.7-4):

$$t_d = \frac{2m_d u_{max}}{F_{mi}} \tag{2.7-4}$$

For an equivalent elastic static analysis, the impact force should be multiplied by the dynamic response factor R_{max} specified in Table 2-5. To obtain intermediate values of R_{max}, linear interpolation should be used. For a wall, the impact should be assumed to act along the horizontal centre of the wall, and the natural period should be permitted to be determined based on the fundamental period of an equivalent column with width equal to one-half of the vertical span of the wall. It also should be allowed to use an alternative method of analysis as per Section 2.7.9. However, stiffness sensitivity analysis is needed to find the variation in outputs.

2.7.3 Impact by Vehicles

Forces to account for the impact of floating vehicles should be applied to vertical structural element(s) at any point greater than 1.0 m above grade up to the maximum depth. The impact force from vehicles should be taken as 130 kN multiplied by 1.25.

Table 2-5 Dynamic Response Ratio for Impulsive Loads, R_{max}

Ratio of Impact Duration to Natural Period of the Impacted Structural Element	R_{max} (Response Ratio)
0.0	0.0
0.1	0.4
0.2	0.8
0.3	1.1
0.4	1.4
0.5	1.5
0.6	1.7
0.7	1.8
0.9	1.8
1.0	1.7
1.1	1.7
1.2	1.6
1.3	1.6
≥1.4	1.5

2.7.4 Impact by Submerged Tumbling Boulder and Concrete Debris

Where the maximum inundation depth exceeds 1.80 m, an impact force of 36 kN multiplied by 1.25 should be applied to vertical structural element(s) at 0.60 m above grade.

2.7.5 Damming of Accumulated Debris

The damming effect caused by accumulation of waterborne debris can be treated as a hydrodynamic force enhanced by the breadth of the debris dam against the front face of the structure. Hence, provision of Section 2.6.2.1 can be used to calculate damming effects.

Since debris damming represents an accumulation of debris across the structural frame, the total debris damming force will likely be resisted by a number of structural components, depending on the framing dimensions and the size of debris dam. The debris damming force should be assumed to act as a uniformly distributed load over the extent of the debris dam. It should be assigned to each resisting structural component by an appropriate tributary width, and distributed uniformly over the submerged height of each resisting component.

2.7.6 Site Hazard Assessment for Shipping Containers, Ships, and Barges

Shipping containers and ships or barges disbursed from container yards, ports, and harbours should be evaluated as potential debris impact objects. In such cases, a probable dispersion region should be identified for each source to determine if the Vertical Evacuation Structure (VES) under consideration is located within a debris impact hazard region, as defined by the procedure in this section. If the structure is within the debris impact hazard region, then impact by shipping containers and/or ships and barges, as appropriate, should be evaluated per Sections 2.7.7 and 2.7.8.

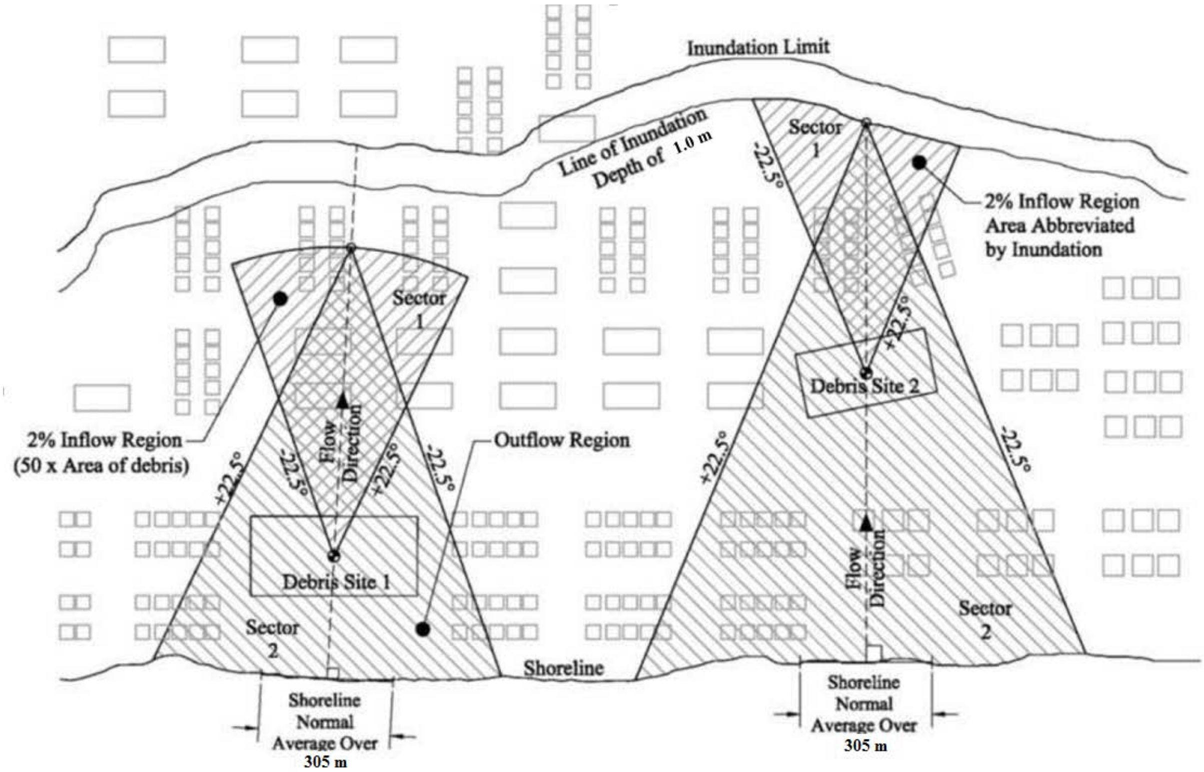
The expected total plan area of the potential debris objects at the source should be determined. For containers, this is the average number of on-site containers multiplied by their plan area. For barges, the area of a nominal AASHTO (2009) design barge (59.5 × 10.67 m, or 635 m^2) should be multiplied by the average number of barges at the source. For ships, the average vessel deck plan area at the site should be used.

