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Prototype Building Tsunami Design Examples

I.N. Robertson(1) and S.A. Thomas(2)

(1) *Professor, University of Hawaii at Manoa, ianrob@hawaii.edu*
(2) *Structural Engineer, KPFF Consulting Engineers, seth.thomas@kpff.com*

Abstract

This paper will demonstrate the application of the new ASCE 7-16 Chapter 6, Tsunami Loads and Effects, when applied to prototypical buildings located in various coastal locations. Because this is a new chapter in ASCE 7, which previously has not included tsunami design, it is important to provide a comprehensive set of worked examples of typical building and structural element designs to demonstrate how all of the different design provisions are to be applied. The authors are responsible for the development of two design manuals to be published by ASCE in 2016 with extensive worked examples of tsunami design. Highlights of these examples will be illustrated in this paper and presentation.

In addition, the financial consequences of including tsunami loads and effects in the design of coastal buildings will be illustrated including cost impacts of the first US vertical evacuation structure. For multi-story reinforced concrete and structural steel buildings designed for moderate to high seismic loads, the tsunami loads will seldom govern the design of the lateral framing system. However, for low rise critical facilities the increase in lateral framing system strength and foundation systems will be more significant. In addition, local structural elements such as gravity load columns or shear walls on the exterior of the building may need to be enhanced. The tsunami loads that tend to control individual member designs on the exterior of the structure are the hydraulic drag force with debris accumulation against the building frame, and impact forces due to floating debris such as logs and shipping containers. Examples of these load calculations and member designs will be demonstrated, along with the implications for building cost.

Keywords: ASCE7-16, Tsunami Loading, Structural Design, Building Codes, Tsunami Design Examples

1. Introduction

The newly adopted ASCE 7-16 Tsunami Loads and Effects chapter will become the first US national, consensus-based standard for tsunami resilience of critical and essential facilities, Tsunami Vertical Evacuation Refuge Structures, and other multi-story building structures [1]. The tsunami design provisions provide a comprehensive design methodology based on the principles of probabilistic hazard analysis, tsunami physics, and fluid mechanics. They are specifically intended for use in the five Western US states exposed to a well defined tsunami threat, namely Alaska, Washington, Oregon, California and Hawaii. However, they can be utilized in any tsunami-prone community once the probabilistic tsunami hazard for that location has been established.

The ASCE 7-16 tsunami provisions require that critical and essential facilities, including Tsunami Vertical Evacuation Refuge Structures (Risk Category IV) and large occupancy structures (Risk Category III) that fall within the mapped Tsunami Design Zone (TDZ) must consider tsunami loading and effects. In addition, the provisions recommend that local jurisdictions include tsunami design requirements for taller general building structures (Risk Category II) so as to provide a last resort refuge for people stranded in the inundation zone without sufficient time to evacuate to high ground. Community resilience to future tsunamis would also be enhanced by requiring tsunami design of larger buildings that will provide the basis for reestablishing the community after the event.

As a new chapter in ASCE 7-16, focusing on fluid loading on structures, Chapter 6, Tsunami Loads and Effects, will address material that is relatively unfamiliar to many structural engineers and others who utilize the provision of ASCE 7-16. In order to demonstrate the application of these new tsunami design provisions to typical coastal buildings, the authors are working on design manuals that will aid those using these provisions for the first time. The first of these design manuals will explain the background to the tsunami design provisions and demonstrate their application to prototypical multi-story buildings in the TDZ [2]. The second manual will provide addition example applications of the ASCE 7-16 tsunami design provisions to critical and essential facilities including vertical evacuation structures [3].

This paper presents example applications of the ASCE 7-16 tsunami provisions to prototypical multi-story reinforced concrete buildings, and a vertical evacuation building recently completed in Oregon.

2. Prototypical Buildings

5.1 Reinforced Concrete Buildings

Two prototypical reinforced concrete buildings with different structural systems were analyzed and designed for wind and seismic conditions at three locations: Hilo, Hawaii; Waikiki, Hawaii; and Monterey, California [4]. They were then subjected to tsunami loading appropriate for their location to determine whether strengthening was required, and how much impact this might have on the construction cost.

One building is a six story office building consisting of exterior moment resisting frames (MRF), a flat plate floor system, and interior gravity load resisting columns (Figure 1). When located in Hilo and Monterey this building required perimeter and interior special moment frames for seismic design, while the prototypical building located in Waikiki has a perimeter intermediate moment frame. The other building is a seven story residential building consisting of shear walls (SW) at elevators and stairwells, a flat plate floor system, and gravity load resisting columns (Figure 2). For the Hilo and Monterey locations special reinforced concrete shear walls were required for the seismic design, while the Waikiki building has ordinary reinforced concrete shear walls.

