Criteria for Tsunami Design of Buildings and Other Structures

Department of Earthquake Engineering
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CRITERIA FOR TSUNAMI DESIGN OF BUILDINGS AND OTHER STRUCTURES

PREFACE

This document has been prepared by the Department of Earthquake Engineering at NED University of Engineering and Technology, Pakistan as part of a research project entitled Strengthening Tsunami and Earthquake Preparedness in Coastal Areas of Pakistan. The project was funded by The United Nations Development Programme (UNDP). A team of the following expert faculty members participated in this project: (1) Prof Muhammad Masood Rafi; (2) Prof Sarosh Lodi; (3) Dr Haider Hasan; (4) Dr Aslam Faqeer Muhammad; (5) Dr Hira Lodhi; (6) Mr Adnan Rais; (7) Mr Muhammad Ahmed; (8) Mr Shoaib Ahmed; (9) Mr Muhammad Saad Khan. The purpose of this document is to provide guidelines for tsunami design and assessment of structures in the coastal regions of Pakistan. The technical procedures and contents provided in this document are targeted for engineers, architects, building officials, building and property owners, and infrastructure operators for buildings and structures located in the tsunami design zones. The need for this document is based primarily for life safety by identifying tsunami safe and refuge structures for safeguard against a tsunami event. The document is also aimed to protect existing infrastructure from excessive loss during a tsunami event, and to develop a tsunami resistant and sustainable built environment in Pakistan.

The design and assessment procedures described in this document are adapted from Chapter 6 of American Society of Civil Engineers (ASCE) standard for *Minimum design loads and associated criteria for buildings and other structures* (ASCE/SEI 7-16). Tsunami hazard and loading quantification procedures specific for Pakistan have been carried out using computer simulations and are incorporated in this document for national application only. Tsunami performance and assessment procedures for tsunami effects on structures and structural components provided in ASCE/SEI 7-16 were observed to have general application, and have been incorporated in their original form in this document. The contribution made by all the team members is also gratefully acknowledged.

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CONTENTS

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1 INTRODUCTION

1.1 Background

Historically, primary sources of tsunami initiation are based on high magnitude (M_w> 7.0) shallow earthquakes generated from slips / thrusts in subduction zones. The subduction of the Arabian plate into the Eurasian plate creates the 900 km long Makran Subduction Zone (MSZ) (Figure 1.1), extending from the Strait of Hormoz (Iran) to near Karachi (Pakistan) (Haiderzadeh et al 2008). The Makran Subduction Zone forms the main tsunamigenic source for Pakistan and the Arabian Sea region, due to its potential of generating high magnitude earthquakes (upto M_w 9.0). The last reported major tsunami affecting the Pakistani coastline dates back to 1945, from news clippings and eye-witness accounts. Inferences of other tsunami events in the MSZ region have been made in literature, through historical accounts dating back till 326 B.C. (Table 1.1) (Haiderzadeh et al. 2008), highlighting the underlying tsunami hazard from MSZ.

In the aftermath of the 2004 Indian Ocean Tsunami, a global initiative was led by the UNESCO - Intergovernmental Oceanic Commission (IOC) to develop and harmonize tsunami ready capacities and practices for tsunami prone countries in the Indian Ocean region. The Intergovernmental Coordination Group for the Indian Ocean Tsunami Warning and Mitigation System (ICG/IOTWMS) comprises 28 member states, which includes Pakistan along with other countries. This initiative forms the basis for Pakistan to assess the underlying tsunami hazard and develop provisions for a safer built environment in the coastal cities.

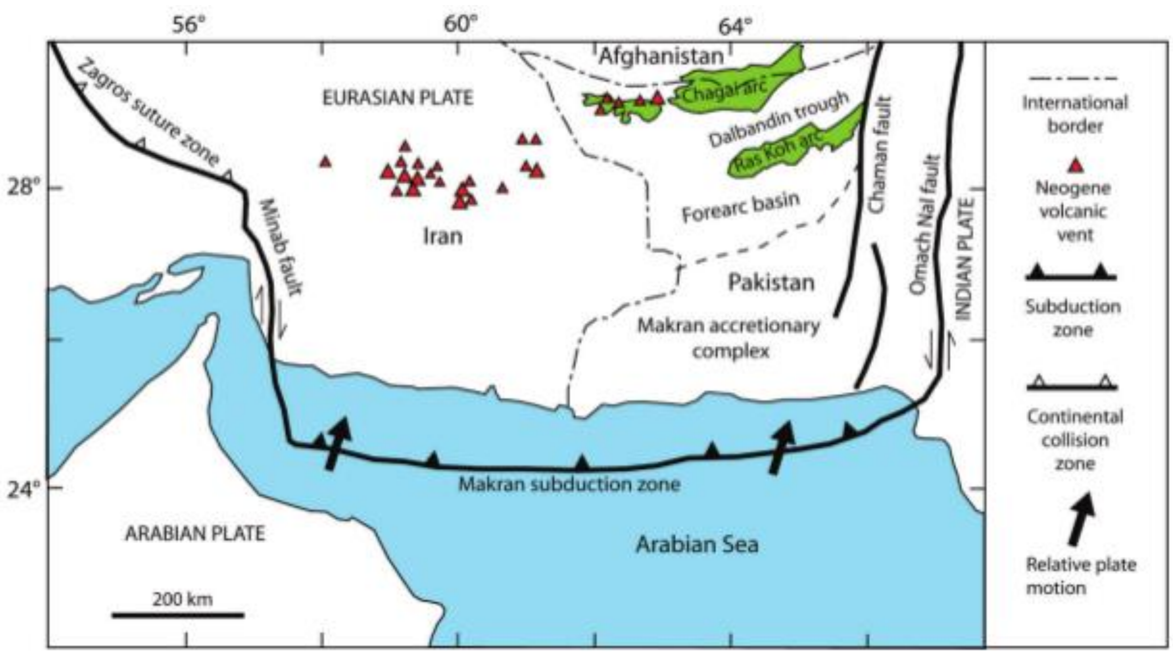

Figure 1.1 - Makran Subduction Zone and inter-plate boundaries. (Peters et. al. 2011)

Table 1.1 - Historical Tsunami in MSZ region (Haiderzadeh et al. 2008)

Year	Longitude (°E)	Latitude (°N)	Source	Magnitude (M_w)	Lives Lost	Confidence	Remarks
326 BC	67.30	24.00	Earthquake	N/A	N/A	1	Macedonian fleet destroyed
1008	60.00	25.00	Earthquake	N/A	1000	2	Large waves drowning ships
1524	N/A	N/A	Earthquake	N/A	N/A	1	Tsunami in Dabhul reported by Portuguese fleet
1897	62.30	25.00	Volcano	-	N/A	1	Fish washed up on Makran coasts
1945	63.00	24.50	Earthquake	8.1	4000	3 (max)	Second deadliest Tsunami in Indian Ocean

1.2 Purpose and Overview

This document provides technical information for design and assessment of structures against tsunami loads and effects. The contents and procedures provided in this document are targeted for engineers, architects, building officials, building and property owners, and infrastructure operators for buildings and structures located in the tsunami design zones along the coastal cities of Pakistan. The need for this document is based primarily for life safety by identifying tsunami safe and refuge structures for safeguard against a tsunami event. The document is also aimed to preserve regional infrastructure from excessive loss during a tsunami event, by employing use of mentioned procedures to develop a tsunami resistant and sustainable built environment.

The flow chart in Figure 1.2 provides an overview of the procedures for estimating tsunami loads and effects on a structure. These procedures also refer the relevant sections of the document to facilitate the users.

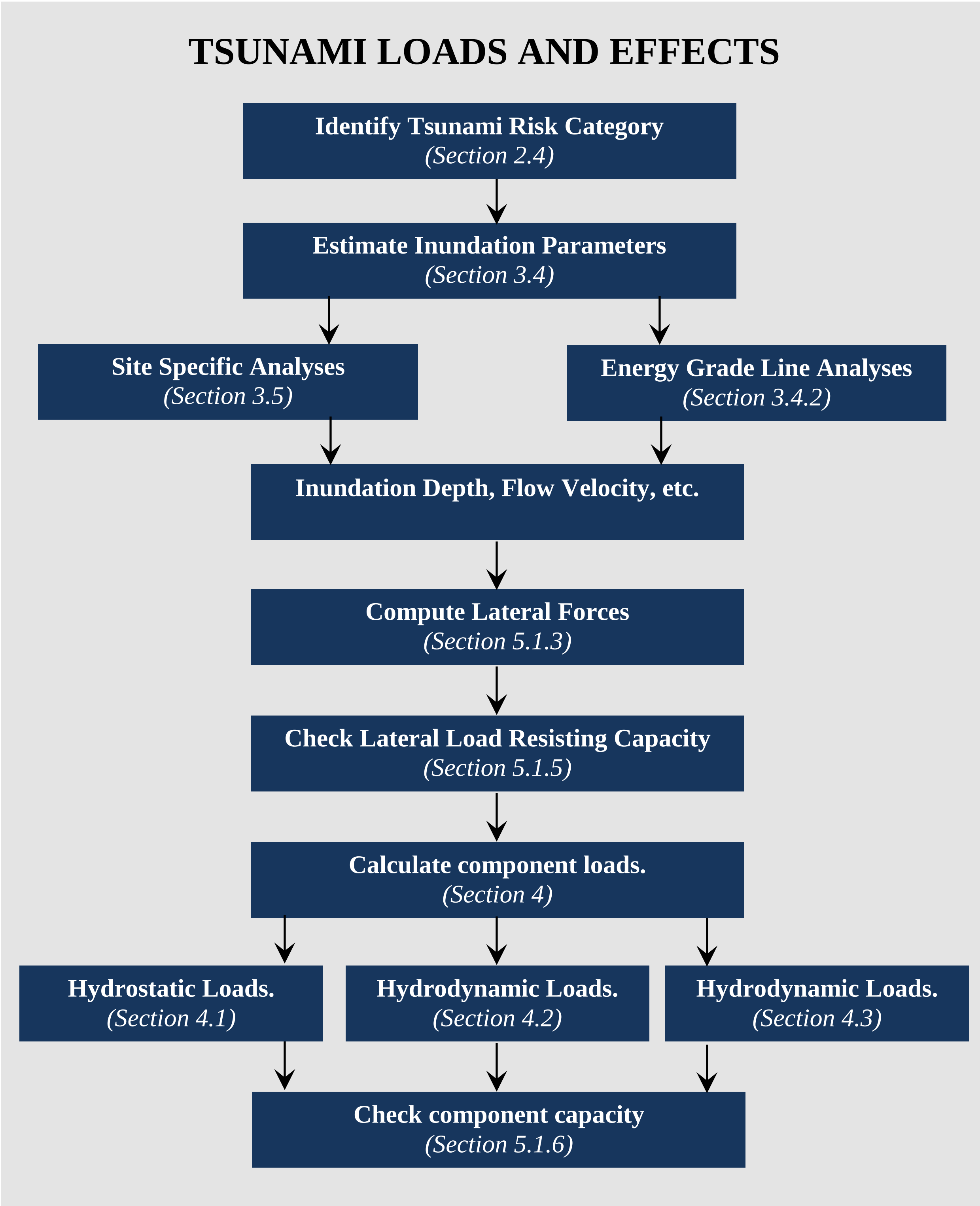

Figure 1.2 - Tsunami design procedures for buildings and other structures

2 GENERAL REQUIREMENTS

2.1 Scope and Requirements

The following buildings and other structures located within the Tsunami Design Zone (section 3.2) in coastal regions of Pakistan shall be designed for the effects and loads of Maximum Considered Tsunami (MCT), in accordance with this chapter:

1. Tsunami Risk Category IV buildings and structures;

2. Tsunami Risk Category III buildings and structures with inundation depth greater than 0.914 m (3 ft), and

3. Where required by a national or local adopted building code statute to include design for tsunami effects, Tsunami Risk Category II buildings with mean height above grade plane greater than the height designated in the statute and having inundation depth greater than 0.914 m (3 ft).

Exception: *Tsunami Risk Category II single- story structures of any height without mezzanines or any occupiable roof level and not having any critical equipment or systems need not be designed for the tsunami loads and effects specified here.*

Designated nonstructural components and systems associated with Tsunami Risk Category III Critical Facilities and Tsunami Risk Category IV structures shall be located above, protected from, or otherwise designed for inundation in accordance with section 5.1 so that they are able to provide their essential functions immediately following the MCT event.

2.2 Symbols and Notations

A_{beam}	=	vertical projected area of an individual beam element
A_{col}	=	vertical projected area of an individual column element.
A_d	=	vertical projected area of obstructing debris accumulated on structure
A_{wall}	=	vertical projected area of an individual wall element
b	=	width subject 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	=	dead load

D_a = diameter of rock armor

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 = earthquake load

E_g = hydraulic head in the Energy Grade Line Analysis

E_m = horizontal Seismic Load Effect, including over-strength factor, defined in section 5.1.5

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 = $u / \sqrt{(gh)}$

F_{TSU} = Tsunami load or effect

f_{uw} = equivalent uniform lateral force per unit width

F_v = buoyancy force

F_w = load on wall or pier

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

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_e = inundated height of an individual element

h_i = inundation depth at point (i)

h_{max} = maximum inundation depth above grade plane at the structure

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

I_{tsu} = importance Factor for Tsunami forces to account for additional uncertainty in estimated parameters

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 that are not considered in section 5.1.7

L = live load

L_{refuge} = public assembly live load effect in the Tsunami refuge floor area

l_w = length of a structural wall

LWT = lightship Weight of vessel

m = component demand modification factor accounting for expected ductility, applied to the expected strength of a ductility-governed element action, to obtain the acceptable structural component capacity at a particular performance level when using a linear static analysis procedure

$m_{contents}$ = 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

q = discharge per unit width over an overtopped structure

Q_{CE} = expected strength of the structural element

Q_{CS} = specified strength of the structural element

Q_{UD} = ductility-governed force caused by gravity and Tsunami loading

Q_{UF} = maximum force generated in the element caused by gravity and Tsunami loading

R = mapped Tsunami runup elevation

R_{max} = dynamic response ratio

R_s = net upward resistance from foundation elements

s = friction slope of the energy grade line

S = snow load

t = time

t_d = duration of debris impact

TDZ = Tsunami Design Zone

t_o = offset time of the wave train

u = Tsunami flow velocity

U = jet velocity of plunging flow

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

x = horizontal distance inland from shoreline

x_R = mapped inundation limit distance inland from shoreline

z = ground elevation above datum

α = Froude number coefficient in the Energy Grade Line Analysis

β = effective wake angle downstream of an obstructing structure to the structure of interest

γ_s = minimum fluid weight density for design hydrostatic loads

γ_{sw} = effective weight density of seawater

Δ_{xi} = incremental distance used in the Energy Grade Line Analysis

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

ϕ = structural resistance 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

Ω_O = over-strength factor for the lateral-force-resisting system.

2.3 Definitions

Bathymetric Profile is a cross section showing ocean depth plotted as a function of horizontal distance from a reference point (such as a coastline).

Channelized Scour is scour that results from broad flow that is diverted to a focused area such as return flow in a preexisting stream channel or alongside a seawall.

Closure Ratio (of Inundated Projected Area) is the 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.

Collapse Prevention Structural Performance Level is a post event damage state in which a structure has damaged components and continues to support gravity loads but retains little or no margin against collapse.

Critical Equipment or Critical Systems includes nonstructural components designated essential for the functionality of the critical facility or essential facility or that are necessary to maintain safe containment of hazardous materials.