The geographic centre of the source should be identified, together with the primary flow direction, as defined in Section 2.4.3.1. Lines ± 22.5° from this centreline should be projected in the direction of tsunami inflow, as shown in Figure 2-2. If topography (such as hills) will bound the water from this 45° sector, the direction of the sector should be rotated to accommodate hill lines or the wedge should be narrowed where it is constrained on two or more sides.

Firstly, an arc of the debris impact hazard region for inflow should be drawn as follows: one arc and the two radial boundary lines of the 45° sector defines a circular sector region with an area that is 50 times the total sum debris area of the source, representing a 2% concentration of debris. However, the inland extent of the arc should be permitted to be curtailed in accordance with any of the following boundaries:

a. The extent of the sector should be permitted to be curtailed where the maximum inundation depth is less than 0.9 m, or in the case of ships where the inundation depth is less than the ballasted draft plus 0.6 m.
b. Structural steel and/or concrete structures should be permitted to be considered to act as an effective grounding depth terminator of the sector if their height is at least equal to (1) for containers and barges, the inundation depth minus 0.6 m, or (2) for ships, the inundation depth minus the sum of the ballasted draft and 0.6 m.

Figure 2-2 Illustration of Determination of Floating Debris Impact Hazard Region (Reproduced with permission from ASCE 7-16)



Second, the debris impact hazard region for inflow and outflow should be determined by rotating the circular segment by 180° and placing the centre at the intersection of the centreline and the arc that defines the 2% concentration level or approved alternative boundary, as defined above.

Buildings and other structures contained only in the first sector should be designed for strikes by a container and/or other vessels carried with the inflow. Buildings and other structures contained only in the second sector should be designed for strikes by a container and/or other vessel carried in the outflow. Buildings and other structures contained in both sectors should be designed for strikes by a container and/or other vessel moving in either direction.

2.7.7 Shipping Containers

The impact force from shipping containers should be calculated from Equations (2.7-2) and (2.7-3) where;

the mass m_d is the mass of the empty shipping container.

It should be assumed that the strike contact is from one bottom corner of the front (or rear) of the container. The container stiffness is to be taken as k = EA/L, in which E is the modulus of elasticity of the bottom rail of the container, A is the cross-sectional area of the bottom rail, and L is the length of the bottom rail of the container.

Minimum values for weight and debris stiffness are provided in Table 2-6.

C_o, the orientation factor, should be taken as equal to 0.65 for shipping containers.

The nominal design impact force, F_{ni}, from Eq. (2.7-2) for shipping containers need not be taken as greater than 980 kN.

For empty shipping containers, the impulse duration for elastic impact should be calculated from Eq. (2.7-4). For loaded shipping containers the duration of the pulse is determined from Eq. (2.7-5):

$$t_d = \frac{(m_d + m_{contents})\, u_{max}}{F_{ni}} \quad (2.7\text{-}5)$$

where, $m_{contents}$ should be taken to be 50% of the maximum rated content capacity of the shipping container. Minimum values of (m_d + $m_{contents}$) are given in Table 2-6 for loaded shipping containers. The design should consider both empty and loaded shipping containers.

For an equivalent static analysis, the impact force should be multiplied by the dynamic response factor R_{max} specified in Table 25. To obtain intermediate values of R_{max}, linear interpolation should be used. For a wall, the impact should be assumed to act along the horizontal centre of the wall, and the natural period should be permitted to be determined based on the period of an equivalent column with width equal to one-half of the vertical span of the wall. It also should be permitted to use an alternative method of analysis per Section 2.7.9.

2.7.8 Extraordinary Debris Impacts

Where the maximum inundation depth exceeds 3.60 m, extraordinary debris impacts should be considered.

Extraordinary debris should be assumed to be the largest deadweight tonnage vessel with ballasted draft less than the inundation depth within the debris hazard region of piers and wharves defined in Section 2.7.6 should be assumed to impact the perimeter of Vertical Evacuation Structure (VES) structures anywhere from the base of the structure up to 1.3 times the inundation depth plus the height to the deck of the vessel. The load should be calculated from Eq. (2.73), based on the stiffness of the impacted structural element and a weight equal to the Lightship Weight (LWT) plus 30% of Deadweight Tonnage (DWT).

An alternative analysis of Section 2.7.9 should be permitted. Either as the primary approach, or where the impact loads exceed acceptability criteria for any structural element subject to impact, it is permitted to accommodate the impact through the alternative load path progressive collapse provisions of Section 3.4.3, applied to all framing levels from the base up to the story level above 1.3 times the inundation depth plus the height to the deck of the vessel as measured from the waterline.

2.7.9 Alternative Methods of Response Analysis

A dynamic analysis is permitted to be used to determine the structural response to the force applied as a rectangular pulse of duration time t_d with the magnitude calculated in accordance with Eq. (2.7-4). If the impact is large enough to cause inelastic behaviour in the structure, it should be permitted to use an equivalent single degree of freedom mass-spring system with a nonlinear stiffness that considers the ductility of the impacted structure for the dynamic analysis. Alternatively, for inelastic impact, the structural response should be permitted to be calculated based on a work-energy method with nonlinear stiffness that incorporates the ductility of the impacted structure. The velocity applied in the work-energy method of analysis should be u_{max} multiplied by a factor equal to 1.25, and the orientation factor, C_o.

