APPENDIX

B

Loads Due to Tsunami

B.1 Background

The current structural design codes provide very little guidance for loads specifically induced by tsunami effects on coastal structures; the established design codes focus mainly on loadings due to riverine floods and storm waves. There are significant differences in physical conditions between tsunami and other floods. For a typical tsunami, the water surface fluctuates near the shore with an amplitude of several metres, during a period of a few to tens of minutes. This timescale is intermediate between the typical of riverine floods stretching from hours to days, and the tens of seconds or less associated with cyclic loading of storm waves. This intermediate timescale makes the behaviour and characteristics of tsunami quite distinct from other coastal hazards, and the effects cannot be inferred from common knowledge or intuition. The timescale is long enough to allow tsunami to penetrate a great distance inland, while it is still short enough to make the tsunami flow highly transient (Yeh et al. 2005). Rapid rise and fall of tsunami inundation appears to enhance scour around a building. A relatively short lead-time for warning and evacuation likely leaves unsecured potentially hazardous objects behind, e.g., propane gas cylinders, automobiles, and boats. These objects can cause devastating effects once they become water-borne missiles. Moreover, extended run-up duration of tsunami introduces the potential to transport floating bodies such as automobiles far inland, impacting buildings in their path. Since tsunami inundation fluctuates faster, there may be greater buoyant forces exerted on buildings. Rapid water level fluctuations induce pore-pressure gradients in the soil, which may loosen the foundations, causing liquefaction of soil in some cases. Tsunami characteristics and behaviour in the run-up zone are often unpredictable since they are influenced by the type of tsunami and also the surrounding topography (Yeh et al. 2005).

B.2 Existing Information in Codes of Practices

The codes classify areas that are subject to wave heights in excess of 1 m or high-velocity wave run-up or wave-induced erosion as *coastal high-hazard zones* or *V-zones*. The following loads must be considered in the design and construction of buildings and structures in V-zones: hydrostatic (vertical and lateral), hydrodynamic, impact, surge, wave, and breaking wave loads. The effects of long-term erosion, storm-induced erosion, and local scour are to be included in the design of foundations of buildings or other structures in coastal high hazard areas. Foundation embedment should be below the depth of potential scour. All buildings and structures in potential flood zones must be designed and constructed to resist floatation from floodwater at the regulatory flood elevation. Load factors for the building materials used should be the same as the building code provisions for wind or earthquake loads combined with gravity loads. In other words, loads and stresses due to tsunamis are to be treated in the same fashion as for earthquake loading. A brief introduction of the various loads due to tsunami, as available in the existing codes, is given in the following sub-sections.

B.2.1 Hydrostatic Force

It has been observed that hydrostatic forces are normally relatively small compared to surge and drag forces for the case of bore-like tsunamis, however for tsunamis that act as rapidly rising tide, the hydrostatic forces generally become increasingly important (Dames & Moore 1980). The following equation does not include the direct drag at the top of a wall, when the wall is less than h in height.

$$F_h = \frac{1}{2}\rho g[h + (u_p^2/2g)]^2 \tag{B.1}$$

where F_h is the hydrostatic force on a wall, per unit width of wall, ρ is the water density, g is the gravitational acceleration, h is the water depth, and u_p is the velocity component normal to the wall. The resultant force will act horizontally at a distance of h_R above the base of the wall where

$$h_R = \frac{1}{3}[h(u_p^2/2g)] \tag{B.2}$$

B.2.2 Bouyant Force

All codes (ASCE 7, FEMA 55, and CCH) provide the following expression for buoyant force

$$F_b = \rho g V \tag{B.3}$$

where V is the volume of water displaced by the building.