Critical Facility includes buildings and structures that provide services that are designated by federal, state, local, or tribal governments to be essential for the implementation of the response and recovery management plan or for the continued functioning of a community, such as facilities for power, fuel, water, communications, public health, major transportation infrastructure, and essential government operations. Critical facilities also comprise of all public and private facilities deemed by a community to be essential for the delivery of vital services, protection of special populations, and the provision of other services of importance for that community.

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 is the nominal strength multiplied by a resistance factor (ϕ).

Design Tsunami Parameters consist of inundation depths and flow velocities at the stages of inflow and outflow most critical to the structure and momentum flux.

Designated Nonstructural Components and Systems comprise of components and systems that are assigned a component Importance Factor (I_p) equal to 1.5.

Ductility-Governed Action comprise of any action on a structural component characterized by post-elastic force versus deformation curve that has sufficient ductility and results from an impulsive short-term force that is not sustained.

Force-Sustained Actions comprise of any action on a structural component characterized by a sustained force or a post-elastic force versus deformation curve that is not ductility governed due to lack of sufficient ductility.

Froude Number, Fr is a dimensionless number defined by $u/\sqrt{(gh)}$, where (u) is the flow velocity averaged over the cross section perpendicular to the flow, which is used to quantify the normalized tsunami flow velocity as a function of water depth.

General Erosion is general wearing away and erosion of land surface over a significant portion of inundation area, excluding localized scour actions.

Grade Plane is the horizontal reference plane at the site representing the average elevation of 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 1.83 m (6 ft) from the structure, between the structure and points 1.83 m (6 ft) from the structure.

Hazard-Consistent Tsunami Scenario comprise of one or more surrogate tsunami scenarios generated from the principal disaggregated seismic source.

Hydrodynamic Loads comprise of loads imposed on an object by water flowing against and around it.

Hydrostatic Loads comprise of loads imposed on an object by a standing mass of water.

Immediate Occupancy Structural Performance Level is a post event damage state in which a structure remains safe to occupy.

Impact Loads comprise of loads that result from debris or other object transported by the design tsunami striking a structure or portion thereof.

Inundation Depth is the depth of design tsunami water level, including relative sea level change, with respect to the grade plane at the structure.

Inundation Elevation is the elevation of the design tsunami water surface, including relative sea level change, with respect to vertical datum.

Inundation Limit is the maximum horizontal inland extent of flooding 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 datum elevation is zero.

Life Safety Structural Performance Level is a post event damage state is that in which a structure has damaged components but retains a margin against onset of partial or total collapse.

Liquefaction Scour is the limiting case of pore pressure softening associated with hydrodynamic flow, where the effective stress drops to zero. In non-cohesive soils, the shear stress required to initiate sediment transport also drops to zero during liquefaction scour.

Local Coseismic Tsunami is the tsunami preceded by an earthquake with damaging effects felt within the subsequently inundated area.

Local Scour is the removal of material from a localized portion of land surface, resulting from flow around, over, or under a structure or structural element.

Maximum Considered Tsunami is a deterministic tsunami envelope of potential mega-thrust seismogenic scenarios of the Makran Subduction Zone (MSZ).

Momentum Flux is 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.

Nonbuilding Critical Facility Structure comprise of non-building structure whose Tsunami Risk Category is designated as either III or IV.

Nonbuilding Structure comprise of structures other than a building.

Open Structure comprise of structures in which the portion within the inundation depth has a closure ratio lesser than 20%, and in which the closure does not include any Tsunami Breakaway Walls, and which does not have interior partitions or contents that are prevented from passing through and exiting the structure as unimpeded waterborne debris.

Pile Scour is a special case of enhanced local scour that occurs at a pile, bridge pier, or similar slender structure.

Plunging Scour is a special case of enhanced local scour that occurs when the flow passes over a complete or nearly complete obstruction, such as a barrier wall, and drops steeply onto the ground below, scouring out a depression.

Pore Pressure Softening is a mechanism that enhances scour through increased pore-water pressure generated within the ground during rapid tsunami loading and the release of that pressure during drawdown.

Primary Structural Component comprise of structural components required to resist tsunami forces and actions and inundated structural components of the gravity-load-carrying system.

Recognized Literature comprise of published research findings and technical papers that are approved by the Authority Having Jurisdiction.

Reference Sea Level is the sea level datum used in site-specific inundation modeling that is typically taken to be Mean High Water Level (MHWL).

Relative Sea Level Change is the local change in the level of the ocean relative to the land, which might be caused by ocean rise and/or subsidence of the land.

Runup Elevation is the ground elevation at the maximum tsunami inundation limit, including relative sea level change, with respect to the datum.

Shoaling is the increase in wave height and wave steepness caused by the decrease in water depth as a wave travels into shallower water.

Soliton Fission comprise of 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 comprise of components of a building that provide gravity-load-carrying or lateral-force resistance as part of a continuous load path to foundation, including beams, columns, slabs, braces, walls, wall piers, coupling beams, and connections.

Structural Wall comprises of walls that provide gravity-load carrying support or one that is designed to provide lateral-force resistance.

Surge refers to rapidly rising water level resulting in horizontal flow inland.

Sustained Flow Scour is the enhanced local scour that results from flow acceleration around a structure. The flow acceleration and associated vortices increase the bottom shear stress and scour out a localized depression.

Toe Scour is a special case of enhanced local scour that occurs at the base of a seawall or similar structure on the side directly exposed to the flow. Toe scour can occur whether or not the structure is overtopped.

Topographic Transect is the 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 is a series of waves with variable long periods, typically resulting from earthquake-induced uplift or subsidence of the seafloor.

Tsunami Amplitude is 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 is 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 (section 3.4.5). Soliton fission in the Nearshore Profile can often lead to the occurrence of tsunami bores.

Tsunami Bore Height is the height of a broken tsunami surge above the water level in front of the bore or grade elevation if the bore arrives on nominally dry land.

Tsunami Breakaway Wall comprises of any type of wall subject to flooding that is not required to provide structural support to a building or other structure and that is designed and constructed such that, before the development of the design flow conditions of Inundation Load Case 1, as defined in section 5.1.3, the wall will collapse or detach in such a way that (1) it allows substantially free passage of floodwaters and external or internal waterborne debris, including unattached building contents and (2) it does not damage the structure or supporting foundation system.

Tsunami Design Zone is the area identified on the Tsunami Design Zone Map between the shoreline and the inundation limit, within which structures are analyzed and designed for inundation by the Maximum Considered Tsunami.

Tsunami Design Zone Map are the maps given in section 3.2 designating the potential horizontal inundation limit of the Maximum Considered Tsunami, or a state or local jurisdiction's deterministic map produced in accordance with section 3.5.

Tsunami Evacuation Map is the evacuation map based on a tsunami inundation map, that are developed from assumed scenarios and provided to a community by the applicable state agency. Tsunami inundation maps for evacuation may be significantly different in extent than the Tsunami Design Zone, and Tsunami Evacuation Maps are not intended for design or land use purposes.

Tsunami-Prone Region comprise of coastal regions addressed in this document with runup greater than 0.914 m (3 ft) caused by tsunamigenic earthquakes in accordance with the Tsunami Hazard Analysis method given in section 3.5.

Tsunami Risk Category is the Risk Category from section 2.4.

Tsunami Vertical Evacuation Refuge Structure is a structure designated and designed to serve as a point of refuge to which a portion of the community's population can evacuate above a tsunami when high ground is not available.

2.4 Tsunami Risk Categories

2.4.1 General Risk Categories

For the purposes of this document, buildings and other structures shall be generally classified, based on the risk to human life, health, and welfare associated with their damage or failure by nature of their occupancy or use, according to Table 2.1. Each building or other structure shall be assigned to the highest applicable Risk Category or Categories.

2.4.2 Tsunami Design Modifications

For the purposes of tsunami load design, buildings and other structures shall be classified according to Risk Categories given in Table 2.1, with following modifications:

1. Federal, provincial, or local governments shall be permitted to include Critical Facilities in Tsunami Risk Category III, such as power-generating stations, water-treatment facilities for potable water, wastewater-treatment facilities, and other public utility facilities not included in Risk Category IV.

2. State or local governments shall be permitted to include Emergency Response Centers as Tsunami Risk Category II or III, such as:
   a. Fire stations, ambulance facilities, and emergency vehicle garages;
   b. Other natural disaster shelters;
   c. Emergency aircraft hangars; and
   d. Police stations that are not required for post-disaster emergency response.

3. Tsunami Vertical Evacuation Refuge Structures shall be included in Tsunami Risk Category IV.

2.5 Countermeasures

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

Table 2.1 - Risk Category of Buildings and Structures for Tsunami General Loads.

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure.	**I**
All buildings and other structures except those listed in Risk Categories I, III, and IV.	**II**
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released.[1]	**III**
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released.[1] Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	**IV**

[1]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment that a release of the substances is commensurate with the risk associated with that Risk Category.

2.5.1 Open Structures

Open Structures shall not be subject to Load Case 1 of section 5.1.3. The load effect of debris accumulation against or within the Open Structure shall be evaluated by assuming a minimum closure ratio of 50% of the inundated projected area along the perimeter of the Open Structure.

2.5.2 *Tsunami Barriers*

Tsunami barriers used as an external perimeter structural countermeasure shall be designed consistent with the protected structure performance objectives to jointly achieve the performance criteria. These criteria include barrier strength, stability, slope erosion protection, toe scour, and geotechnical stability requirements and barrier height and footprint to fully prevent inundation during the MCT. Where a barrier is designed to be overtopped by the design event or intended to provide only partial impedance of the design event, the protected structure and its foundation shall be designed for the residual inundation resulting from the design event. The foundation countermeasures shall also be applied.

2.5.4 *Site Layout*

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

1. The tsunami barrier shall be set back from the protected structure for perimeter protection. Any alignment change shall have a minimum radius of curvature equal to at least half the maximum inundation depth.

2. For overtopping or partial impedance to inundation, at a minimum the barrier limits shall protect the structure from inundation flow based on an approach angle of ±22.5 degrees from the shoreline. The flow approach angle shall be evaluated in accordance with sections 5.1.9.

2.5.5 *Foundation Countermeasures*

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

2.5.5.1 *Structural Fill*

Structural fill shall be designed to be stable during inundation and to resist the loads and effects specified in section 5.2.

2.5.5.2 *Protective slab on grade*

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

1. Shear forces from sustained flow at maximum tsunami flow velocity (u_{max}) 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.

2.5.5.3 Ground / Soil improvement systems

Ground improvement countermeasures shall be designed using soil–cement mixing to provide non-erodible scour protection per section 5.2 and at minimum provide soil–cement mass strength reinforcement of 0.69 MPa (100 psi) average unconfined compressive strength.

Geotextiles shall be designed and installed in accordance with manufacturers' installation requirements and as recommended in recognized literature. Resistance factors required in section 5.2.1 shall be provided for bearing capacity, uplift, lateral pressure, internal stability, and slope stability. The following reinforced earth systems shall 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 a maximum lift thickness of 0.3 m (1 ft) and facing protection; and

3. Geocell 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.

2.5.5.4 Facing systems

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

1. Vegetative facing for general erosion and scour resistance where tsunami flow velocities are less than 3.81 m/s (12.5 ft/s). Design shall be in accordance with methods and requirements 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, including soil retention, permeability, clogging resistance, and survivability.

3. Mattresses providing adequate flexibility and including energy dissipation characteristics. Edges shall be embedded to maintain edge stability under design inundation flows.

4. Concrete facing provided in accordance with protective slab on grade countermeasures and containing adequate anchorage to the reinforced earth system under design inundation flows.

5. Stone armoring and riprap provided to withstand tsunami shall be designed as follows: Stone diameter shall not be less than the size determined according to design criteria based on tsunami 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 shall be specifically considered, using methods in the recognized literature.

Subject to independent review, it shall be permitted to base designs on physical or numerical modeling.

3 TSUNAMI HAZARD

3.1 Design Basis Tsunami

Unless otherwise required, buildings shall be designed for an MCT event, that is based on a deterministic envelope of potential mega-thrust seismogenic scenarios of MSZ, as given by Smith et al. (2013). The scenarios are given in Table 3.1.

3.2 Tsunami Design Zones

The inundation limit zone for the cities of Karachi, Gwadar, Jiwani, Pasni and Ormara have been produced in accordance with this section (Figures 3.1 to 3.5). These zones are based on potential horizontal inundation limit for a MCT level event. The tsunami zoning maps for above mentioned cities are available in the Tsunami Load Calculation Program which is downloadable from Department of Earthquake Engineering at NED University website (*eqd.neduet.edu.pk/tsunami*).

Table 3.1 Potential scenarios for deterministic envelope (Smith et al. 2013).

Scenario	**Length** [km (mi)]	**Width** [km (mi)]		**Rupture Area** [km^2 (mi^2)]		**Seismic Moment** [N.m (lb.ft)]		**M$_W$**	
		Min	Max	Min	Max	Min	Max	Min	Max
Scenario 1: *Full length of MSZ*	800 (497)	210 (130)	355 (221)	1.68E+11 (6.5E+10)	2.84E+11 (1.1E+11)	5.04E+22 (3.72E+22)	8.52E+22 (6.28E+22)	9.07	9.22
Scenario 2: *Eastern Half of MSZ, east of Sistan Suture Zone*	400 (248.5)	210 (130)	355 (221)	8.40E+10 (3.24E+10)	1.42E+11 (5.48E+10)	2.52E+22 (1.86E+22)	4.26E+22 (3.14E+22)	8.87	9.02
Scenario 3: *Sistan Suture Zone to Little Murray Ridge*	220 (136.7)	210 (130)	355 (221)	4.62E+10 (1.78E+10)	7.81E+10 (3.02E+10)	1.39E+22 (1.03E+22)	2.34E+22 (1.73E+22)	8.69	8.85

Note: Estimated potential magnitudes generated by different rupture scenarios (does not account for partial / heterogeneous rupture). 10 m (32.8 ft) of coseismic slip is used. Maximum potential rupture width is taken from the deformation front to the 350ºC contour. Minimum width is taken from the limit of significant offshore seismicity (~60 km (37.28 mi) landward of the deformation front) to the forearc Moho (30 km (18.64 mi)) - subducting plate intersection.

The requirements of the tsunami zoning map shall be superseded if a site-specific hazard analysis is carried out for a building or structure, as outlined in section 3.5.