Table 2-6 Weight and Stiffness of Shipping Container Waterborne Floating Debris

Type of Debris	Weight	Debris Stiffness (k)
6.1 m standard shipping container oriented longitudinally	Empty: 2,270 kg Loaded: 13,150 kg	42,900 kN/m
12.2 m standard shipping container oriented longitudinally	Empty: 3,810 kg Loaded: 17,240 kg	29,800 kN/m

SECTION 3

Structural Design Procedure

Structures, components, and foundations should conform to the recommendations of this section when subjected to the loads and effects of the Maximum Considered Tsunami. Minimum structural and non-structural performance objectives for Vertical Evacuation Structure (VES) are presented in section 2.1.

3.1 Structural Performance of Tsunami Vertical Evacuation Structures

Structural systems of tsunami Vertical Evacuation Structures (VES), including foundations should be designed to be able to be occupied immediately after the Maximum Considered Tsunami (MCT).

Design of structures to meet the operational continuity requirement of AS/NZS 1170.0 (importance level is equal to 4) is perceived to be sufficient to ensure that the vertical evacuation structure able to be occupied immediately in a scenario including the Serviceability Limit State (SLS2) level earthquake followed by the Maximum Considered Tsunami. In some areas of New Zealand, the MCT generating earthquake might impose greater pre-tsunami seismic actions than the SLS2 level, in which case a higher design threshold for operational continuity needs to be applied.

However, the assessment of designed structure according the recognized assessment guidelines could be considered as an alternative pathway to re-confirm the performance objectives in explicit manner.

3.2 Structural Performance Evaluation

Strength and stability should be evaluated to determine that the structure is capable of resisting the tsunami for the Load Cases defined in Section 3.3.

The structural acceptance criteria for structural performance evaluation should be in accordance with sections 3.4.1, 3.4.2, 3.4.3. In addition, the stiffness and inter-storey drift limits might control the design variables as stringent requirements are needed to meet the Serviceability Limit States.

3.3 Load Cases

As a minimum, the following three Inundation Load Cases should be evaluated:

› **Load Case 1:** At an exterior inundation depth not exceeding the design inundation depth nor the lesser of one story or the height of the top of the first-story windows, the minimum condition of combined hydrodynamic force with buoyant force should be evaluated with respect to the depth of water in the interior. The interior water depth should be evaluated in accordance with Section 2.5.1.

EXCEPTION: Load Case 1 need not be applied to Open Structures nor to structures where the soil properties or foundation and structural design prevent detrimental hydrostatic pressurization on the underside of the foundation and lowest structural slab.

› **Load Case 2:** Depth at two/thirds of the design inundation depth when the maximum velocity and maximum specific momentum flux should be assumed to occur in either incoming or receding directions whichever is critical.

› **Load Case 3:** Depth at the design inundation depth when velocity should be assumed at one-third of maximum in either incoming or receding directions.

The inundation depths and velocities defined for Load Cases 2 and 3 should be determined through the site-specific tsunami analysis performed in accordance with Section 2.3.1.2.

3.3.1 Load Combinations

Not all tsunami loads and effects will occur simultaneously, nor will they all affect a particular structural component at the same time. This section describes combinations of tsunami loads that should be considered for the overall structure and for individual structural components. Other potential combinations should be considered as needed, based on the particular siting, structural system, and design of the structure under consideration.

Tsunami forces are combined on the overall structure as follows:

› Uplift due to buoyancy effect, F_v, and hydrodynamic uplift have the effect of reducing the total dead weight of a structure, which may impact the overturning resistance. Buoyancy and hydrodynamic uplift appropriate for the design inundation level should be considered in all load combinations.
› Debris impact forces are short duration loads. Because of the extremely short duration associated with debris impact loads, ASCE/SEI 7-16 does not require that impact loads be combined with hydrodynamic forces.
› Design of floor systems to withstand the effects of potential retained water pressure, p_r, can be performed independently of the lateral loading on the structure.

The following tsunami forces should be combined and assumed to act concurrently on individual structural components (e.g., columns, walls, and beams):

› Exterior structural elements must be designed to resist hydrodynamic loads associated with Load Cases 2 and 3, including the increased tributary width resulting from debris damming.
› Exterior structural components must also be designed for debris impact. The impact force can be applied as a static load at any location along the submerged component to cause maximum bending moment and maximum shear force in the component. The impact force can also be applied as a short duration dynamic impulse and evaluated dynamically, including component nonlinearity. Finally, the component can also be evaluated using an appropriate energy method. Although it is possible that more than one floating object impact a building during a tsunami event, the probability of two or more impacts occurring simultaneously is considered small. Therefore, only one impact need be considered to occur at any point in time. Debris impact loads need not be combined with hydrodynamic loads on individual components.
› Interior structural components need not consider impact loads associated with waterborne debris because larger floating objects will be trapped, or at least slowed, by the exterior of the building. Interior structural components must be designed for hydrodynamic drag, but need not consider debris damming effects.
› Hydrostatic pressure, Fh, on walls enclosing watertight areas of a structure, for maximum h.

For uplift on floor framing components, the following combinations of tsunami forces should be considered:

› Buoyancy effect, F_v, of submerged floor framing components including the effects of entrapped air and upturned beams or walls, for maximum h.
› Hydrodynamic uplift due to rapidly rising flood waters, for flow velocity at a depth equal to the soffit of the floor system, h_s.
› Maximum uplift case: The larger of the above uplift loads combined with 90% dead load and zero live load on the floor system, for design against uplift effects on the floor slabs, beams, and connections.

For downward load on floor framing components due to retained water, the following force combination should be considered:

› Downward load due to water retained by exterior walls, pr, combined with 100% dead load.