Similar design examples were generated for prototypical structural steel buildings [5], but are not presented here because of space limitations.

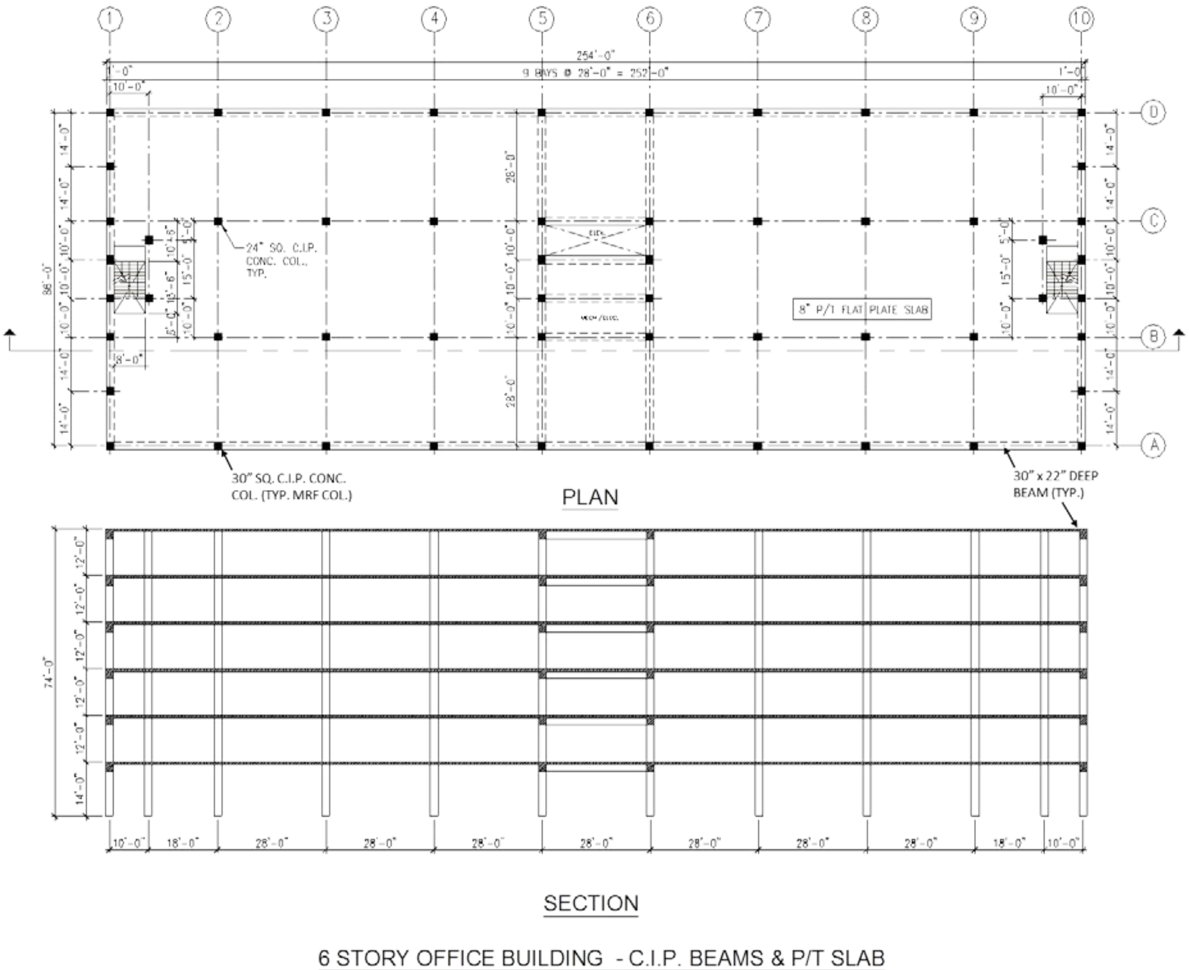

Figure 1 - Prototype 6-Story Reinforced Concrete MRF Office Building Plan and Section

3. Seismic and Wind Design

The prototype buildings were designed for various levels of seismic and wind loads per ASCE 7-10 following standard procedures. The lower stories of each building were designed and detailed following ACI 318-08 seismic design provisions [6]. Experience has shown that most multi-story buildings exceeding 65 feet height that have been designed for high seismic conditions, will be able to withstand tsunami loads with relatively little increase in structural member properties [7, 8]. Carden, et al. [9] show that the seismic structural system of multi-story buildings provides considerable capacity for tsunami lateral loads, though it is still necessary to evaluate individual members for the appropriate loading conditions.