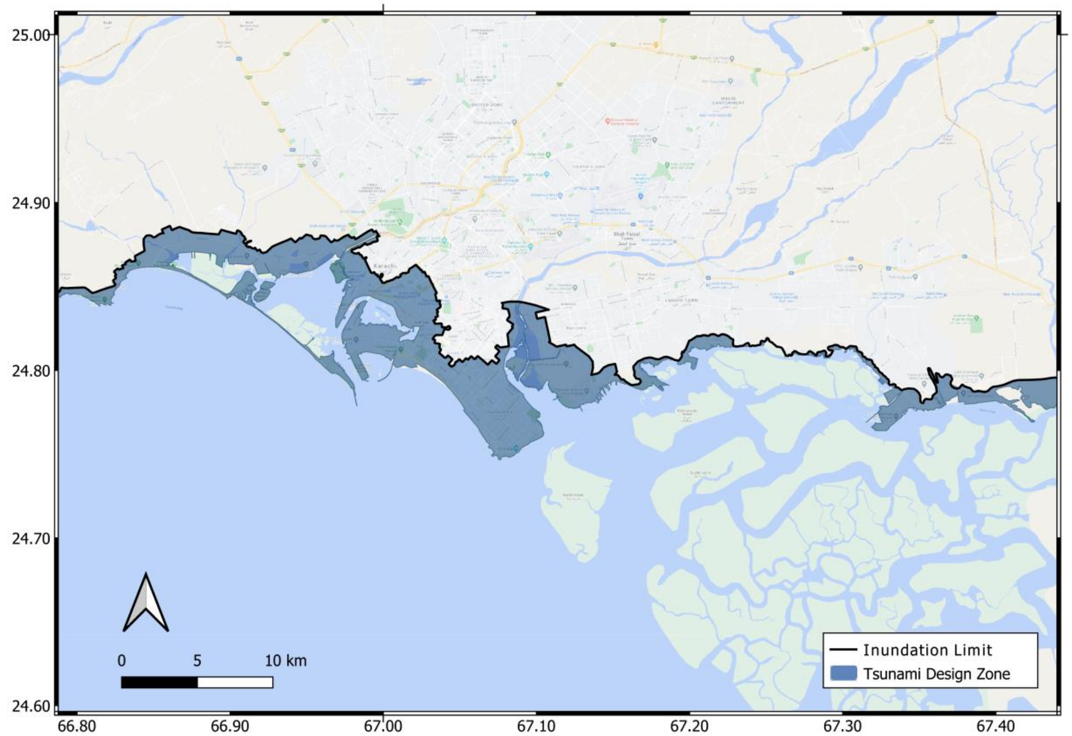

Figure 3.1 -Inundation limit zone for Karachi

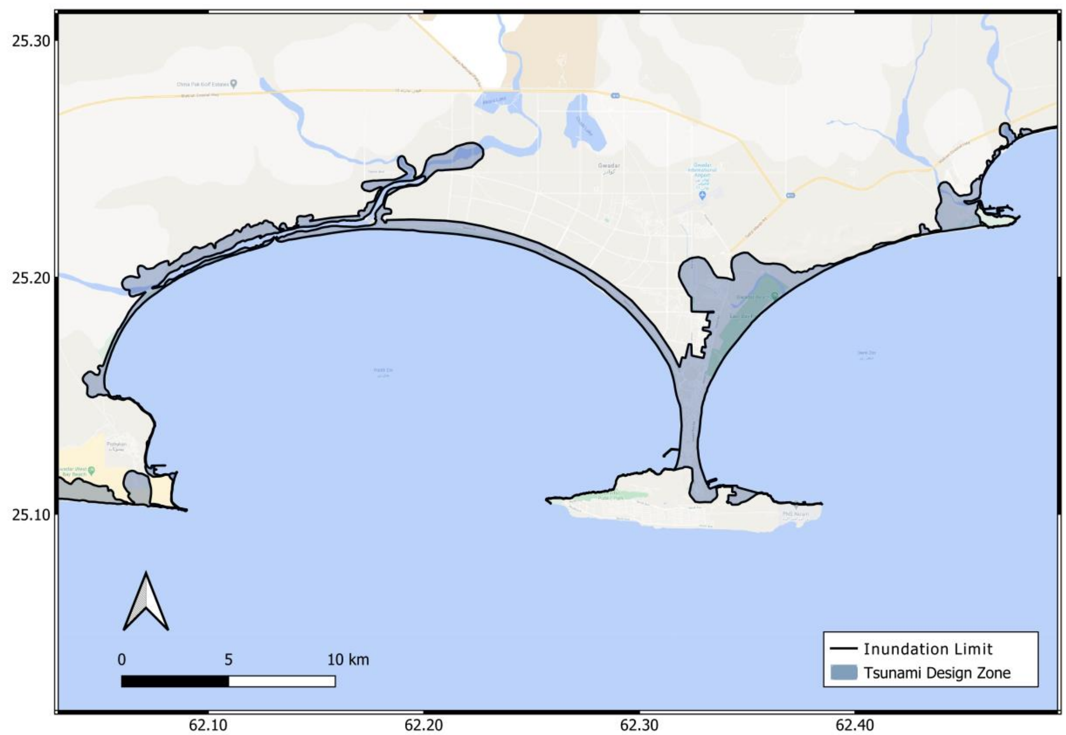

Figure 3.2 - Inundation limit zone for Gwadar

Figure 3.3 - Inundation limit zone for Jiwani

Figure 3.4 - Inundation limit zone for Pasni

Figure 3.5 - Inundation limit zone for Ormara

3.3 Hazard Analysis Criteria

3.3.1 Tsunami Risk Category II and III Structures

MCT inundation depth and tsunami flow velocity characteristics at a Tsunami Risk Category II or III building or other structure shall be determined by using the Energy Grade Line Analysis (EGLA), using the geographic topography, inundation limit and runup provided in the Tsunami Hazard Tool. The site-specific Deterministic Tsunami Hazard Analysis (DTHA) in section 3.5 shall be permitted as an alternate to the Energy Grade Line Analysis.

Exception*: For coastal regions subject to tsunami inundation and not covered by Figures 3.1 to 3.5, inundation limits, and runup elevations shall be determined using the site-specific procedures of section 3.5.*

3.3.2 Tsunami Risk Category IV Structures

EGLA of section 3.4.2 shall be performed for Tsunami Risk Category IV buildings and other structures, and the site-specific DTHA of section 3.5 shall also be performed. Site-specific velocities determined by site-specific DTHA determined to be less than the EGLA shall be subjected to limitations in section3.5. Site-specific velocities determined to be greater than the EGLA shall be used.

Exception: *For structures other than Tsunami Vertical Evacuation Refuge Structures, a site-specific DTHA shall not be performed where the inundation depth resulting from the EGLA is determined to be less than 3.66 m (12 ft).*

3.3.3 Sea Level Change

The direct physical effects of potential relative sea level change shall be considered in determining the maximum inundation depth during the project lifecycle. A project lifecycle of not less than 50 years shall be used. The minimum rate of potential relative sea level change shall be the historically recorded sea level change rate for the site. The potential increase in relative sea level during the project lifecycle of the structure shall be added to the Reference Sea Level and to the tsunami runup elevation.

3.4 Hazard Analysis

3.4.1 Maximum Inundation Depth and Flow Velocities

The maximum inundation depths and flow velocities associated with the stages of tsunami flooding shall be determined in accordance with section 3.4.2. Calculated flow velocity shall not be taken as less than 3.0 m/s (10 ft/s) and as greater than the lesser of $1.5(gh_{max})^{1/2}$ and 15.2 m/s (50 ft/s).

Where the maximum topographic elevation along the topographic transect between the shoreline and the inundation limit is greater than the runup elevation, one of the following methods shall be used:

1. The site-specific procedure of section 3.5 shall be used to determine inundation depth and flow velocities at the site, subject to the above range of calculated velocities.

2. For determination of the inundation depth and flow velocity at the site, EGLA shall be used assuming a runup elevation and horizontal inundation limit that has at least 100% of the maximum topographic transect.

3.4.2 Energy Grade Line Analysis

The maximum velocity and maximum inundation depth along the ground elevation profile up to the inundation limit shall be determined using EGLA. The orientations of the topographic transect profiles used shall be determined considering the requirements of section 5.1.9. The ground elevation along the transect (z_i) shall be represented as a series of linear sloped segments each with a Manning's coefficient consistent with the equivalent terrain macro-roughness friction of that terrain segment. EGLA shall be performed incrementally using Eq. (3.4.-1) across the topographic transect in a stepwise procedure. Eq. (3.4.-1) shall be applied across the topographic transect from the runup where the hydraulic head at the inundation limit (x_R) is zero, and the water elevation is equal to the runup (R) by calculating the change in hydraulic head at each increment of terrain segment toward the shoreline until the site of interest is reached, as shown in Figure 3.6.

$$E_{g,i} = E_{g,i-1} + (\varphi_i + s_i)\Delta x_i \tag{3.4-1}$$

where,

$E_{g,i}$ = Hydraulic head at point $i = h_i + u_i^2/2g = h_i(1 + 0.5F_{ri}^2)$;

h_i = Inundation depth at point i;

u_i = Maximum flow velocity at point i;

F_{ri} = Froude number = $u/(gh)^{1/2}$ at point i;

Δx_i = $x_{i-1} - x_i$, the increment of horizontal distance, which shallot be coarser than 30.5 m (100 ft) spacing;

x_i = Horizontal distance inland from vertical datum shoreline at point i;

φ_i = Average ground slope between points i and $i - 1$;

s_i = Friction slope of the energy grade line between points i and $i - 1$, is calculated per Eq. (3.4-2).

$$\begin{aligned} s_i &= u_i^2/((1.00/n)^2\, h_i^{4/3}) = gF_{ri}^2/((1.00/n)^2\, h_i^{1/3}) \text{ [SI]} \\ s_i &= u_i^2/((1.49/n)^2\, h_i^{4/3}) = gF_{ri}^2/((1.49/n)^2\, h_i^{1/3}) \text{ [FPS]} \end{aligned} \tag{3.4-2}$$

where n is the manning's coefficient of the terrain segment in consideration (Table 3.2).

Velocity shall be determined as a function of inundation depth, in accordance with the prescribed value of the Froude number calculated according to Eq. (3.4-3).

$$F_r = \alpha \left(1 - \frac{x}{x_R}\right)^{0.5} \tag{3.4-3}$$

where the value for Froude number coefficient (α) 1.0 shall be used. Where tsunami bores are required to be as considered per section 3.4.5, the tsunami bore conditions specified in sections 4.2.2 and 4.2.3 shall be applied using the values of h_e and $(h_e u^2)_{bore}$ evaluated with α= 1.3.

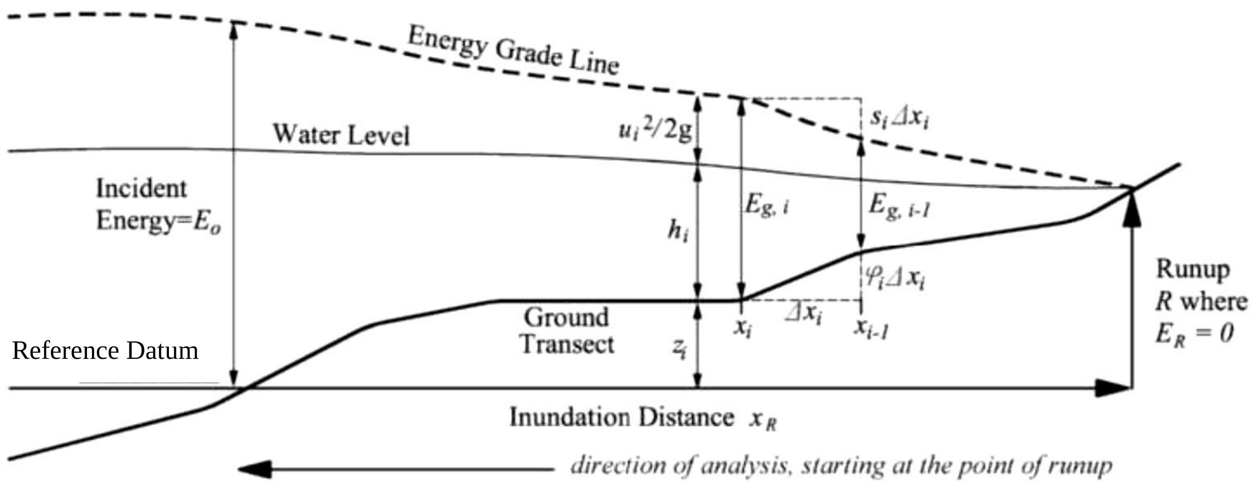

Figure 3.6 - Energy Grade Line Method for Tsunami Inundation Depth and Velocity.

Table 3.2 - Manning's roughness coefficient, *n*

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 atleast urban density	0.04

3.4.3 Tsunami Hazard Tool

For the purpose of tsunami design and assessment of buildings, the design inundation and runup for the coastal cities, as shown in Figure 3.1 to 3.5, shall be used from the geocoded Tsunami Hazard Tool software tool, developed in accordance with this document. The tool and supporting user manual is downloadable from the website of the Department of Earthquake Engineering at NED University, Pakistan (*eqd.neduet.edu.pk/tsunami*).

3.4.4 Terrain Roughness

It shall be permitted to perform inundation analysis assuming bare-earth conditions with equivalent macro-roughness. Bed roughness shall be prescribed using the Manning’s coefficient

n. It shall be permitted to use the values listed in Table 3.2 or other values based on terrain analysis in the recognized literature or as specifically validated for the inundation model used.

3.4.5 Tsunami Bores

Tsunami bores shall be considered where any of the following conditions exist:

1. The prevailing nearshore bathymetric slope is 1/100 or milder,

2. Shallow fringing reefs or other similar step discontinuities in nearshore bathymetric slope occur,

3. Where historically documented,

4. As described in the recognized literature, or

5. As determined by a site-specific inundation analysis.

Where tsunami bores are deemed to occur, the tsunami bore conditions specified in sections 4.2.2 and 4.2.3 shall be applied.

3.4.6 Amplified Flow Velocities

Flow velocities determined in this section shall be adjusted for flow amplification in accordance with section 5.1.8 as applicable. The adjusted value need not exceed the maximum limit specified in section 3.4.1.

3.5 Site Specific Hazard Analysis

When required by section 3.3, inundation depth and flow velocities shall be determined by site-specific inundation studies complying with the requirements of this section. Site-specific analysis shall use the deterministic approach while considering an envelope of the worst-case scenarios mentioned in Table 3.1 for tsunami inundation depths (h), and flow velocities (u).

3.5.1 Tsunamigenic Sources

Tsunami sources shall consider the following to the extent that deterministic hazards are documented in the recognized literature:

1. Local subduction zone sources: It shall be permitted to use a system of delineated and discretized subduction zones in the Indian Ocean basin comprised of systems of rectangular subfaults and their corresponding tectonic parameters.

2. Principal seismic sources shall include MSZ where the maximum moment magnitude considered shall include the values given in Table 3.1.

3. Local, non-subduction zone seismic fault sources capable of moment magnitude of 7 or greater, including offshore and/or submarine fault sources that are tsunamigenic.

4. Local coastal and submarine landslide sources documented in the recognized literature as being tsunamigenic of similar runup, as determined by historical evidence.

3.5.2 Treatment of Modeling and Natural Uncertainties

Epistemic uncertainties for consideration of model parameters and aleatory uncertainties for natural variability shall be considered while computing inland tsunami inundation. These considerations are provided as follows:

1. *Epistemic uncertainties*: Two approaches are allowed to account for uncertainties in the model parameters provided in Table 3.1.
    a. Develop envelope of results from assessment of worst case tsunami inundation scenarios, identified based on parameters in Table 3.1 and sources identified in section 3.5.1.
    b. Use a statistically weighted logic tree approach, by assigning weights against scenarios based on parameters in Table 3.1 and sources identified in section 3.5.1.

2. *Aleatory uncertainties*: Uncertainties based on natural variability in source processes, modeling, and tidal variation shall be included as they relate to nearshore processes and runup. When accounting for long wave durations with multiple maxima in the tsunami time series, it shall be permitted to consider tidal variability by selecting a rational tidal elevation independently for worst case tide stage for each wave maximum.

3.5.3 Earthquake Rupture Source Parameters for Inland Tsunami Inundation

The tsunami modeling algorithm shall be based on earthquake rupture slips for worst case tsunami events, which shall be permitted for direct computation of inundation runup, depth, and velocities from the sources and uncertainties consistent with section 3.5.1 & 3.5.2, and computing conditions set out in section 3.5.4.

3.5.4 Computation of Tsunami Inundation and Runup

3.5.4.1 Selection of worst case tsunami inundation scenarios and parameters

Worst case tsunami event scenarios shall be developed based on parameters in Table 3.1 and considerations provided in section 3.4. Rupture locations for worst case scenarios shall be taken as directly under the specific site, and to the immediate east and west of the site. Each tsunami event shall be analyzed to determine representative design parameters consisting of maxima of runup, inundation depth, flow velocity, and momentum flux.