Principal tsunami forces and effects should be combined with other specified loads in accordance with the load combinations of Eq. (3.3-1):

$$[0.9G_{,} F_{TSU}, H_{TSU}] \quad (3.3\text{-}1)\ a$$

$$[1.2G_{,} F_{TSU}, \psi_E Q, L_{refuge}, H_{TSU}] \quad (3.3\text{-}1)\ b$$

where:

F_{TSU} = tsunami load effects for incoming and receding directions of flow,

H_{TSU}= load caused by tsunami-induced lateral foundation pressures developed under the submerged conditions. Where the net effect of H_{TSU} counteracts the principal load effect, the load factor for H_{TSU} should be 0.9,

G= load caused by permanent actions (dead loads),

Q= imposed actions (live loads), and

ψ_E = earthquake combination factor as defined according to AS/NZS 1170.0.

The snow loads need not to be considered in combination with tsunami load cases in New Zealand.

3.4 Acceptance Criteria

Design should be carried out according to latest versions of AS/NZS 1170 series and relevant material standards deemed to comply with the performance objective of this guideline at Ultimate Limit State (ULS) and Serviceability Limit State (SLS2).

Irrespective of any chosen analysis and/or assessment method, capacity design procedures should be used to ensure a ductile global mechanism in vertical evacuation structures.

The explicit assessment of designed structure according to recognized assessment standards could be considered as an alternative pathway to justify the design objectives. Then, the lateral-force-resisting system could be explicitly analysed to evaluate acceptance criteria for the primary structural components.

3.4.1 Acceptance Criteria for Lateral-Force-Resisting System

To evaluate the capacity of the structural system to achieve the immediate occupancy performance objective, the lateral force resisting system should be explicitly analysed and evaluated under the governing seismic actions for Serviceability Limit State (SLS2) requirements. Alternatively, the structure could be designed for fully elastic performance under the Maximum Considered Tsunami (MCT) loads and effects, and then checked for compliance with SLS2 seismic requirements.

Structural components should be designed for the forces that result from the overall tsunami forces on the structural system combined with any resultant actions caused by the tsunami pressures acting locally on the individual structural components for that direction of flow. Acceptance criteria of structural components should be in accordance with Section 3.4.2, or in accordance with alternative procedures of 3.4.3, as applicable.

3.4.2 Acceptability Criteria by Component Design Strength

Internal forces and system displacements should be determined using a linearly elastic, static analysis. The structural performance criteria required in Section 3.1, and Section 3.2, as applicable, should be deemed to comply if the design strength of the structural components and connections are shown to be greater than the Maximum Considered Tsunami loads and effects computed in accordance with the load combinations of Section 3.3.1. Requirements for operational continuity following possible pre-tsunami earthquakes must also be met.

Material reduction factors,ϕ, should be used as prescribed in the material-specific standards for the component and behaviour under consideration.

3.4.3 Alternative Acceptability by Progressive Collapse Avoidance

Where tsunami loads or effects exceed acceptability criteria for a structural element or where required to accommodate extraordinary impact loads in structural components located below the design inundation depth, it should be permitted to check the residual load-carrying capacity of the structure. The checks must assume that the element has been failed and an alternate load path progressive collapse avoidance procedure be carried out as per recognized literature.

Reducing the potential for disproportionate (i.e., progressive) collapse due to the loss of one or more structural components will increase the likelihood that a vertical evacuation structure will remain standing if a column is severely damaged due to waterborne debris.

The decision to include progressive collapse considerations in the design for a particular structure will depend on the site and the nature of the debris that could potentially impact the structure. Because the potential exists for localized severe damage due to debris impact, design for progressive collapse prevention is strongly encouraged.

In the United States, primary design approaches for progressive collapse include measures to implement “tie force”, “enhanced local resistance” and “alternative load path” mitigation measures. For essential facility occupancies, including Vertical Evacuation Structures (VES), the application of all three measures is required. It is strongly recommended to use the alternative load path design technique to span over a missing vertical load carrying column or wall element.

3.5 Structural Design Concepts and Additional Considerations

3.5.1 Attributes of Tsunami-Resistant Structures

Structural system selection and configuration, from foundation to roof framing, can have a significant effect on the ability of a vertical evacuation structure to withstand anticipated tsunami, earthquake, and wind loading. Many common structural systems can be engineered to resist tsunami load effects. Structural attributes that have demonstrated good behaviour in past tsunamis include:

- Strong systems with the reserve strength capacity to resist the extreme forces;
- Open systems that allow water to flow through with minimal resistance;
- Ductile systems that resist extreme forces without brittle failure;
- Redundant systems that can experience partial failure without progressive collapse;

Various lateral force resisting systems can exhibit these attributes. Examples include; reinforced concrete and steel moment frame systems; steel braced frame systems; reinforced concrete shear wall systems and any combination of those systems (dual structural systems).

Moreover, the following general design recommendations are highly desirable to ensure the enhanced performance for the vertical evacuation structures:

- Floor systems in vertical evaluation structures should be designed to resist likely uplift forces in accordance to this guideline. These induced actions might be the critical load case in pre-cast and pre-stressed floor system.
- Regardless of designed ductility demands, the vertical evacuated structures should be detailed to perform in a fully or limited ductile manner as a minimum. In other words, even if the structure is designed to behave elastically under the governing seismic event, the detailing requirement in potential plastic zones of structure should meet the fully or limited ductile requirements of relevant material standards. This recommendation offers inherent resiliency to vertical evacuation structures under seismic events higher than the anticipated governing seismic event.
- Using a base isolation system in a Vertical Evacuation Structures (VES) is not desirable. The use of base isolation systems in tsunami hazard areas needs further research and field observations before they are implemented.