B.2.3 Design Flood Velocity

FEMA 55 and CCH provide the following estimate of the flood velocity u in the surge depth d_s, based on Dames and Moore (1980)

$$u = 2\sqrt{(gd_s)} \tag{B.4}$$

B.2.4 Hydrodynamic Force

Both CCH and FEMA 55 provide the following expression for the hydrodynamic force (drag force)

$$F_d = \frac{1}{2}\rho C_d A u_p^2 \tag{B.5}$$

where C_d is the drag coefficient and A is the projected area of the body on the plane normal to the direction of flow. The CCH recommends $C_d = 1.0$ for circular piles, 2.0 for square piles, and 1.5 for wall sections. FEMA 55 recommends $C_d = 2.0$ for square or rectangular piles and 1.2 for round piles. In addition, Table B.1 gives drag coefficients for larger obstructions.

Table B.1 Drag coefficient

Width to depth ratio (w/d_s or w/h)	*Drag coefficient C_d*
From 1–12	1.25
13–20	1.3
21–32	1.4
33–40	1.5
41–80	1.75
81–120	1.8
>120	2

B.2.5 Surge Force

CCH adopted the following equation for surge force

$$F_s = 4.5\rho g h^2 \tag{B.6}$$

where h is the surge height (Dames & Moore 1980). The resultant force acts at a distance approximately h above the base of the wall. This equation is applicable for walls with heights equal to or greater than $3h$. Walls whose heights are less than $3h$ require surge forces to be calculated using appropriate combination of hydrostatic and hydrodynamic force equations for the given situation.

B.2.6 Impact Force

CCH, FEMA 55, and ASCE 7 contain similar equations that resulted in the following generalized expression for impact force F_I acting at the still water level:

$$F_\mathrm{I} = m[du_b/dt] = m(u_\mathrm{I}/\Delta) \tag{B.7}$$

where u_b is the velocity of the impacting body, u_I is its approach velocity that is assumed equal to the flow velocity, m is the mass of the body, Δt is the impact duration that is equal to the time between the initial contact of the body with the building and the maximum impact force. CCH recommends Δt values for wood construction as 1.0 second, steel construction as 0.5 second, and reinforced concrete as 0.1 second. FEMA 55 provides the Δt values shown in Table B.2.

Table B.2 Impact duration recommended by FEMA 55

Type of construction	*Duration (t) of impact (sec)*	
	Wall	*Pile*
Wood	0.7–1.1	0.5–1.0
Steel	NA	0.2–0.4
Reinforced concrete	0.2–0.4	0.3–0.6
Concrete masonry	0.3–0.6	0.3–0.6

B.2.6.1 Breaking Waves Forces

Breaking wave loads on vertical piling and columns ASCE 7 and FEMA 55 provide the following expression for the breaking wave force F_{brkp}:

$$F_{\mathrm{brkp}} = \frac{1}{2}\rho g C_{\mathrm{db}} D H_b^2 \tag{B.8}$$

where C_{db} is a shape coefficient (ASCE 7 and FEMA 55 recommend C_{db} values of 2.25 for square or rectangular piles and 1.75 for round piles), D is the pile diameter, and H_b is the breaking wave height (FEMA 55 recommends $H_b = 0.78d_s$, where d_s is the design still-water flood depth).

Breaking wave loads on vertical walls The equations given by FEMA 55 and ASCE 7 differ slightly. FEMA 55 gives the following equations, which incorporate the lateral hydrostatic force. If these formulae are used, then the hydrostatic force should not be added.

Case 1 (enclosed dry space behind the wall)

$$F_{\mathrm{brkw}} = 1.1C_p\gamma d_s^2 + 2.41\gamma d_s^2 \tag{B.9}$$

Case 2 (equal still water level on both sides of wall):

$$F_{\mathrm{brkw}} = 1.1C_p\gamma d_s^2 + 1.9\gamma d_s^2 \tag{B.10}$$

where F_{brkw} is the total breaking wave load per unit length of wall acting at the still water level (d_s), C_p is the dynamic pressure coefficient from Table B.3, and γ is the specific weight of water.