4. Tsunami Flow Parameters

Because these are Risk Category II buildings, the Energy Grade Line (EGL) analysis can be used to determine the maximum flow depth and velocity at the building location. The EGL method is based on a step-wise calculation of the energy in the tsunami flow, starting at the runup point and progressing towards the shoreline [10]. The energy is accumulated based on the topographic slope, and the equivalent slope due to surface friction. At any location along the EGL, the total energy is a combination of potential energy and kinetic energy. Because there is only one energy equation but two unknowns, depth h, and velocity, u, it is necessary to know the relationship between these two to solve the problem. This relationship is the Froude number, $F_r = u/\sqrt{gh}$, which decreases from a maximum at the shoreline to zero at the runup location. Further information on the EGL is provided in a companion paper [11].

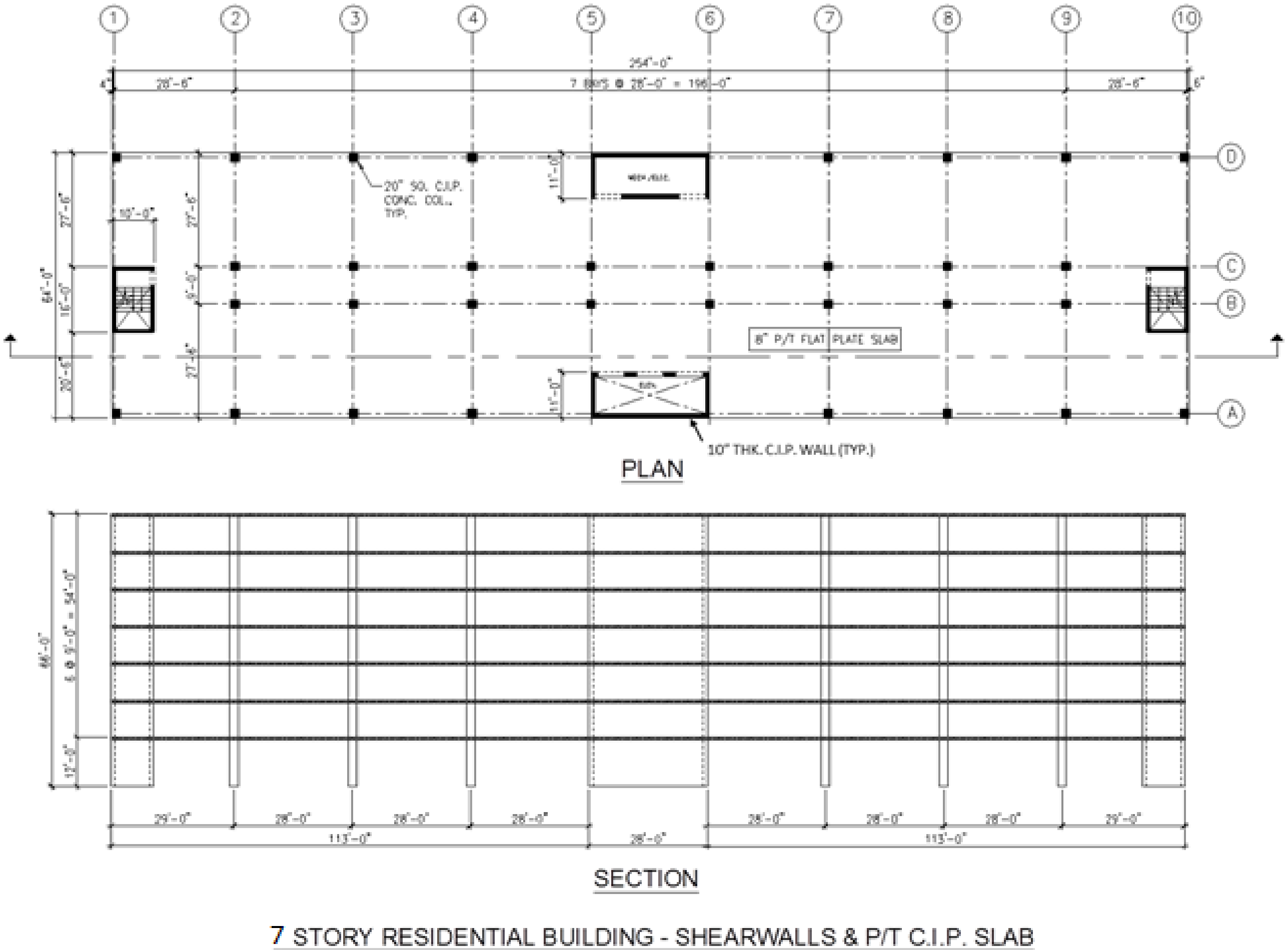

Figure 2 - Prototype 7-Story Reinforced Concrete Shear Wall Residential Building Plan and Section

The EGL is applied along a transect perpendicular to the shoreline, and two transects rotated 22.5^{o} either side of this primary transect. Figure 3 shows these three transects for the Hilo building location. Note that the transects are rotated about the building location. The EGL results for flow depth and flow velocity for all three Hilo transects are shown in Figure 4 and Figure 5, respectively. The largest results at the project site are to be used as h_{max} and u_{max} for the structural analysis. Similar analyses were performed for the other locations. Table 1 lists the maximum velocity and flow depth for each building location determined using the Energy Grade Line Method.