3.5.2 Structural Considerations for Tsunami Loads and Effects

Design of individual columns for tsunami lateral loads should be performed assuming the appropriate degree of fixity at the column base and at each floor level.

Column shape is also important. Round columns will result in lower drag forces than square or rectangular shapes. In addition, waterborne debris will be less likely to directly impact round columns, hence round columns are less likely to be subject maximum design impact forces.

If shear walls are used, the plan orientation of the walls is important. It is recommended that the shear walls be oriented parallel to the anticipated direction of tsunami flow to reduce associated hydrodynamic forces and impact forces from waterborne debris.

Design of reinforced concrete walls for tsunami forces should consider the full load on the wall, including hydrodynamic and debris impact forces, spanning vertically between floor levels. Reinforced concrete beams poured integral with the floor will be braced by the slab. Design of beams for horizontal tsunami forces should take into account the lateral bracing provided by the floor slab. Isolated beams must be designed for shear and bending induced by tsunami loads.

Floor systems and columns must be designed for the effects of buoyancy and hydrodynamic uplift, which will induce shear and bending effects that are opposite to those resulting from gravity loads. Even though lower levels of a vertical evacuation structure are not intended for use during a tsunami, failure could result in damage, loss of lateral restraint or collapse of columns supporting upper levels, including the tsunami refuge area.

In structural steel floor systems, lateral torsional buckling of beam bottom flanges must be considered when subjected to uplift loading. In reinforced concrete floor systems, continuity of reinforcement should be provided in beams and slabs for at least 50% of both the top and bottom reinforcement.

Pre-stressed concrete floor systems must be carefully checked for buoyancy and hydrodynamic uplift effects when submerged. Internal pre-stressing forces used to oppose dead loads may add to these effects. Web elements of typical pre-stressed joist systems are susceptible to compression failure under uplift conditions, and many typical bearing connections are not anchored for potential net uplift forces. Localized damage to the concrete in a pre-stressed floor system can result in a loss of concrete compressive capacity, and release of the internal pre-stressing forces.

3.5.3 Concepts for Modifying and Retrofitting Existing Structures

It may not always be feasible to construct new buildings in an area that requires vertical evacuation refuge. Although retrofitting existing buildings to perform as a vertical evacuation structure could be expensive and disruptive to current users of the building, it may be the most viable option available.

Existing buildings considered for use as vertical evacuation structures should possess the structural attributes listed in Section 2.1 that are associated with tsunami-resistant structures, and should be evaluated for tsunami load effects in accordance with Section 3. In the case of near source-generated tsunamis, existing buildings should also be evaluated for seismic effects.

Because of the importance of vertical evacuation structures, and the need for these facilities to function as a refuge when exposed to extreme tsunami and seismic loading, reduced loading criteria for existing buildings, as is the current state-of-practice for seismic evaluation of existing buildings, is not recommended for evaluation of potential tsunami vertical evacuation structures.

The following concepts can be considered in the modification and retrofit of existing buildings for use as vertical evacuation structures:

› Roof system; Upgrade roof systems to support additional live loads associated with refuge occupancy. Protect or relocate existing building functions at the roof level (e.g., mechanical equipment) that would be a risk or unsafe in the immediate vicinity of high occupancy areas. Modify existing roof parapets for fall protection of refuge occupants.

› Wall system; Consider modifying walls and wall connections in the lower levels of the building to perform as breakaway walls to minimize tsunami hydrostatic, hydrodynamic, and surge forces on the building.

- Access; Modify ingress into the building and improve vertical circulation through the use of new entrances, ramps, and stairs. Consider placing access points on the outside of the building for ease of construction and high visibility.
- Potential Debris; Remove or relocate building ground level functions that may become potential water-borne debris.
- Existing hazards at the site; consider and protect against other hazards that might exist at the building site, including other adjacent buildings that could collapse, and the presence of hazardous or flammable materials near the site.
- The existing building should achieve, as a minimum, the operational continuity requirements following the Serviceability Limit State (SLS2) seismic event as defined in NZS 1170.5. In addition, the objective of NZS 1170.5 under the ultimate limit state conditions must be equally justified by recognized assessment standards.
- Precast floor systems in existing buildings that are most common in New Zealand construction practice are likely to be subjected to uplift forces in floor levels subjected to MCT. Further structural design consideration of precast floors is needed to resist all potential tsunami induced actions.

3.5.4 Breakaway Wall Concepts

Solid enclosure walls below the tsunami inundation level will result in large tsunami loads on the overall building. These walls will also increase the potential for wave scour at grade beams and piles. Non-structural walls below the anticipated tsunami flow depth can be designed as breakaway walls to limit the hydrostatic, buoyancy, hydrodynamic, and impulsive forces on the overall building and individual structural members. Breakaway wall recommendations are described in detail in the FEMA 55 Coastal Construction Manual (FEMA, 2005). Breakaway walls can create wave reflection and run-up prior to failure and hence need careful attention.

Walls, partitions, and connections to the structure that are intended to break away are designed for the largest of the following loads acting perpendicular to the plane of the wall:

- The wind load specified in AS/NZS 1170.2.
- The earthquake load specified in NZS 1170.5.
- 0.50 kN/m^2
- Not more than 1.0 kN/m^2 unless the design meets the following conditions:
  i. breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood; and
  ii. the supporting foundation and the elevated portion of the building is designed to resist collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads.

Standard engineering practice can often result in considerable design overstrength, which would be detrimental to a breakaway wall system and the supporting structure. Care should be taken to avoid introducing unnecessary conservatism into the design. All components, including sheathing, siding, and window frame supports, must be considered in determining the actual strength of the breakaway wall system, and the resulting maximum load on the supporting structure. The most desirable fusing mechanism includes failure of the top and side connections while the bottom connection remains intact, allowing the wall panel to lay down under the tsunami flow, ideally without becoming detached and part of the debris flow.