Table B.3 Dynamic pressure coefficient recommended by FEMA 55

C_p	*Building type*	*Probability of exceedance*
1.6	Accessory structure, low hazard to human life or property in the event of failure	0.5
2.8	Coastal residential building	0.01
3.2	High-occupancy or critical facility	0.001

ASCE 7 suggests the following equations:

$$P_{\max} = C_p\gamma d_s + 1.2\gamma d_s \tag{B.11}$$

$$F_t = 1.1C_p\gamma d_s^2 + 2.4\gamma d_s^2 \tag{B.12}$$

where $P_{\max}$ is the maximum combined dynamic ($C_p\gamma d_s$) and static ($1.2\gamma d_s$) wave pressure, also referred to as shock pressure. F_t is the total breaking wave force per unit length of the structure, and is also referred to as shock, impulse, or wave impact force acting near the still water elevation (d_s). C_p is the dynamic pressure coefficient ($1.6 < C_p < 3.5$; see Table B.4), and d_s is the still water depth at base of the building, where the wave breaks. If free water exists behind the wall, the hydrostatic component of the wave pressure and force disappears and the dynamic wave pressure and the net force are computed by

$$P_{\max} = C_p \gamma d_s \tag{B.13}$$

$$F_t = 1.1 C_p \gamma d_s^2 \tag{B.14}$$

Table B.4 Dynamic pressure coefficient recommended by ASCE 7

Building Category	C_p
I (low hazard)	1.6
II (standard hazard)	2.8
III (Substantial hazard)	3.2
IV (essential facilities)	3.5

Loading Combinations Individual loading conditions must be applied to standard elements in appropriate load combinations. The following loading combinations are suggested by FEMA 55 (Yeh et al. 2005). The structural members considered are piles or open foundations, columns, walls, and basements. Outer edge beams at the second floor level are not mentioned because the reference codes are intended for smaller scale residential structures.

Type 1—Columns in tsunami prone areas (required):

The following combinations are used to calculate the force on a column from a tsunami with the additional impact force from debris.

F_{brkp} (on column) + F_{I} (on column) or F_d (on column) + F_{I} (on column)

Type 2—Solid walls facing the shoreline in tsunami prone areas:

Construction of non-breakaway walls or solid walls parallel to the shorelines is not recommended in structural designs. The following combinations provided are for walls, which are perpendicular to the flow of the tsunami. Tsunami effects on structural walls with additional impact force of debris.

F_{brkw} (on walls facing shoreline) + F_{I} (on one corner) or

F_s (on walls facing shoreline) + F_{I} (on one corner) or

F_d (on walls facing shoreline) + F_{I} (on one corner)

Type 3—Vertical (buoyant) forces on structure:

This loading combination is used when there is a sudden increase in the water level. The buoyant force on the structure must also be considered with other lateral forces.

F_b (for basements, swimming pools, empty above ground, and below ground tanks)

B.3 Comments

All of the equations presented earlier appear reasonable and might be applicable to tsunami cases. However, Yeh et al. (2005) have made the following comments based on recent analysis of the tsunami run-up.

(a) Equation (B.1)—representation of hydrostatic force—may not be relevant to a building with—finite breadth, for which the water can flow around and quickly fill up behind the building. Hydrostatic force is usually important for a 2-D structure such as seawalls and dikes or for evaluation of an individual wall panel, where the water level outside differs substantially from the level inside.

(b) The surge-speed estimate given by Eqn (B.4) was suggested by Camfield (1980) and is equivalent to a classic solution of the leading-tip of surge on a frictionless horizontal plane generated by breaking a dam with the quiescent impoundment depth of d_s. Hence, the computed tip velocity does not represent the velocity of flow depth d_s, and therefore this equation is not appropriate to represent the flow velocity for a tsunami flood passing through a structure.

(c) The surge-force computation by Eqn (B.6) may result in excessively overestimated values. Note that Eqn (B.6) was derived by summing the hydrostatic force and the change in linear momentum at the impingement of a surge front on a vertical wall. The surge impingement is treated as steady using the speed evaluated by Eqn (B.4). The estimation made by Eqn (B.6) implies that the surging force would be nine times the hydrostatic force alone. Such an excessively large surging force is contradictory to the laboratory results by Ramsden (1993). Further information about the loads to be considered for a tsunami resistant design may be found in the references.