3.5.4.2 Seismic subsidence

Modeling code capable of using co-seismic deformation of the sea-floor shall be used for simulation of tsunami events from the near source Makran Trench.

3.5.4.3 Model macro-roughness

It shall be permitted to perform the inundation mapping under bare-earth conditions with macro-roughness. Bed-roughness shall be permitted to be described using Manning's coefficient with a default value of 0.025 for ocean bottom and on land. Use of other values based on terrain analysis shall be justified from recognized literature or shall be specifically validated for the inundation model.

3.5.4.4 Nonlinear modeling of inundation

Nonlinear shallow water wave equations or equivalent modeling techniques shall be used to compute inundation and runup. The following effects shall be included as applicable to the bathymetry and topography:

- Shoaling, refraction, and diffraction to determine nearshore tsunami amplitude;
- Dispersion effects in the case of short-wavelength sources, such as landslides and volcanic sources;
- Reflected waves;
- Channeling in bays;
- Edge waves, and shelf and bay resonances;
- Bore formation and propagation; and
- Harbor and port breakwaters and levees.

3.5.4.5 Model spatial resolution

A Digital Elevation Model (DEM) for the bathymetry shall have a resolution not coarser than 30.0 m (99 ft), and for topography not coarser than 10.0 m (33ft). If the model uses a nested grid or adaptive mesh refinement, the reduction in grid spacing between consecutive grids should not be by more than a factor of 5. Use of the best available integrated DEM data shall be permitted when approved by the Authority having Jurisdiction

3.5.4.6 Built environment

If buildings and other structures are included for the purposes of more detailed flow analysis, the DEM shall have a minimum resolution of 10 ft (3.0 m) and to capture flow deceleration and acceleration in the built environment.

3.5.4.7 Site-specific inundation flow parameters

Inundation parameters for the scenarios from each disaggregated source region shall be determined. Each tsunami event shall be analyzed to determine representative parameters such as maximum runup, inundation depth, flow velocity, and/or specific momentum flux by either of techniques provided in section 3.5.2.

In urban environments, the resulting flow velocities at a given structure location shall not be reduced from 90% of those determined in accordance with section 3.4 before any velocity adjustments caused by flow amplification. For other terrain roughness conditions, the resulting flow velocities at a given structure location shall not be taken as less than 75% of those determined in accordance with section 3.4 before any velocity adjustments caused by flow amplification.

3.5.4.8 Tsunami design parameters for inland flow

The flow parameters of inundation depth, flow velocity, and/or specific momentum flux at the site of interest shall be captured from a time history inundation analysis. Tsunami inundation depth and velocity shall be evaluated for the site at the stages of inundation defined by the Load Cases in section 5.1.3. 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 shall be considered in addition to the Load Cases.

3.6 Impact Hazard at Ports

3.6.1 Impact Zones from Shipping Containers, Ships and Barges

Shipping containers and ships or barges disbursed from container yards, ports, and harbors shall be evaluated as potential debris impact objects. In such cases, a probable dispersion region shall be identified for each source to determine if the structure 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, shall be evaluated per sections 4.3.5 and 4.3.6.

The expected total plan area of the debris objects at the source shall be determined as follows:

1. For containers, this is the average number of on-site containers multiplied by their plan area.

2. For barges, the area of a nominal AASHTO (2009) design barge [59.5 × 10.67 m, or 635 m^2 (195 × 35 ft, or 6,825 ft^2)] shall be multiplied by the average number of barges at the source.

3. For ships, the average vessel deck plan area at the site shall be used.

The geographic center of the source shall be identified, together with the primary flow direction, as defined in section 5.1.9. Lines $\pm$22.5 degrees from this centerline shall be projected in the direction of tsunami inflow, as shown in Figure 3.5. If topography (such as hills) will bound the water from this 45 degrees sector, the direction of the sector shall be rotated to accommodate hill lines or the wedge shall be narrowed where it is constrained on two or more sides.

First, an arc of the debris impact hazard region for inflow shall be drawn as follows: one arc and the two radial boundary lines of the 45 degrees 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 shall be permitted to be curtailed in accordance with any of the following boundaries:

a. The extent of the sector shall be permitted to be curtailed where the maximum inundation depth is less than 0.914 m (3 ft), or in the case of ships where the inundation depth is less than the ballasted draft plus 0.61 m (2 ft).

b. Structural steel and/or concrete structures shall 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 0.61 m (2 ft), or (2) for ships, the inundation depth minus the sum of the ballasted draft and 0.61 m (2 ft).

Second, the debris impact hazard region for inflow and outflow shall be determined by rotating the circular segment by 180 degrees and placing the center at the intersection of the centerline 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 shall be designed for strikes by a container and/or other vessel carried with the inflow. Buildings and other structures contained only in the second sector shall be designed for strikes by a container and/or other vessel carried in the outflow. Buildings and other structures contained in both sectors shall be designed for strikes by a container and/or other vessel moving in either direction.

3.6.2 Shipping Container Impact Zones for Ports in Pakistan

Karachi and Gwadar are two key port cities in Pakistan. Procedures outlined in section 3.6.1 were used to develop shipping container impact zones in Karachi. There are two key port regions in Karachi, Karachi Port (Location A) and Port Qasim (Location B). Shipping container debris impact hazard region indicating container sites with inflow and outflow impact zones (wedges) are provided in Figures 3.6 and 3.7 for these ports.

Buildings and other structures contained within the debris impact hazard region shall be designed for strikes by a container and/or other vessel moving in the relevant direction. The impact load by shipping containers and/or ships and barges, as appropriate, shall be evaluated per sections 4.3.5 and 4.3.6.

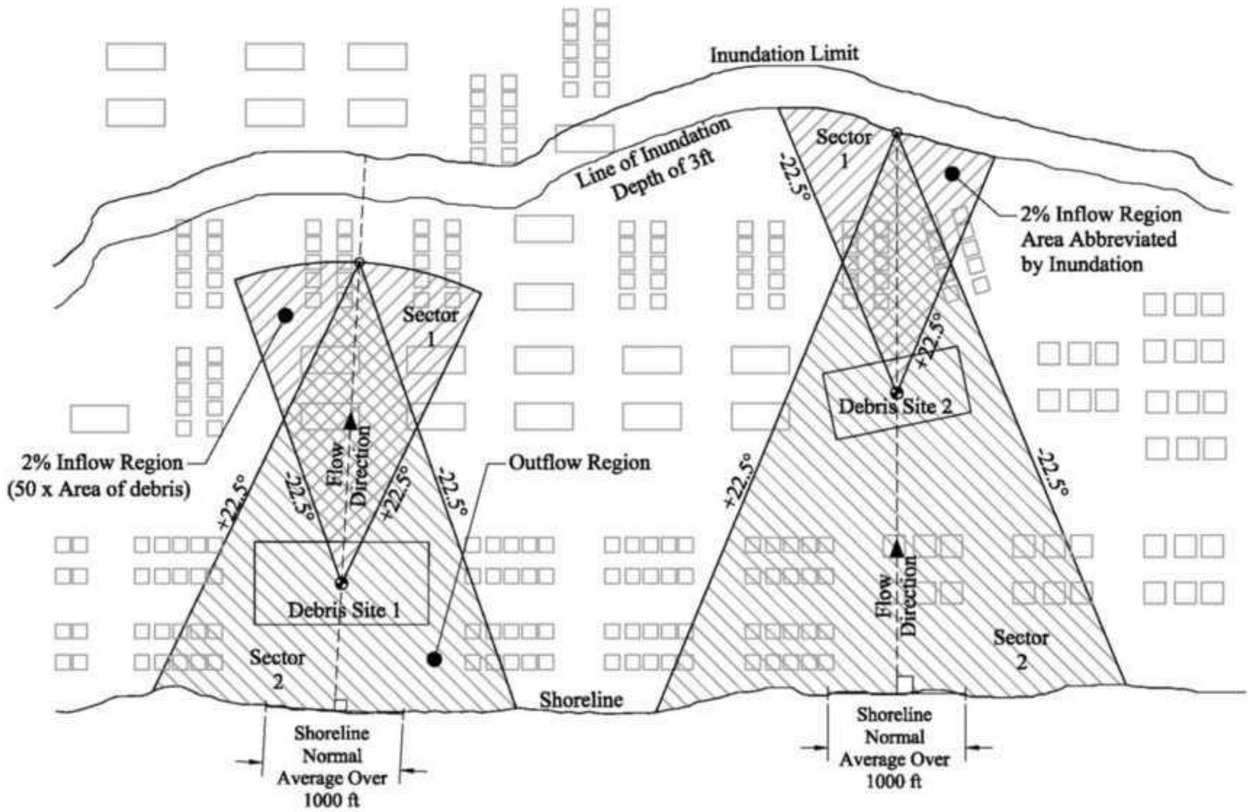

Figure 3.5 - Illustration for determining floating debris impact region.

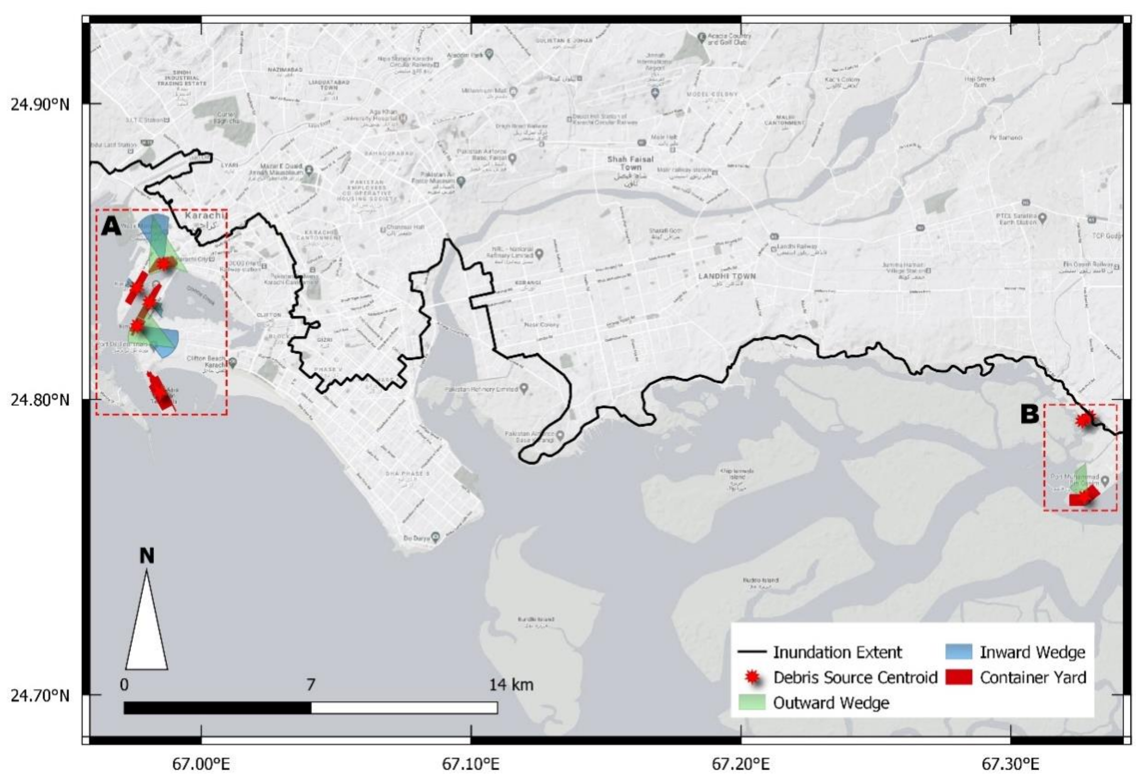

Figure 3.6 - Port areas in Karachi

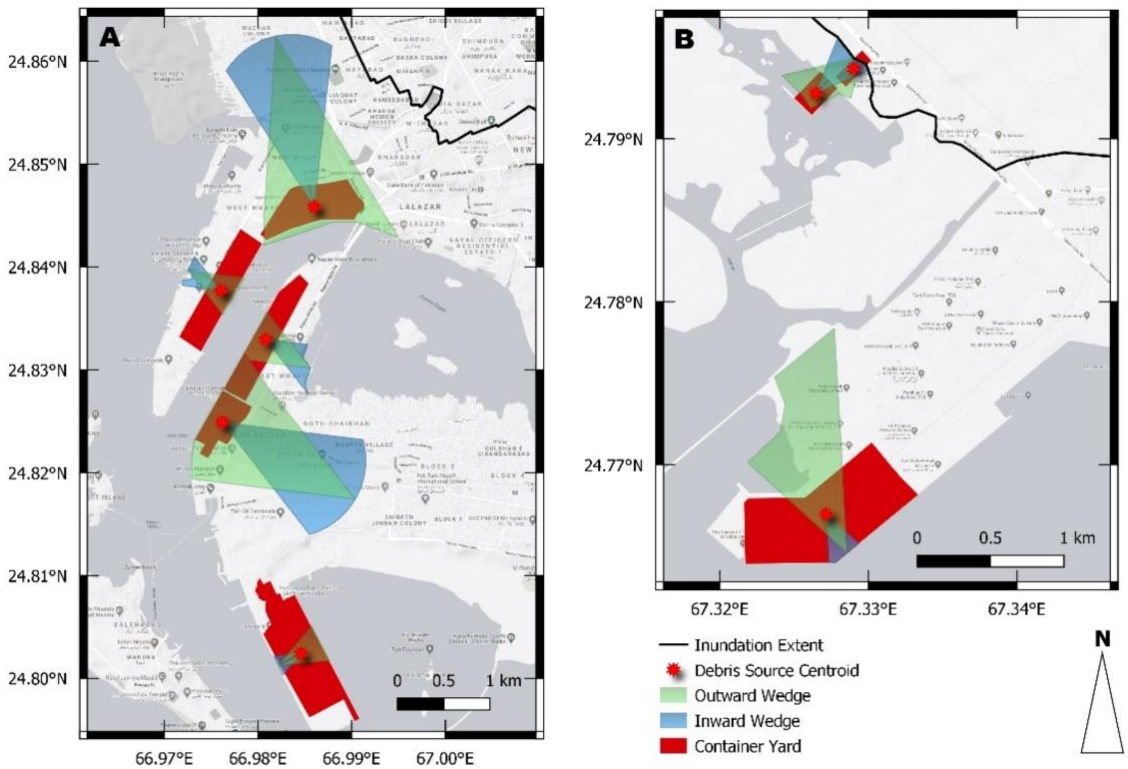

Figure 3.7 - Shipping container debris impact zone for port areas in Karachi

4 TSUNAMI LOADS

4.1 Hydrostatic Loads

4.1.1 Buoyancy

Reduced net weight caused by buoyancy shall be evaluated for all inundated structural and designated nonstructural elements of the building in accordance with Eq. (4.1-1). Uplift caused by buoyancy shall include enclosed spaces without tsunami breakaway walls that have opening area less than 25% of the inundated exterior wall area.