Notwithstanding all of the above, the need for potential breakaway wall systems to comply with New Zealand Building Code requirements relating to factors such as weather tightness, insulation, fire rating and acoustic performance should be considered in parallel with performance under the tsunami loads.

3.5.4.1 Metal Stud Walls

Metal stud infill walls are commonly used as part of the building envelope.

Unless properly galvanized, metal studs will corrode rapidly in the coastal environment. Recent lateral load testing of typical metal stud wall configurations shows that ultimate failure occurs when the studs separate from either the top or bottom tracks. However, the load required to produce this failure is as much as four times the wind load for which the studs were initially designed.

Where metal stud walls are to be considered for breakaway walls design, it is therefore necessary to introduce some sort of a “fuse” at the top track connection to ensure that the wall fails at a predictable load. Such a fuse might include a reduced stud section at the top of the studs. Testing of fuse mechanisms would be required to verify that they have the capacity needed to resist design loads, but will fail at predictably higher load levels

3.5.4.2 Masonry Walls

Masonry walls are commonly used as enclosures in the lower levels of larger buildings.

Masonry walls to be considered as breakaway walls can be restrained with the use of a dowel pin fuse system around the top and sides of the wall, without bonded contact to the structure. Such a system should be tested to verify that it will fail at predictable load levels that exceed design loads. If properly fused, the masonry wall will cantilever from the foundation and load will no longer be applied to the surrounding structural frame, after failure of the dowel pins. To allow wall failure due to foundation rotation without damage to the remaining structure, separation of the wall foundation from the building foundation should be considered.

3.5.5 Structural Countermeasures for Tsunami Loading

The following countermeasures should be permitted to reduce the structural effects of tsunamis.

3.5.5.1 Open Structures

Open Structures should not be subject to Load Case 1 of Section 3.3. The load effect of debris accumulation against or within the Open Structure should be evaluated by assuming a minimum closure ratio of 50% of the inundated projected area along the perimeter of the Open Structure.

3.5.5.2 Other Measures

Tsunami barriers are another form of structural countermeasure for tsunami loading. These structures are used as external perimeter structures to resist the actions induced by tsunami waves. Design provision of these countermeasure structures is outside of the scope of this guideline. However, the following recommendation might be used for spatial limits of site layout of these structures.

The spatial limits of the layout of tsunami barriers should include the following:

1. The tsunami barrier should be set back from the protected structure for perimeter protection. Any alignment change should have a minimum radius of curvature equal to at least half the design inundation depth.
2. For overtopping or partial impedance to inundation, as minimum, the barrier should protect the structure from inundation flow based on an approach angle of ±22.5 degrees from the shoreline. The flow approach angle should be evaluated in accordance with Sections 2.4.3.1 and 2.4.3.2.

3.6 Foundation Design

The design of structure foundations and tsunami barriers to provide resistance to the loads and effects of Section 3.6.3, should provide capacity to support the structural load combinations defined in Section 3.3, and should accommodate the displacements determined in accordance with Section 3.6.3.6.

Foundation embedment depth and the capacity of the exposed piles to resist the structural loads, (including the grade beam loads) should both be determined taking into account the cumulative effects of general erosion and local scour. Alternatively, it should be permitted to use the performance-based criteria of Section 3.6.4.

3.6.1 Seismic Effects on the Foundations Preceding Local Subduction Zone Maximum Considered Tsunami

Where a site may be subject to a local subduction zone tsunami from an offshore subduction earthquake, the structure should be designed for the preceding co-seismic effects. The foundation of the structure should be designed to resist the preceding earthquake ground motion and associated effects.

Building foundation design should allow for changes in the site surface and the in situ soil properties resulting from the design seismic event as initial conditions for the subsequent design tsunami event. Geotechnical investigation reporting should include evaluation of foundation effects in reference to seismic effects preceding the tsunami; consideration of slope instability, liquefaction, total and differential settlement, and surface displacement caused by faulting, and seismically induced lateral spreading or lateral flow. The additional recommendations of Section 3.6 should also be evaluated.

3.6.2 Resistance Factors for Foundation Stability Analyses

The resistance factor of ϕ should be assigned a value of 0.67 applied to the resisting capacities used with stability analyses and for potential failures associated with bearing capacity, lateral pressure, internal stability of geotextile and reinforced earth systems, and slope stability, including drawdown conditions. A resistance factor of 0.67 should also be assigned for the resisting capacities of uplift resisting anchorage elements.

3.6.3 Tsunami Load and Effect Characterization

Foundations and tsunami barriers should be designed to accommodate the effects of lateral earth pressure in accordance with Section 3.2, hydrostatic forces computed in accordance with Section 2.5, hydrodynamic loads computed in accordance with Section 2.6, and uplift and under-seepage forces computed in accordance with Section 3.6.3.1.

Foundations should provide the capacity to withstand uplift and overturning from tsunami hydrostatic, hydrodynamic, and debris loads applied to the building superstructure. In addition, the effect of soil strength loss, general erosion, and scour should be considered in accordance with the recommendations of this section. A minimum of two wave cycles should be considered for such effects.

3.6.3.1. Uplift and Under-seepage Forces

Tsunami uplift and under-seepage forces should be evaluated as described in this section.

1) Uplift and under-seepage forces should include the three inundation Load Cases defined in Section 3.3.
   a) Strength loss caused by scour and other soil effects such as liquefaction and pore pressure softening should be considered. Additionally, uplift and under-seepage forces on the foundation should be determined for cases where
      – The soil is assumed to be saturated before the tsunami, or
      – Soil saturation is anticipated to occur over the course of the incoming series of tsunami waves, or
      – The area of concern is expected to remain inundated after the tsunami.
2) The effect of live load and snow load should not be used for uplift resistance.