Buoyancy shall 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 missile wind-borne debris impact or blast loading, shall 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, shall be included in this calculation of uplift.

$$F_v = \gamma_S V_w \tag{4.1-1}$$

4.1.2 Unbalanced Lateral Hydrostatic Force

Inundated structural walls with openings less than 10% of the wall area and either longer than 9.14 m (30 ft) without adjacent tsunami breakaway walls or having a two- or three-sided perimeter structural wall configuration shall be designed to resist an unbalanced hydrostatic lateral force given by Eq. (4.1-2), occurring during the Load Case 1 and the Load Case 2 inflow cases defined in section 5.1.3. In conditions where the flow overtops the wall, h_{max} in Eq. (4.1-2) is replaced with the height of the wall.

$$F_h = \frac{1}{2}\gamma_S\, b\, h_{max}^2 \tag{4.1-2}$$

4.1.3 Residual Water Surcharge Load on Floors and Walls

All horizontal floors below the maximum inundation depth shall be designed for dead load plus a residual water surcharge pressure given by Eq. (4.1-3). Structural walls that have the potential to retain water during drawdown shall also be designed for lateral residual water hydrostatic pressure. Where h_r shall not exceed the height of the perimeter structural element at the floor.

$$p_r = \gamma_S\, h_r \quad where,\ \ h_r = h_{max} - \ h_s \tag{4.1-3}$$

4.1.4 Hydrostatic Surcharge Pressure on Foundation

Hydrostatic surcharge pressure caused by tsunami inundation is given by Eq. (4.1-4).

$$p_s = \gamma_S \, h_{max} \tag{4.1-4}$$

4.2 Hydrodynamic Loads

Hydrodynamic loads shall be determined in accordance with this section. The structural lateral load-resisting system and all structural components below the inundation elevation at the site shall be designed for the hydrodynamic loads given in either section 4.2.1 or 4.2.2. All wall and slab components shall also be designed for all applicable loads given in section 4.2.3.

4.2.1 Simplified Equivalent Uniform Lateral Static Pressure

It shall be permitted to account for the combination of any unbalanced lateral hydrostatic and hydrodynamic loads by applying an equivalent maximum uniform pressure determined in accordance with Eq. (4.2-1), applied over 1.3 times the calculated maximum inundation depth (h_{max}) at the site, in each direction of flow.

$$p_{uw} = 1.25 \, I_{tsu} \, \gamma_S \, h_{max} \tag{4.2-1}$$

4.2.2 Detailed Lateral Forces

4.2.2.1 Drag force on structure

The building lateral-force-resisting system shall be designed to resist overall drag forces at each level caused either by incoming or outgoing flow at Load Case 2 given by Eqs. (4.2-2) and (4.2-3).

$$F_{dx} = \frac{1}{2}\rho_S I_{tsu} C_d \, C_{cx} B \, (hu^2) \tag{4.2-2}$$

where C_d is given in Table 4.1, and C_{cx} is determined as:

$$C_{cx} = \frac{\sum(A_{col} + \ A_{wall}) + \ 1.5A_{beam}}{B \ h_{sx}} \tag{4.2-3}$$

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 story heights below the inundation height for each load case defined in section 5.1.3.Any wall that is not a tsunami breakaway wall shall be included in the A_{wall}. C_{cx} shall not be taken as less than the closure ratio value given in section 5.1.10 but shall not be taken as greater than 1.0.

Table 4.1 - Drag Coefficients for Rectilinear Structures

Width to Inundation Depth[1] 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
$\geq$ 120	2.0

[1]Inundation depth for each of the three Load Cases of inundation specified in section 5.1.3. Interpolation shall be used for intermediate values of width to inundation depth ratio B/h_{sx}.

4.2.2.2 Drag force on structural components

The lateral hydrodynamic load given by Eq. (4.2-4) shall be applied as a pressure resultant on the projected inundated height (h_e) of all structural components and exterior wall assemblies below the inundation depth.

For interior components the values of C_d given in Table 4.2 shall be used. For exterior components, a C_d value of 2.0 shall be used, and width dimension (b) shall be taken as the tributary width multiplied by the closure ratio value given in section 5.1.10.

The drag force on component elements shall not be additive to the overall drag force computed in section 4.2.2.1.

$$F_d = \frac{1}{2}\rho_S I_{tsu} C_d b\ (h_e u^2) \qquad (4.2\text{-}4)$$

Table 4.2 - Drag Coefficients for Structural Components

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

4.2.2.3 Loads on vertical structural components

The force on vertical structural components shall be determined as the hydrodynamic drag force in accordance with Eq. (4.2-5a).

$$F_w = \frac{1}{2}\rho_S I_{tsu} C_d b\ (h_e u^2) \tag{4.2-5a}$$

Where flow of a tsunami bore occurs with a Froude number greater than 1.0 and where individual wall, wall pier, or column components have a width to inundation depth ratio of 3 or more, force shall be determined by Eq. (4.2-5b).

$$F_w = \frac{3}{4}\rho_S I_{tsu} C_d b\ (h_e u^2)_{bore} \tag{4.2-5b}$$

Force F_w is applied to all vertical structural components that are wider than 3 times the inundation depth corresponding to Load Case 2 during inflow as defined in section 5.1.3.

4.2.2.4 Load on perforated walls

For walls with openings that allow flow to pass between wall piers, the force on the elements of the perforated wall shall be permitted to be determined using Eq. (4.2-6), but shall not be less than F_d per Eq. (4.2-4):

$$F_{pw} = (0.4C_{cx} + 0.6)F_w \tag{4.2-6}$$

4.2.2.5 Load on angled walls

For walls oriented at an angle less than 90 degrees to the flow directions considered in section 5.1, the transient lateral load per unit width shall be determined in accordance with Eq. (4.2-7).

$$F_{w\theta} = F_w \sin^2\theta \tag{4.2-7}$$

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

4.2.3 Pressure on Slabs

4.2.3.1 Flow stagnation pressure

Walls and slabs in buildings spaces that are subject to flow stagnation pressure shall be designed to resist pressure determined in accordance with Eq. (4.2-8).

$$P_p = \frac{1}{2}\rho_S I_{tsu} u^2 \tag{4.2-8}$$

4.2.3.2 Surge uplift

Slabs and other horizontal components shall be designed to resist the applicable uplift pressures given in this section.

1. *Submerged slabs during Tsunami Inflow:* Horizontal slabs that become submerged during tsunami inundation inflow shall be designed for a minimum hydrodynamic uplift pressure of 0.958 kPa (20 psf) applied to the soffit of the slab. This uplift is an additional Load Case to any hydrostatic buoyancy effects required by section 4.1.1.

2. *Slabs over Sloping Grade:* Horizontal slabs located over grade slope, φ, greater than 10 degrees shall be designed for a redirected uplift pressure applied to the soffit of the slab, given by Eq. (4.2-9), but not less than 0.958 kPa (20 psf).

$$P_u = 1.5\rho_s I_{tsu} u_v^2 \tag{4.2-9}$$

where u_v is u tan φ, 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 is the average slope of grade plane beneath the slab.

4.2.3.3 Bore flow entrapped in wall-slab recesses

Hydrodynamic loads for bore flows entrapped in structural wall-slab recesses shall be determined in accordance with this section. The reductions of load given for cases below (2-5) may be combined multiplicatively, but the net load reduction shall not exceed the maximum individual reduction given by any one of these sections.

1. *Pressure load in structural wall-slab recesses:* Where flow of a tsunami bore beneath an elevated slab is prevented by a structural wall connected to the slab, following shall be considered;
   - The slab located within h_s distance from the wall, and the wall be designed for the outward pressure (P_u) of 16.76 kPa (350 psf).
   - Slab located beyond h_s but within a distance of h_s + l_w from the wall, shall be designed for an upward pressure of half of P_u [i.e., 8.38 kPa (175 psf)].
   - The slab beyond a distance of h_s + l_w from the wall shall be designed for an upward pressure of 1.436 kPa (30 psf).

2. *Reduction of load with inundation depth:* Where the inundation depth is less than two-thirds of the clear story height, the uplift pressures specified in case 1 shall be permitted to be reduced in accordance with Eq. (4.2-10) but shall not be taken as less than 1.436 kPa (30 psf).

$$P_u = I_{tsu}\left(28.25 - 7.66\frac{h_s}{h}\right)\ [kPa] \tag{4.2-10}$$

$$P_u = I_{tsu}\left(590 - 160\frac{h_s}{h}\right)\ [psf]$$

3. *Reduction of Load for Wall Openings:* Where the wall blocking the bore below the slab has openings through which the flow can pass, the reduced pressure on the wall and slab shall be determined in accordance with Eq. (4.2-11).

$$P_{ur} = C_{cx}P_u \tag{4.2-11}$$

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 w_g, adjacent to the wall, then the uplift pressure on the remaining slab shall be determined in accordance with Eq. (4.2-12).

$$P_{ur} = C_{hx}P_u \tag{4.2-12}$$

for $w_g < 0.5h_s$ $$C_{hx} = 1 - \frac{w_g}{h_s} \geq 0 \tag{4.2-13}$$

for $w_g \geq 0.5h_s$ $$C_{hx} = 0.56 - 0.12\frac{w_g}{h_s} \geq 0 \tag{4.2-14}$$

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 shall be permitted to be determined in accordance with section 4.2.3.1, but it need not exceed the pressure equivalent to the breakaway wall capacity.

4.3 Debris Impact Loads

Debris impact loads shall be determined in accordance with this section. These loads shall not be combined with other tsunami related loads as determined in other sections of this chapter.

Where the minimum inundation depth is 0.914 m (3 ft) or greater, design shall include the effects of debris impact forces. The most severe effect of impact loads within the inundation depth shall be applied to the perimeter gravity-load-carrying structural components. Except as specified below, loads shall be applied at points critical for flexure and shear on all such members in the inundation depth being evaluated. Inundation depths and velocities corresponding to Load Cases defined in section 5.1.3 shall be used. Impact loads shall not be applied simultaneously to all affected structural components.

Tsunami Risk Category III Critical Facilities and Tsunami Risk Category IV buildings and structures determined to be in the hazard zone for strikes by ships and barges in excess of 39,916 kg (88,000 lb) Deadweight Tonnage (DWT), as determined by the procedure of section 4.3.5, shall be designed for impact by these vessels in accordance with section 4.3.7.

4.3.1 Alternative Debris Impact Static Load

It shall be permitted to alternatively account for debris impact by poles, logs, vehicles, tumbling boulders, concrete debris, and shipping containers applying the force given by Eq. (4.3-1) as a maximum static load, in lieu of the loads defined in sections 4.3.2 to 4.3.6. This force shall be applied on all critical structural members in the inundation depth corresponding to Load Case 3 defined in section5.1.3.

$$F_i = 1470\ C_o I_{tsu}\ \ [kN]$$
$$F_i = 330\ C_o I_{tsu}\ \ [kips] \tag{4.3-1}$$

where C_o is the orientation coefficient, equal to 0.65.

Where it is determined that the site is not in an impact zone for shipping containers, ships, and barges, then it shall be permitted to reduce the simplified debris impact force to 50% of the value given by Eq. (4.3-1).

4.3.2 Load from Wood, Logs and Poles Impact

The nominal maximum instantaneous debris impact force shall be determined in accordance with Eq. (4.3-2).

$$F_{ni} = u_{max}\sqrt{km_d} \tag{4.3-2}$$

The design instantaneous debris impact force shall be determined in accordance with Eq. (4.3-3).

$$F_i = C_o I_{tsu} F_{ni} \tag{4.3-3}$$

where C_o is the orientation coefficient, equal to 0.65 for logs and poles; k is the effective stiffness is the impacting debris or the lateral stiffness of the impacted structural element(s) deformed by the impact, whichever is less; and, m_d is the mass W_d/g of the debris.

Logs and poles are assumed to strike longitudinally for calculation of debris stiffness in Eq. (4.3-2). The stiffness of the log or pole shall 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 454 kg (1,000 lb) and minimum log stiffness of 61,300 kN/m (350 kip/in.) shall be assumed.

The impulse duration for elastic impact shall be calculated from Eq. (4.3-4):

$$t_d = \frac{2m_d u_{max}}{F_{ni}} \tag{4.3-4}$$

For an equivalent elastic static analysis, the impact force shall be multiplied by the dynamic response factor (R_{max}) specified in Table 4.3. For a wall, the impact shall be assumed to act along the horizontal center of the wall, and the natural period shall 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 shall be allowed to use an alternative method of analysis per section 4.3.7.

Table 4.3 - Dynamic Response Ratio for Impulsive Loads (R_{max})

Ratio of Impact Duration to Natural Period of Structural Element	R_{max}
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
$\geq$ 1.4	1.5

4.3.3 Load from Vehicle Impact

An impact of floating vehicles shall be applied to vertical structural element(s) at any point greater than 0.914 m (3 ft) above grade up to the maximum depth. The impact force shall be taken as 30 kip (130 kN) multiplied by I_{tsu}.

4.3.4 Load from Submerged Boulder and Concrete Debris Impact

Where the maximum inundation depth exceeds 1.83 m (6 ft), an impact force of 36 kN (8,000 lb) multiplied by I_{tsu} shall be applied to vertical structural element(s) at 0.61 m (2 ft) above grade.

4.3.5 Load from Shipping Containers Impact

The impact force from shipping containers shall be calculated from Eqs. (4.3-2) and (4.3-4). The mass m_d is the mass of the empty shipping container. It shall be assumed that the strike contact is from one bottom corner of the front (or rear) of the container. The container stiffness is $k = EA/L$, 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 are provided in Table 4.4. C_o, the orientation factor, shall be taken as equal to 0.65 for shipping containers.

Table 4.4 - Weights and Stiffness of Waterborne Floating Debris

Type of Debris	**Weight**	**Stiffness (*k*)**
6.1 m (20 ft) standard shipping container oriented longitudinally	Empty: 2,270 kg (5,000 lb) Loaded: 13,150 kg (29,000 lb)	42,900 kN/m (245 kip/in.)
12.2 m (40 ft) standard shipping container oriented longitudinally	Empty: 3,810 kg (8,400 lb) Loaded: 17,240 kg (38,000 lb)	29,800 kN/m (170 kip/in.)

The nominal design impact force from Eq. (4.3-2) for shipping containers shall not be taken greater than 980 kN (220 kips).

For empty shipping containers, the impulse duration for elastic impact shall be calculated from Eq. (4.3-4). For loaded shipping containers the duration of the pulse is determined from Eq. (4.3-5):

$$t_d = \frac{(m_d + m_{contents})u_{max}}{F_{ni}} \tag{4.3-5}$$

where $m_{contents}$ shall 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 4.4 for loaded shipping containers. The design shall consider both empty and loaded shipping containers.

For an equivalent static analysis, the impact force shall be multiplied by the dynamic response factor (R_{max}) specified in Table 4.3. For a wall, the impact shall be assumed to act along the horizontal center of the wall, and the natural period shall 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 shall be permitted to use an alternative method of analysis per section 4.3.7.

4.3.6 Load from Extraordinary Debris Impact

Where the maximum inundation depth exceeds 3.66 m (12 ft), extraordinary debris impacts of 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 3.6 shall be assumed for impact. Impact shall be assumed at the perimeter of Tsunami Risk Category III Critical Facilities and Tsunami Risk Category IV buildings and 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 shall be calculated from Eq. (4.3-3), based on the stiffness of the impacted structural element and a weight equal to the Lightship Weight (LWT) plus 30% of DWT.

An alternative analysis of section 4.3.7 shall 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 5.1.6.3, 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.

4.3.7 Alternative Methods for Response Analysis

A dynamic analysis is permitted 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. (4.3-3). If the impact is large enough to cause inelastic behavior in the structure, it shall 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 shall 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 shall be u_{max} multiplied by the product of Importance Factor (I_{tsu}) and the orientation factor (C_o).

5 STRUCTURAL DESIGN PROCEDURES

5.1 General Design Procedures

5.1.1 Basis for Design

The design of structures, components, and foundations in consideration with tsunami risk categories, section 2.4, when subjected to hydrostatic, hydrodynamic and debris impact loads, sections 4.1, 4.2 and 4.3, and effects from the MCT hazard, section 3, shall conform to the requirements of this section. Structural strength and stability shall be evaluated to determine if the design of the structure is capable of resisting the tsunami at the load cases defined in section 5.1.3. The structural acceptance criteria for this evaluation shall be in accordance with either section 5.1.5 or section 5.1.6.

5.1.2 Performance Requirements for Structures

1. For tsunami risk category II and III, structural components, connections, and foundations of buildings and structures shall be designed to meet Collapse Prevention Structural Performance criteria or better.

2. Tsunami risk category III critical facilities and risk category IV buildings and structures located within the Tsunami Design Zone shall be designed to meet the following requirements:
   a. The operational nonstructural components and equipment of the building necessary for essential functions and the supporting horizontal structural member shall be above the inundation elevation of the MCT.
   b. Occupiable levels shall be permitted only above the MCT inundation elevation. Structural components and connections in occupiable levels and foundations shall be designed in accordance with Immediate Occupancy Structural Performance criteria.
   c. Tsunami Vertical Evacuation Refuge Structures shall also comply with section 5.3.

5.1.3 Load Cases and Combinations

The following three Inundation Load Cases shall be evaluated:

Load Case 1: Where the exterior inundation depth does not exceed the maximum inundation depth and the lesser of one story or the height of the top of the first- story windows, the condition of combined hydrodynamic force with buoyant force shall be evaluated with respect to the depth of water in the interior, to consider maximum buoyancy effects. The interior water depth shall be evaluated in accordance with section 4.1.1.

Exception: *Load Case 1 need not be applied to Open Structures and to structures where the soil properties or foundation and structural design prevent buoyancy.*

Load Case 2: Maximum hydrodynamic forces shall be considered at two-thirds of maximum inundation depth, when the maximum velocity and maximum specific momentum flux is assumed to occur in either incoming or receding directions.

Load Case 3: Hydrodynamic forces at maximum inundation depth shall be considered, when velocity is assumed at one-third of maximum in either incoming or receding directions.

The inundation depths and velocities defined for Load Cases 2 and 3 shall be determined by Figure 5.1, unless a site-specific tsunami analysis is performed in accordance with section 3.5.

Principal tsunami forces and effects shall be combined with other specified loads in accordance with the load combinations of Eq. (5.1-1):

$$0.9D + F_{TSU} + H_{TSU} \quad (5.1\text{-}1a)$$

$$1.2D + F_{TSU} + 0.5L + 0.2S + H_{TSU} \quad (5.1\text{-}1b)$$

where F_{TSU} is the tsunami load effect for incoming and receding directions of flow; and, H_{TSU} is the load caused by tsunami-induced lateral foundation pressures developed under submerged conditions. Where the net effect of H_{TSU} counteracts the principal load effect, the load factor for H_{TSU} shall be 0.9

5.1.4 Tsunami Importance Factors

The Tsunami Importance Factors (I_{tsu}) given in Table 5.1 shall be applied to the tsunami hydrodynamic and impact loads in sections 4.2 and 4.3, respectively.

Table 5.1 - Tsunami Importance Factors (I_{tsu})

Tsunami Risk Category	I_{tsu}
II	1.0
III	1.25
Tsunami Risk Category IV, Vertical Evacuation Refuges, and Tsunami Risk Category III Critical Facilities	1.25

5.1.5 Acceptance Criteria for Lateral Load Resisting System

To resist the lateral force effects of the design tsunami event, evaluation of structural system capacity at the Life Safety Structural Performance Level for Seismic Design Category D, E, or F (Table 5.2 & 5.3), it is permitted to use the value of 0.75 times the required Horizontal Seismic Load Effect (E_m) which includes the system's overstrength factor (Ω_0) as defined in seismic provisions. For Immediate Occupancy Structural Performance objectives, the lateral-force-resisting system shall be explicitly analyzed and evaluated.

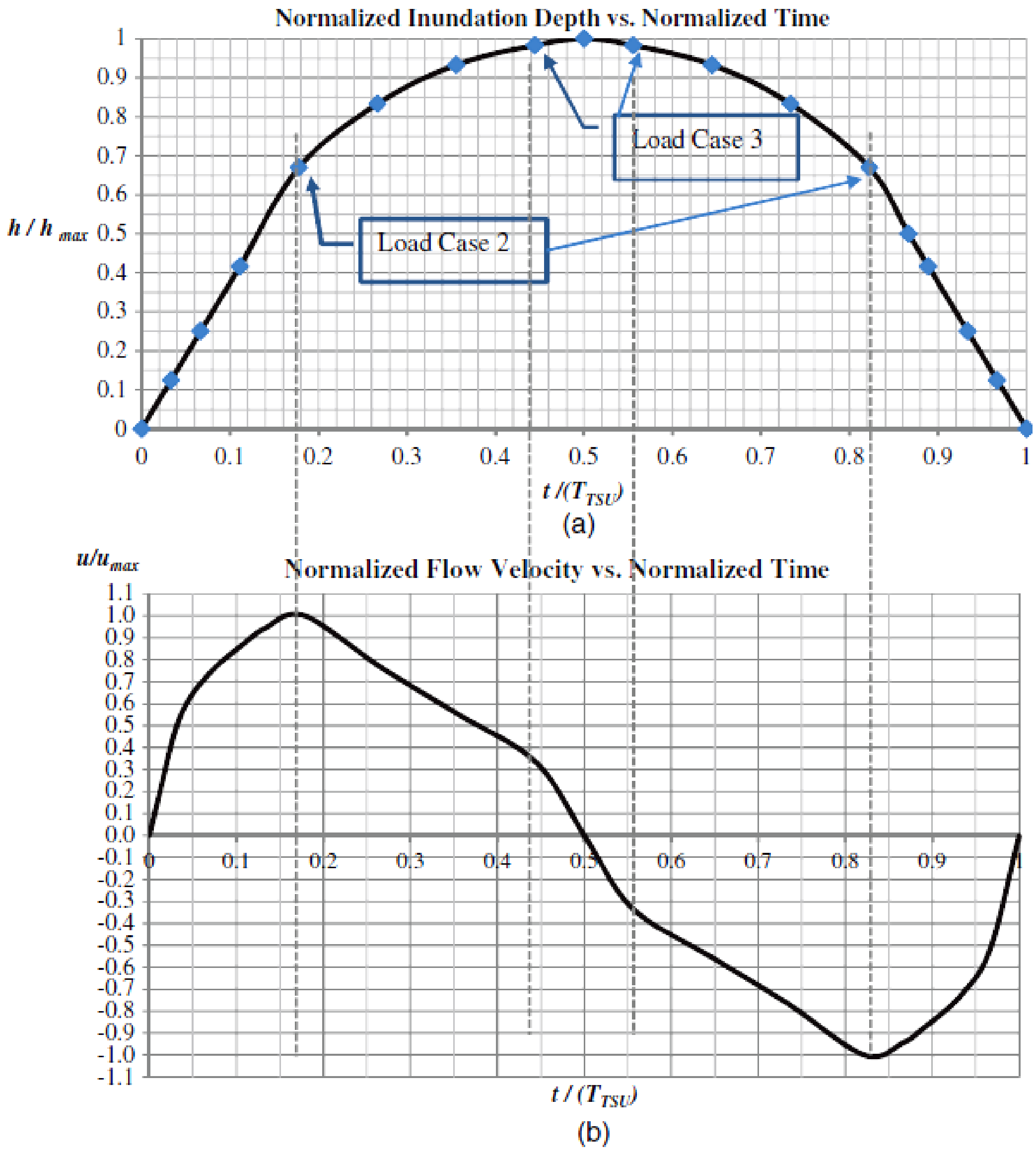

Figure 5.1 - Inundation load cases 2 and 3

Table 5.2 - Seismic Design Category based on short-period response acceleration (S_{DS})

Value of S_{DS}	***Risk Category***	
	I, II or III	**IV**
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

5.1.6 Design Criteria for Structural Components

Structural components shall be designed for the forces that result from the overall tsunami loads 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 shall be in accordance with section5.1.6.1, or in accordance with alternative procedures of 5.1.6.2 or 5.1.6.3, as applicable.

Table 5.3 - Seismic Design Category based on 1s-period response acceleration (S_{D1})

Value of S_{D1}	**Risk Category**	
	I, II or III	**IV**
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{DS} < 0.75$	D	D
$0.75 \leq S_{DS}$	E	F

5.1.6.1 Design by strength

Internal forces and system displacements shall be determined using a linearly elastic, static analysis. The structural performance criteria required in section 5.1.1 to 5.1.6, as applicable, shall be deemed to comply if the design strength of the structural components and connections are shown to be greater than the MCT loads and effects computed in accordance with the load combinations of section 5.1.3. Material resistance factors (ϕ) shall be used as prescribed in the material-specific standards for the component and behavior under consideration.

5.1.6.2 Design by performance

1. Alternative Analysis Procedures: It shall be permitted to use either a linear or nonlinear static analysis procedure. In a linear static analysis procedure, buildings and structures shall be modeled using an equivalent effective stiffness consistent with the secant value at or near the yield point. For a nonlinear static analysis procedure, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components of the structure shall be subjected to monotonically increasing loads until the required tsunami forces and applied actions are reached. For nonlinear static analysis procedures, expected deformation capacities shall be greater than or equal to the maximum deformation demands calculated at the required tsunami forces and applied actions. For debris impacts, it shall be permitted to use a nonlinear dynamic analysis procedure.

2. Alternative Structural Component Acceptability Criteria. All actions shall be classified as either ductility- governed actions or force-sustained actions based on component inelastic behavior and the duration of the load effect, as follows:
   a. Fluid forces in primary and secondary structural components detailed in accordance with the requirements of Seismic Design Category D, E, or F shall be evaluated as force-sustained actions.
   b. Debris impacts and foundation settlement effects on primary and secondary structural components shall be evaluated as ductility-governed actions.
   c. Debris impacts and foundation settlement effects on primary and secondary structural components not detailed in accordance with Seismic Design Category D, E, or F shall be evaluated as force-sustained actions.

For force-sustained actions, structural components shall have specified design strengths greater than or equal to the maximum design forces. Force-sustained actions shall be permitted to satisfy Eq. (5.1-2):

$$Q_{CS} \geq Q_{UF} \tag{5.1-2}$$

where Q_{CS} is the specified strength of the structural element, and Q_{UF} is the maximum force generated in the element because of gravity and tsunami loading.

Expected material properties as defined in national practicing codes shall be permitted to be used for ductility-governed actions. Results of a linear analysis procedure shall not exceed the component acceptance criteria for linear procedures of seismic provisions, for the applicable structural performance criteria required for the building or structure Tsunami Risk Category.

5.1.6.3 Design for progressive collapse avoidance

Where tsunami loads or effects exceed acceptability criteria for a structural element or where required to accommodate extraordinary impact loads, it shall be permitted to check the residual load-carrying capacity of the structure, assuming that the element has failed, in accordance with the residual load capacity combination (Eq. 5.1-3) and an alternate load path progressive collapse procedure in the recognized literature.

$$(0.9\ or\ 1.2)D + 0.5L + 0.2(L_r\ or\ S\ or\ R) \tag{5.1-3}$$

where D is the dead load; L is the live load; L_r is the roof live load; S is snow load; and R is the rain load.

5.1.7 Fluid Density

Seawater specific weight density γ_{sw} shall be taken as 10 kN/m^3 (64.0 lb/ft^3). Seawater mass density (ρ_{sw}) shall be taken as 1,025 kg/m^3 (2.0 slugs per cubic foot (sl/ft^3)). The minimum fluid specific weight density (γ_s) for determining tsunami hydrostatic loads accounting for suspended solids and debris flow-embedded smaller objects shall be

$$\gamma_s = k_s \gamma_{sw} \tag{5.1-4}$$

The minimum fluid mass density (ρ_s) for determining tsunami hydrodynamic loads accounting for suspended solids and debris flow-embedded smaller objects shall be

$$\rho_s = k_s \rho_{sw} \tag{5.1-5}$$

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

5.1.8 Flow Velocity Amplification

The effect of obstructing buildings and structures shall be permitted to be considered at a site that is exposed to the flow diffracting conditions given in section 5.1.8.1 by any of the following:

1. A site-specific inundation analysis that includes modeling of the built environment in accordance with section 3.5, or

2. The built environment is considered in the selection of Manning's roughness of Table 3.2 in accordance with the EGLA of section 3.4.2, or

3. Site-specific physical or numerical modeling in accordance with section 5.1.8.2 or section 5.1.12, as applicable.

5.1.8.1 Obstructing structures

The effect of obstructions on flow shall be considered where the obstructions are enclosed structures of concrete, masonry, or structural steel construction located within 152 m (500 ft) of the site, and both of the following apply:

1. Structures have plan width greater than 30.5 m (100 ft) 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 center third of the width of the downstream structure.

5.1.8.2 Physical / Numerical modeling

The effect of obstructing structures on the flow velocity at a site shall be permitted to be evaluated using site-specific numerical or physical modeling, as described in section 3. or 5.1.12. The velocity determined for a "bare-earth" inundation shall be amplified for the conditions of section 5.1.8.1. This analysis is not permitted to reduce the flow velocity except for structural countermeasures designed in accordance with section 2.5.

5.1.9 Flow Directionality

5.1.9.1 General direction

Design of structures for tsunami loads and effects shall consider both incoming and outgoing flow conditions. The principal inflow direction shall be assumed to vary by $\pm$22.5 degrees from the transect perpendicular to the orientation of the shoreline averaged over 152 m (500 ft) to either side of the site. The center of rotation of the variation of transects shall be located at the geometric center of the structure in plan at the grade plane.

5.1.9.2 Site specific direction

A site-specific inundation analysis performed in accordance with section 3.5 shall be permitted to be used to determine directionality of flow, provided that the directionalities so determined shall be assumed to vary by at least $\pm$10 degrees.

5.1.10 Closure Ratio

Loads on buildings shall 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 2.3. The load effect of debris accumulation against or within the Open Structure shall 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 5.1.3.

5.1.11 Tsunami Flow Cycles Consideration

Design shall consider a minimum of two tsunami inflow and outflow cycles, the first of which shall be based on an inundation depth at 80% of the MCT, and the second of which shall be assumed to occur with the Maximum Considered Tsunami inundation depth at the site. Local scour effects determined in accordance with section 5.2, caused by the first cycle, shall be assumed to occur at 80% of the MCT inundation depth at the site and shall be considered as an initial condition of the second cycle.

5.1.12 Physical Modeling of Tsunami Flow, Loads and Effects

Physical modeling of tsunami loads and effects shall be permitted as an alternative to the prescriptive procedures in sections 5.1.8 (flow velocity amplification), 4.2 (hydrodynamic loads), 4.3 (debris impact loads), and 5.2 (foundation design), provided that it meets all the following criteria:

1. The facility or facilities used for physical modeling shall be capable of generating appropriately scaled flows and inundation depths as specified for Load Cases in section 5.1.3.

2. The test facility shall be configured so that reflections and edge effects shall not significantly affect the test section during the duration of the experiments.

3. The scale factors used in the physical modeling shall not be less than those shown in Table 5.4. Scale model tests not directly addressed in the table shall include a justification of the model applicability and scaling procedures.

4. Debris impacts of full or partial components shall be tested at full scale unless accompanied by a justification of the appropriateness of scaled testing in terms of hydrodynamics and structural mechanics as well as material properties.

Table 5.4 - Minimum Scale Factors (SF) for Physical Modeling

Element	**Minimum SF**
Individual buildings	1:25
Flow modeling for groups of buildings	1:200
Structural components (e.g., walls, columns, piers)	1:10
Geotechnical investigations	1:5

5. The report of test results shall 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.

6. Test results shall be adjusted to account for effective density, as calculated in section 5.1.7.

7. Test results shall be adjusted by the Importance Factor from section 5.1.4.

8. Test results shall include the effects of flow directionality in accordance with section 5.1.9. This inclusion can be accomplished either by direct testing of flow at varying angles of incidence or by a combination of numerical and physical modeling that takes into account directionality of flow.