3.6.3.2 Loss of Strength

Loss of shear strength because of tsunami-induced pore pressure softening should be accounted for up to a depth of 1.2 times the design inundation depth. Tsunami-induced pore pressure softening need not be considered at locations where the maximum Froude number is less than 0.5.

3.6.3.3 General Erosion

General erosion during tsunami inundation run-up and drawdown conditions should be considered. Analysis of general erosion should account for flow amplification as described in Section 2.4.2; it should also account for enhancement caused by tsunami-induced pore pressure softening.

EXCEPTION: Analysis of general erosion is not required for rock or other non-erodible strata that are capable of preventing scour from a tsunami flow of 9.0 m/s. General erosions during drawdown conditions should consider flow concentration in channels, including channels newly formed during tsunami inundation and drawdown (channelized scour). Analysis of channelized scour need not include enhancement caused by pore pressure softening.

3.6.3.4 Scour

The depth and extent of scour should be evaluated using the methods of Sections 3.6.3.4.1 and 3.6.3.4.2.

EXCEPTION: Scour evaluation is not required for rock or other non-erodible strata that prevent scour from tsunami flow of 9.0 m/s nor for open structures.

3.6.3.4.1 Sustained Flow Scour

Scour, including the effects of sustained flow around structures and including building corner piles, should be considered.

The sustained flow scour design depth and area extent should be determined by dynamic numerical or physical modelling or empirical methods in the recognized literature. It should be permitted to determine sustained flow scour and associated pore pressure softening in accordance with Table 3-1.

Table 3-1 Design Scour Depth Caused by Sustaining Flow and Pore Pressure Softening

Inundation Depth h	Scour Depth D[a]
< 3.0 m	1.2 h
≥ 3.0 m	3.7 m

[a] Not applicable to scour at sites with intact rock strata.

Local scour depth caused by sustained flow given by Table 3-1 should be permitted to be reduced by an adjustment factor in areas where the maximum flow Froude number is less than 0.5. The adjustment factor should be taken as varying linearly from 0 at the horizontal inundation limit to 1.0 at the point where the Froude number is 0.5. The assumed area limits should be considered to encompass the exposed building perimeter and to extend either side of the foundation perimeter to a distance equal to the scour depth for consolidated or cohesive soils and a distance equal to three times the scour depth for nonconsolidated or non-cohesive soils.

3.6.3.4.2 Plunging Scour

Plunging scour horizontal extent and depth should be determined by dynamic numerical or physical modelling or by empirical methods. In the absence of site-specific dynamic modelling and analysis, the plunging scour depth Ds should be determined by Eq. (3.6-1).

$$D_s = c_{2v}\sqrt{\frac{qU\sin\psi}{g}} \tag{3.6-1}$$

where;

c_{2v} = Dimensionless scour coefficient, permitted to be taken as equal to 2.8;

ψ = Angle between the jet at the scour hole and the horizontal, taken as the lesser value of 75° and the side slope of the overtopped structure on the scoured side, in the absence of other information;

g = Acceleration caused by gravity;

q = Discharge per unit width over the overtopped structure, as illustrated in Figure 3-1 and calculated in accordance with Eq. (3.62); and

U = Jet velocity approaching the scour hole, obtained in accordance with Eq. (3.6-4).

$$q = C_{dis}\,⅔\sqrt{2g}H_o^{3/2} \tag{3.6-2}$$

where C_{dis} is a dimensionless discharge coefficient obtained in accordance with Eq. (3.6-3):

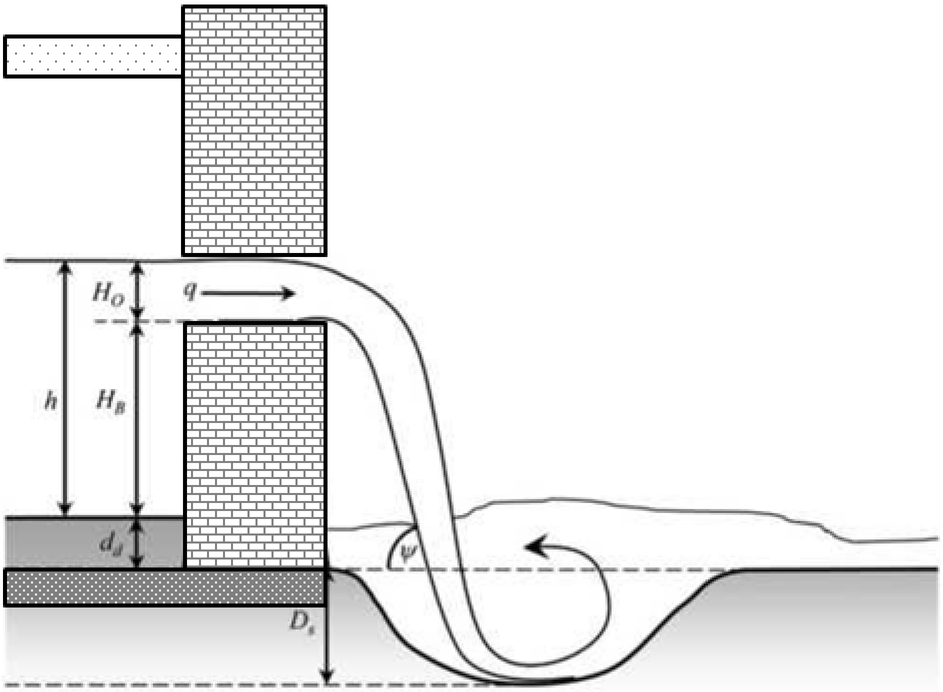
$$C_{dis} = 0.611 + 0.08\frac{H_o}{H_B} \tag{3.6-3}$$

U is the jet velocity approaching the scour hole, resulting from the drop between the height h of the upstream water surface, plus any additional elevation difference d_d on the scouring side, in accordance with Eq. (3.6-4):

$$U = \sqrt{2g(h + d_d)} \tag{3.6-4}$$

where d_d is the additional elevation difference between the upstream and scouring sides of the structure, as illustrated in Figure 3-1.