5.1.13 Non Building Structures

1. For tsunami Risk Category III non-building structures located in the Tsunami Design Zone shall be either protected from tsunami inundation effects or designed to withstand the effects of tsunami loads in accordance with section 5.1 of this chapter and in accordance with the specific performance requirements of section 5.1.2. Tsunami barriers used as inundation protection shall have a top of- wall elevation that is not less than 1.3 times the maximum inundation elevation at the barrier. The tsunami barrier shall also satisfy the requirements of section 2.5.

2. For tsunami Risk Category IV designated nonstructural systems in non-building structures located in the Tsunami Design Zone shall be (1) protected from tsunami inundation effects, (2) positioned above 1.3 times the inundation elevation of the MCT in such a manner that the Tsunami Risk Category IV non-building structure will be capable of performing its critical function during and after the MCT, or (3) designed to withstand the effects of tsunami loads in accordance with section 5.1 of this chapter and the specific performance requirements of section 5.1.2. Tsunami barriers used as inundation protection shall have a top-of wall elevation that is not less than 1.3 times the maximum inundation elevation at the barrier. The tsunami barrier shall also satisfy the requirements of section 2.5.

5.1.14 Non-Structural Components

Designated nonstructural components and systems in structures located in the Tsunami Design Zone shall be either protected from tsunami inundation effects or positioned in the structure above the inundation elevation of the MCT, such that the designated nonstructural components and systems will be capable of performing their critical function during and after the MCT. Tsunami barriers used as inundation protection shall have a top-of-wall elevation that is not less than 1.3 times the maximum inundation elevation at the barrier. The tsunami barrier shall also satisfy the requirements of section 2.5. Alternatively, it shall be permitted to design the designated nonstructural components and systems directly for tsunami effects, provided that inundation would not inhibit them from performing their critical function during and after the MCT.

5.2 Foundation Design Considerations

Design of structure foundations and tsunami barriers shall provide resistance to the loads and effects of section 5.2.2, shall provide capacity to support the structural load combinations defined in section 5.1.3, and shall accommodate the displacements determined in accordance with section 5.2.6. Foundation embedment depth and the capacity of the exposed piles to resist structural loads, including grade beam loads, shall both be determined taking into account the cumulative effects of general erosion and local scour. Alternatively, it shall be permitted to use the performance-based criteria of section 5.2.3. Site characterization shall include relevant information specified in seismic provisions, Geotechnical Investigation Report Requirements for Subsurface Soil Conditions.

5.2.1 Resistance Factors

Resistance factor of ϕ = 0.67 shall be applied to the foundation capacities for stability analyses, uplift design, 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 ϕ = 0.67 shall also be assigned for the resisting capacities of uplift resisting anchorage elements.

5.2.2 Design Criteria

Foundations and tsunami barriers shall be designed to accommodate:

- The effects of lateral earth pressure,
- Hydrostatic forces computed in accordance with section 4.1,
- Hydrodynamic loads computed in accordance with section 4.2, and
- Uplift and underseepage forces computed in accordance with section 5.2.3.

Foundations shall 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 shall be considered in accordance with the requirements of this section. A minimum of two wave cycles shall be considered for such effects.

5.2.3 *Uplift and Seepage Force*

Tsunami uplift and underseepage forces shall be evaluated as described in this section.

1. Uplift and underseepage forces shall include the three inundation Load Cases defined in section 5.1.3.

2. Strength loss caused by scour, liquefaction and pore pressure softening, and other soil effects shall be considered. Additionally, uplift and underseepage forces on the foundation shall be determined for cases where
   a. Soil is expected to be saturated before the tsunami, or
   b. Soil saturation is anticipated to occur over the course of the incoming series of tsunami waves, or
   c. The area of concern is expected to remain inundated after the tsunami.

3. The effect of live load and snow load shall not be used for uplift resistance.

5.2.4 *Strength Loss*

Loss of ground shear strength because of tsunami-induced pore pressure softening shall be accounted up to a depth of 1.2 times the maximum inundation depth, when analyzing under horizontal soil loads per section 5.2.7. Tsunami-induced pore pressure softening shall not be considered at locations where the maximum Froude number is less than 0.5.

5.2.5 *Erosion*

General erosion during tsunami inundation runup and drawdown conditions shall be considered. Analysis of general erosion shall account for flow velocity amplification as described in section 5.1.8; it shall also account for enhancement caused by tsunami-induced pore pressure ground softening.

Exception: *Analysis of general erosion is not required for rock or other non-erodible strata that are capable of preventing scour from tsunami flow of 9.14 m/s (30 ft/s).*

General erosion during drawdown conditions shall consider flow concentration in channels, including newly formed channels during tsunami inundation and drawdown (channelized scour). Analysis of channelized scour shall not include enhancement caused by pore pressure ground softening.

5.2.6 *Scour*

The depth and extent of scour shall be evaluated using the methods of this section stated below.

Exception: *Scour evaluation is not required for rock or other non-erodible strata that prevent scour from tsunami flow of 9.14 m/s (30 ft/s) nor for Open Structures.*

1. *Sustained Flow Scour*: scour, including the effects of sustained flow around structures and including building corner piles, shall be considered. Sustained flow scour design depth and area extent shall be determined by dynamic numerical or physical modeling or empirical methods in the recognized literature. It shall be permitted to determine sustained flow scour and associated pore pressure softening in accordance with Table 5.5 and Figure 5.2. Local scour depth determined shall 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 shall 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 scour area limits shall be considered to encompass the exposed building perimeter and to extend either side of the foundation perimeter 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 noncohesive soils.

Table 5.5 - Design scour depth caused by sustained flow and pore pressure softening

Inundation Depth *h*	**Scour Depth *D*[1]**
<3.05 m(10 ft)	1.2 *h*
≥3.05 m(10 ft)	3.66 m(12 ft)

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

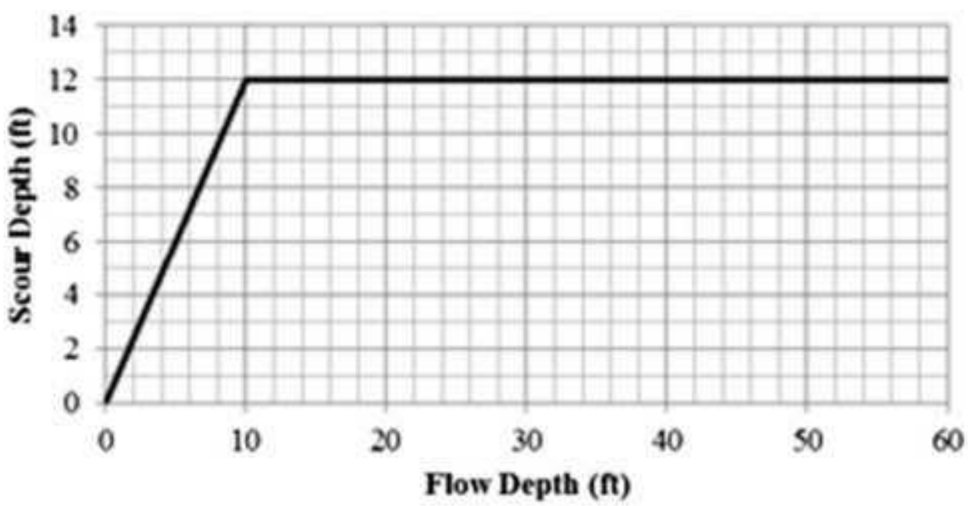

Figure 5.2 - Scour depth caused by sustained flow and pore pressure softening

2. *Plunging Scour*. Plunging scour horizontal extent and depth shall be determined by dynamic numerical or physical modeling or by empirical methods. In the absence of site-specific dynamic modeling and analysis, the plunging scour depth (D_s) shall be determined by Eq. (5.2-1).

$$D_s = c_{2V}\sqrt{\frac{qU\sin\psi}{g}} \tag{5.2-1}$$

where c_{2V} is the dimensionless scour coefficient, permitted to be taken as equal to 2.8; ψ is the angle between the jet at the scour hole and the horizontal, taken as the lesser value of 75 degrees and the side slope of the overtopped structure on the scoured side, in the absence of other information; q is the discharge per unit width over the overtopped structure, as illustrated in Figure 5.3 and calculated in accordance with Eq. (5.2-2)

$$q = C_{dis} \frac{2}{3} \sqrt{2gH_O^{3/2}} \tag{5.2-2}$$

where C_{dis} is a dimensionless discharge coefficient obtained in accordance with Eq. (5.2-3) and, 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 as illustrated in Figure 5.3, in accordance with Eq. (5.2-4):

$$C_{dis} = 0.611 + 0.08 \frac{H_O}{H_B} \tag{5.2-3}$$

$$U = \sqrt{2g(h + d_d)} \tag{5.2-4}$$

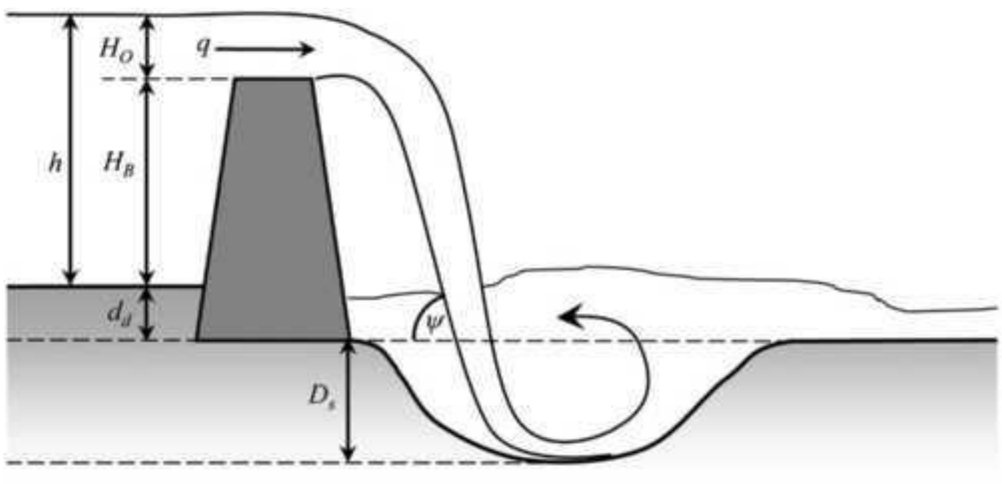

Figure 5.3 - Plunging Scour Parameters and Illustration

5.2.7 *Horizontal Soil Loads*

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

5.2.8 *Displacements*

Vertical and horizontal displacements of foundation elements and slope displacements shall be determined using empirical or elastoplastic analytical or numerical methods from recognized literature by applying tsunami loads determined in section 5.2.2 together with other applicable geotechnical and foundation loads required by this standard.

5.2.9 *Alternative Performance Criteria*

In situ soil stresses from tsunami loads and effects shall be included in the calculation of foundation pressures. For coseismic 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 shall be those existing at the end of seismic shaking, including liquefaction, lateral spread, and fault rupture effects.

Building foundations shall provide sufficient capacity and stability to resist structural loads and the effects of general erosion and scour in accordance from recognized literature. For Tsunami Risk Category IV buildings and structures, it shall be permitted to evaluate the overall performance of the foundation system for potential pore pressure ground softening by performing a two- or three-dimensional tsunami–soil–structure interaction numerical modeling analysis. The results shall be evaluated to demonstrate consistency with the structural performance acceptance criteria in section 5.1. An independent peer review shall also be conducted as part of a review of the performance-based design by the Authority having Jurisdiction.

5.3 Vertical Evacuation Refuge Structures

Tsunami Vertical Evacuation Refuge Structures designated as a means of alternative evacuation by the Authority Having Jurisdiction shall be designed in accordance with the additional requirements of this section.

5.3.1 *Inundation Elevation and Depth*

Tsunami refuge floors shall be located at a height greater of 3.05 m (10 ft) or one- story height above 1.3 times the MCT inundation elevation as determined by a site-specific inundation analysis, as indicated in Figure 5.4. The same site-specific inundation elevation, factored by 1.3, shall also be used for the design of Tsunami Vertical Evacuation Refuge Structure in accordance with sections 5.1 to 5.2.

5.3.2 *Refuge Live Load*

An assembly live load (L_{refuge}) of 4.8 kPa (100 psf) shall be used in any designated evacuation floor area within a tsunami refuge floor level.

5.3.3 *Impact loads*

Where the maximum inundation depth exceeds 1.83 m (6 ft), the laydown impact of adjacent pole structures collapsing onto occupied portions of the building shall be considered.

5.3.4 *Construction Reports*

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

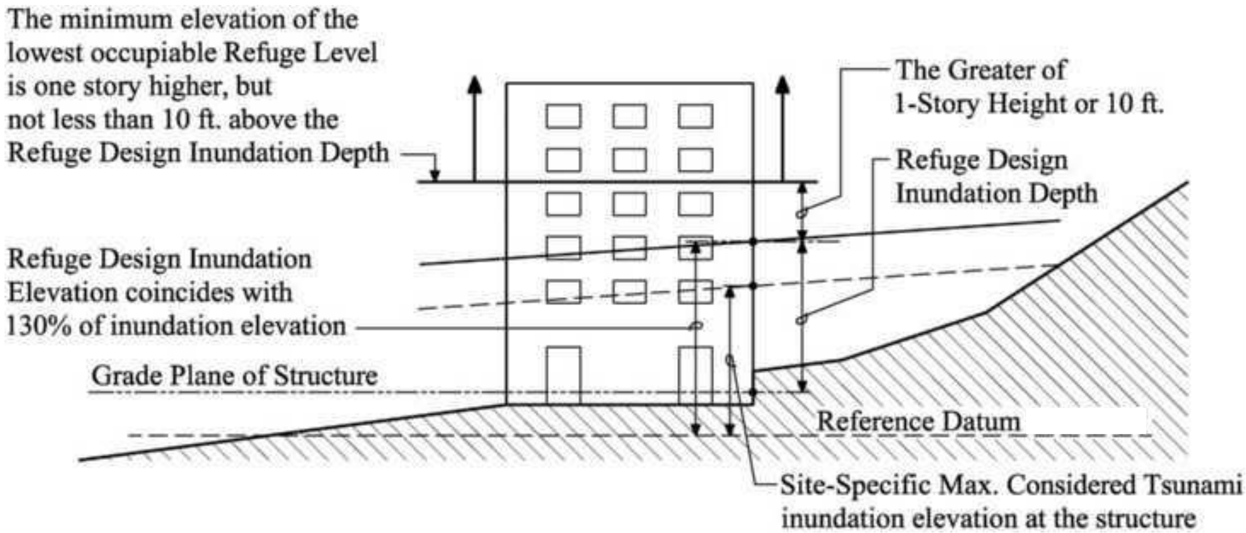

Figure 5.4 - Minimum Refuge Level Elevation Illustration.

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APPENDIX A - SHIPPING CONTAINER HAZARD FOR PORTS IN KARACHI

During tsunami flooding, moveable items present near shore behave as debris which are transported inland by the tsunami waves. The debris consists of everything from small items like bins to large ships and shipping containers. The inland movement of the debris is significant to evaluate the impact loads during structural design and hazard to already built structures. This section addresses the impact hazard assessment for one of the debris item i.e. shipping containers, considering the flow direction and flow depth.