Figure 3-1 Plunging Scour Parameters



3.6.3.5 Horizontal Soil Loads

Horizontal soil loads caused by unbalanced scour should be included in the design of foundation elements.

3.6.3.6 Displacements

Vertical and horizontal displacements of foundation elements and slope displacements should be determined using empirical or elastoplastic analytical or numerical methods in the recognized literature by applying tsunami loads determined in Section 3.6.3 together with other applicable geotechnical and foundation loads required by this standard.

3.6.4 Alternative Foundation Performance-Based Design Criteria

In situ soil stresses from tsunami loads and effects should be included in the calculation of foundation pressures. For local co-seismic tsunami hazards that occur as a result of a local earthquake, the in situ soil and site surface condition at the onset of tsunami loads should be those existing at the end of seismic shaking, including liquefaction, lateral spread, and fault rupture effects.

Building foundations should provide sufficient capacity and stability to resist structural loads and the effects of general erosion and scour in accordance with the recognized literature. It should be permitted to evaluate the overall performance of the foundation system for potential pore pressure softening by performing a two- or three-dimensional tsunami–soil–structure interaction numerical modelling analysis. The results should be evaluated to demonstrate consistency with the structural performance acceptance criteria in Section 3.

3.6.5 Foundation/Scour Design Concepts

Scour around shallow foundations can lead to failure of the supported structural element. Pile foundations can be designed to avoid this failure. However, they must be able to resist all applied loads without lateral ground support after scouring has exposed the pile cap and piles.

3.6.6 Foundation Countermeasures

Fill, protective slab on grade, geotextiles and reinforced earth systems, facing systems, and ground improvement should be permitted to reduce the effects of tsunamis.

3.6.6.1 Fill

Fill used for structural support and protection should be placed in accordance with best international practice such as ASCE 24 (2005), Sections 1.5.4 and 2.4.1. Structural fill should be designed to be stable during inundation and to resist the loads and effect specified in Section 3.6.3.

3.6.6.2 Protective Slab on Grade

Exterior slabs on grade should be assumed to be uplifted and displaced during the Maximum Considered Tsunami unless determined otherwise by site-specific design analysis based upon recognized literature. Protective slabs on grade used as a countermeasure should at a minimum have the strength necessary to resist the following loads:

1. Shear forces from sustained flow at maximum tsunami flow velocity, umax, over the slab on grade;
2. Uplift pressures from flow acceleration at upstream and downstream slab edges for both inflow and return flow;
3. Seepage flow gradients under the slab if the potential exists for soil saturation during successive tsunami waves;
4. Pressure fluctuations over slab sections and at joints;
5. Pore pressure increases from liquefaction and from the passage of several tsunami waves; and
6. Erosion of substrate at upstream, downstream, and flow parallel slab edges, as well as between slab sections.

3.6.6.3 Geotextiles and Reinforced Earth Systems

Geotextiles should be designed and installed in accordance with manufacturers' installation recommendations and as recommended in the recognized literature. Resistance factors required in Section 3.6.2 should be provided for bearing capacity, uplift, lateral pressure, internal stability, and slope stability. The following reinforced earth systems should be permitted to be used:

1. Geotextile tubes constructed of high-strength fabrics capable of achieving full tensile strength without constricting deformations when subject to the design tsunami loads and effects;
2. Geogrid earth and slope reinforcement systems that include adequate protection against general erosion and scour, and have a maximum lift thickness of 0.3 m and facing protection; and
3. Geo-cell earth and slope reinforcement erosion protection system designs, including an analysis to determine anticipated performance against general erosion and scour if no facing is used.

3.6.6.4 Facing Systems

Facing systems and their anchorage should be sufficiently strong to resist uplift and displacement during design load inundation. The following facing methods for reinforced earth systems should be permitted to be used:

1. Vegetative facing for general erosion and scour resistance where tsunami flow velocities are less than 3.80 m/s. Design should be in accordance with methods and recommendations in the recognized literature.
2. Geotextile filter layers, including primary filter protection of countermeasures using a composite grid assuming high contact stresses and high-energy wave action design criteria in AASHTO M288-06, including soil retention, permeability, clogging resistance, and survivability.
3. Mattresses providing adequate flexibility and including energy dissipation characteristics. Edges should be embedded to maintain edge stability under design inundation flows.

4. Concrete facing provided in accordance with protective slab on grade countermeasures in Section 3.6.3.4.2 and containing adequate anchorage to the reinforced earth system under design inundation flows.
5. Stone armouring and riprap provided to withstand tsunami loads should be designed as follows: Stone diameter should not be less than the size determined according to design criteria based on tsunami design inundation depth and currents using design criteria in the recognized literature. Where the maximum Froude number, F_r, is 0.5 or greater, the high-velocity turbulent flows associated with tsunamis should be specifically considered, using methods in the recognized literature. Subject to independent review, it should be permitted to base designs on physical or numerical modelling.

3.6.6.5 Ground Improvement

Ground improvement countermeasures should be designed using soil–cement mixing to provide non-erodible scour protection per Section 3.6.3.4 and as a minimum, provide soil–cement mass strength reinforcement equal to 0.70 MPa on average unconfined compressive strength. Other alternative solutions can also be used to improve the ground.

SECTION 4

References

References

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