Procedures provided in section 3.6.1 were used to identify the methodology for developing debris impact hazard region for ports in Karachi. The methodology identified is as follows:

1. Demarcate the plan area of all container yard located near the shore line.
2. Determine the debris source geometric centroids of all yards.
3. Calculate the radius of arc segment creating wedge, taking 50 times the area of debris at the source.
4. Considering the flow direction as perpendicular to the shoreline at all debris source centroids.
5. Extend lines in clockwise and counterclockwise direction from the debris source centroid at ±22.5 degrees to create a wedge buffer.
6. If the circular arc segment extends beyond inundation limit, then the circular segment radius will be curtailed to the limit.
7. For outward flow the circular arc segment will be developed from the intersection of the inward flow direction and the circular arc segment, or curtailed debris flow extent and extending back to the shoreline.
8. Again outward flow segment will be developed at ±22.5 degrees and exactly opposite in direction to the primary flow.

Karachi's coastline hazard assessment for debris impact from shipping containers was carried out in similar manner. In the first phase container yards near Karachi Port and Port Qasim were identified as shown in Figure A.1 considering the tsunami inundation limit. Identifying geometric centroids of all marked container yards as debris source center.

The identified container yards can be further categorized on the basis of container yard capacity, and volume of container as shown in Table A.1. For assessment it is considered container yards accommodate 6 m (20 ft) and 12 m (40 ft) length containers simultaneously. For each container yard total sum of containers, volumetric distribution is identified by counting maximum number of containers placed in a yard at a given time using aerial imagery as shown in Figure A.2 and listed in Table A.1 for all container yards.

In the second phase the circular arc segment radius is calculated using the plan area of all debris objects at the source based on the number of containers multiplied by their plan area. A standard shipping container is 2.43 m (8 ft) wide; therefore, the total plan area of the m (20 ft) and 12 m (40 ft) length containers calculated for container yard 1;

Table A.1 - Container yard capacity (no. of containers) and distribution.

Name	**Volumetric Distribution**		**Total Containers**
	6 m (20 ft)	**12 m (40 ft)**	
Debris Source Centroid 1	524	455	979
Debris Source Centroid 2	2058	5376	7434
Debris Source Centroid 3	1138	747	1885
Debris Source Centroid 4	3112	3084	6196
Debris Source Centroid 5	4410	7660	12070
Debris Source Centroid 6	475	739	1214
Debris Source Centroid 7	9522	11124	20646

Figure A.1 - Demarcation of container yards near Port areas

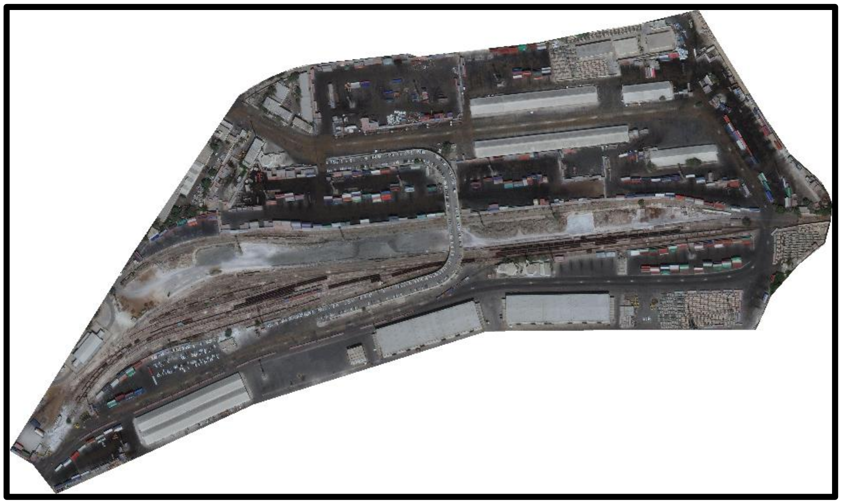

Figure A.2 - Aerial imagery for shipping container yard at Karachi port.

$$A = 524(6 \times 2.43) + 455\ (12 \times 2.43) = 20907.72\ m^2\ (229{,}440\ ft^2)$$

Projecting lines from the geometric center of the debris source by $\pm$22.5 degree angles, the resulting 45 degree wedge extends inland until the circular arc segment is 50 times the total area of the debris at the source. Therefore, the area of the arc segment must equal 50A. The area of a 45 degree arc segment is one-eighth of the full circle area; therefore, the radius of the arc segment is determined as follows:

$$A = \frac{\pi R^2}{8} = 50A = 1{,}045{,}386\ m^2\ (11{,}472{,}000\ ft^2)$$

Therefore R = 1647 m (5404.57 ft), for the arc shown in blue in Figure A.3. If the circular sector extends beyond the inundation limit, then the debris is assumed to run aground at an inundation depth of 0.91 m (3 ft) (Figure A.3). The estimated arc radius and actual arc radius for all container yard considering the run aground length for minimum 0.91 m (3 ft) draft and topographic limit listed in detail in Table A.2. The primary flow direction is considered to be perpendicular to the shoreline and passing through the debris source centroid throughout the coastline.

Table A.1 - List of identified debris source centroids

Name	**Longitude**	**Latitude**	**Estimated Inward Radius**	**Actual Inward Radius**	**Outward Radius**
Debris Source Centroid 1	66.98599	24.84587	502 m (1647 ft)	502 m (1647 ft)	675 m (2216 ft)
Debris Source Centroid 2	66.97601	24.83775	1500 m (4923 ft)	152 m (498 ft)	189 m (621 ft)
Debris Source Centroid 3	66.97619	24.8249	680 m (2232 ft)	224 m (735 ft)	217 m (711 ft)
Debris Source Centroid 4	66.98082	24.83297	1277 m (4190 ft)	69.2 m (227 ft)	134 m (438 ft)

Debris Source Centroid 5	66.98459	24.80241	1862 m (6110 ft)	91.7 m (301 ft)	161 m (529 ft)
Debris Source Centroid 6	67.32647	24.7928	586 m (1922 ft)	119 m (390 ft)	279 m (916 ft)
Debris Source Centroid 7	67.32715	24.76697	2363 m (7753 ft)	90.5 m (297 ft)	466 m (1530 ft)

Procedures provided in section 3.6.1 are adopted for the application of debris impact hazard assessment tool at Karachi's coastal belt which can be represented by Figure A.4 and Figure A.5 in detail. The respective geometric centroids according to their spatial signature in terms of latitude-longitude, and projected radii drawn perpendicular to the shoreline in both directions are listed in Table A.2.

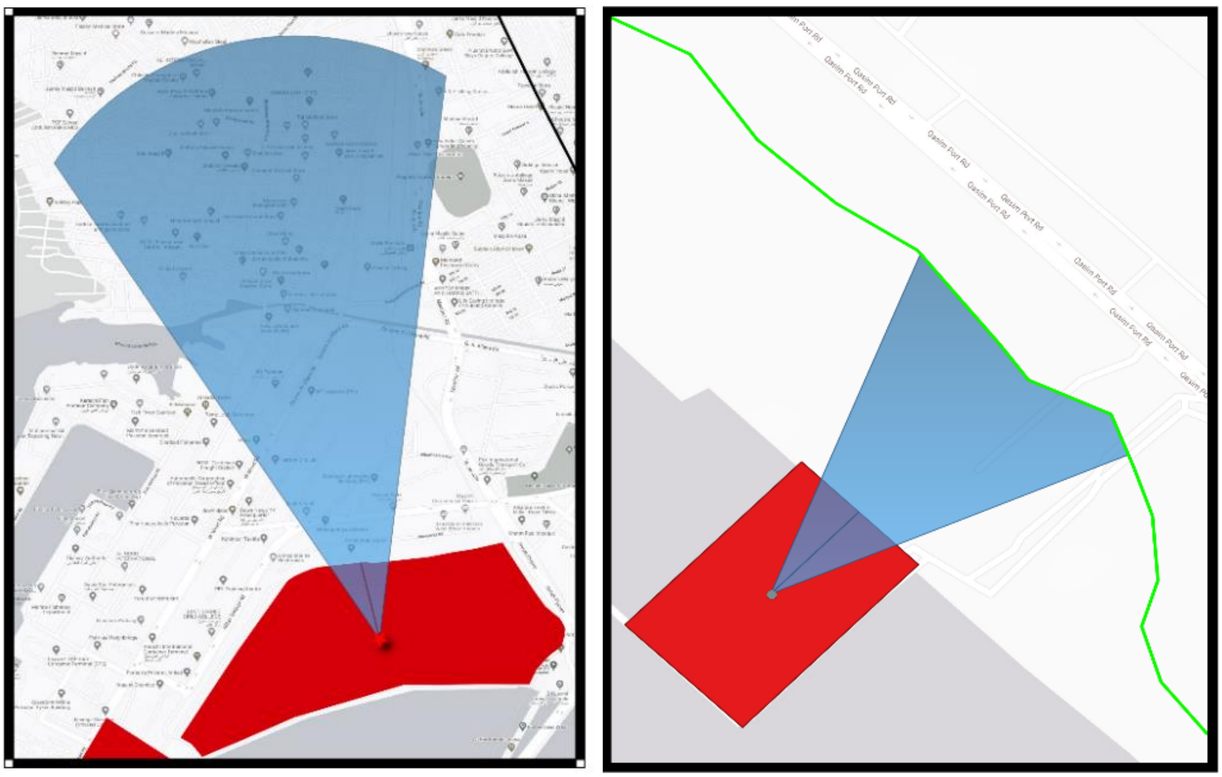

Figure A.3 - Inward wedge projection from debris source centroid till inundation

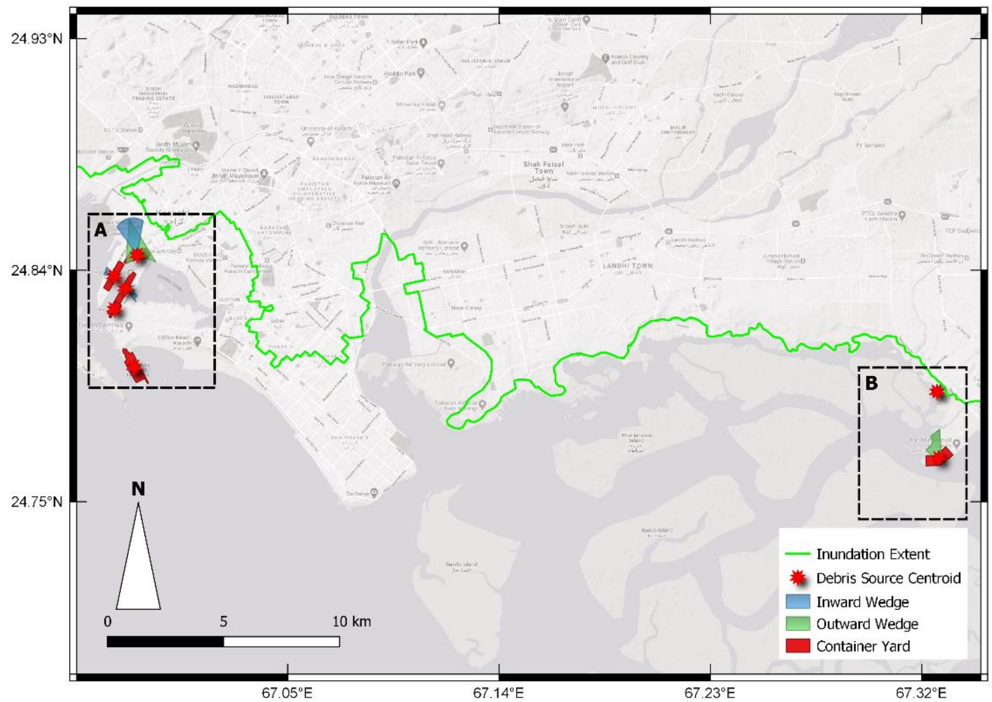

Figure A.4 - Application of debris impact hazard assessment tool at Karachi's coastal belt

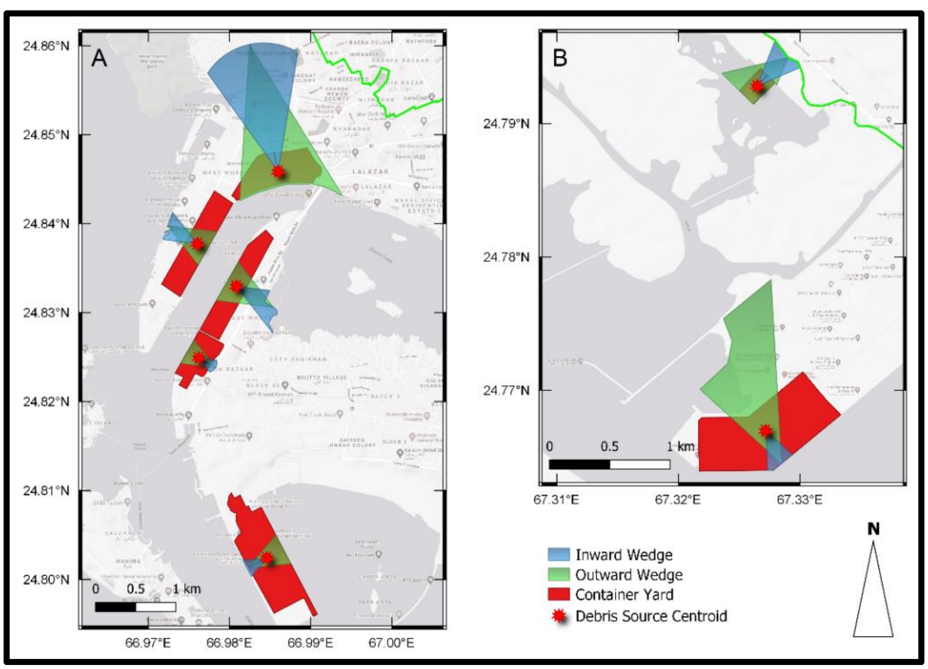

Figure A.5 - Application of debris impact hazard assessment tool at Karachi Port and Port Qasim

Table A.2 - List of identified debris source centroids

Name	Longitude	Latitude	Inward Radius (m)	Outward Radius (m)
Debris source centroid 1	66.98599	24.84587	1684	2536
Debris source centroid 2	66.97601	24.83775	257	606
Debris source centroid 4	66.97619	24.8249	1684	1977
Debris source centroid 3	66.98082	24.83297	744	764
Debris source centroid 5	66.98459	24.80241	296	556
Debris source centroid 6	67.32647	24.7928	410	500
Debris source centroid 7	67.32901	24.79431	85	540
Debris source centroid 8	67.32715	24.76697	297	1530

The impact force from shipping containers shall be calculated from Eq. (4.3-2) and design instantaneous debris impact force shall be calculated from Eq. (4.3-3), as given below:

$$F_{ni} = u_{max}\sqrt{km_d} \tag{4.3-2}$$

$$F_i = C_o I_{tsu} F_{ni} \tag{4.3-3}$$

where C_o is the orientation coefficient, equal to 0.65 for logs and poles; k is the effective stiffness is the impacting debris or the lateral stiffness of the impacted structural element(s) deformed by the impact, whichever is less; and, m_d is the mass W_d/g of the debris.

Minimum values for the weight and stiffness of the containers are given in Table 4.4 (section 4.3.5). The velocity of the shipping container is assumed to match the flow velocity corresponding to the flow depth being considered. Because the shipping container is assumed to float at the surface of the flow, this would imply that the impact force will vary as the flow depth increases. For a 12 m (40 ft) empty container placed at debris source site 1 with an average speed of 0.467 m/s (18.385 in/s), the nominal maximum instantaneous debris impact force (F_{ni}) from Eq. 4.3-2 is calculated to be 3090.4 kN (694.75 kips).