Table 1: Velocity and Flow Depths for Building Location

Flow Parameters	Hilo	Waikiki	Monterey
Max. Inundation Depth, h_{max} (ft)	55	25	13
Max. Flow Velocity, u_{max} (ft/s)	35.8	28	18

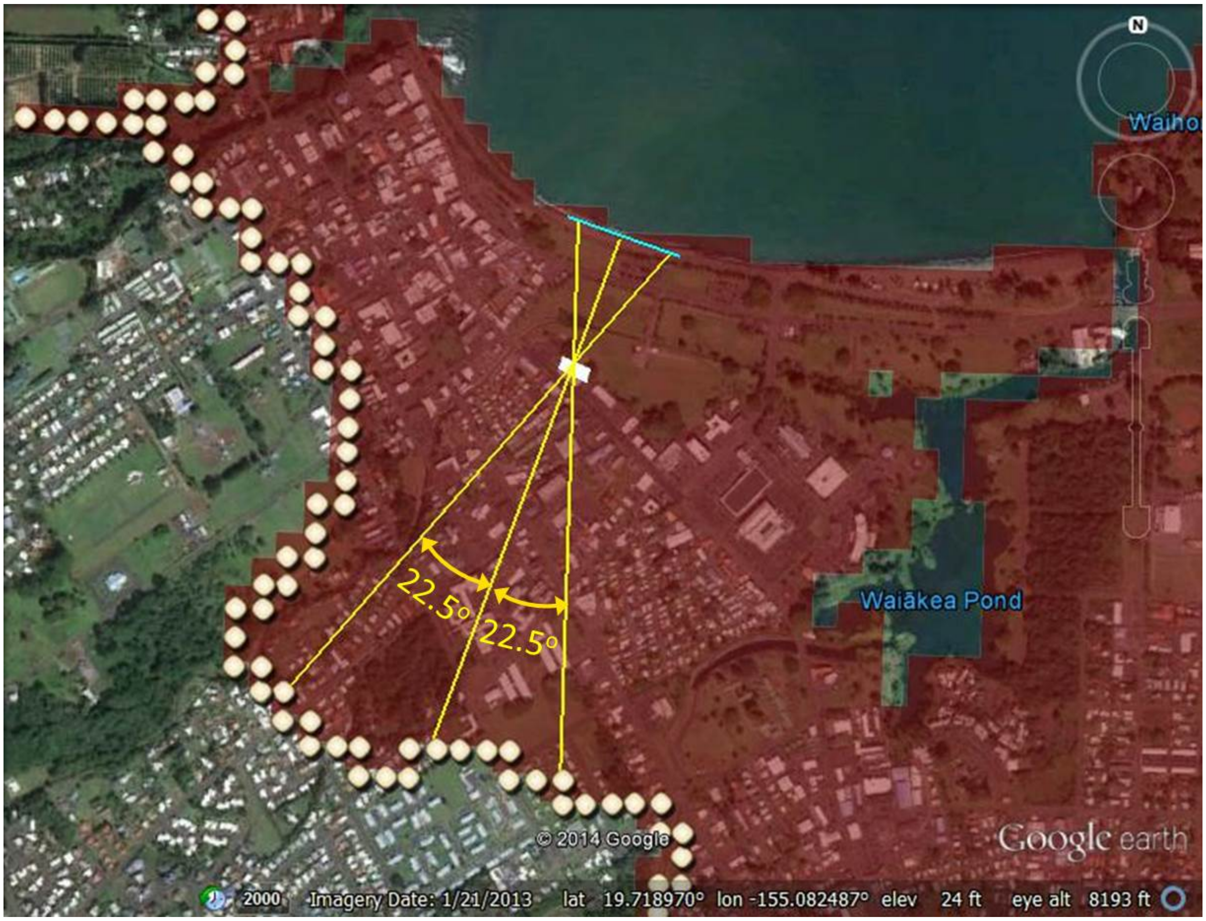

Figure 3 – Energy Grade Line transects for Hilo Bay building location

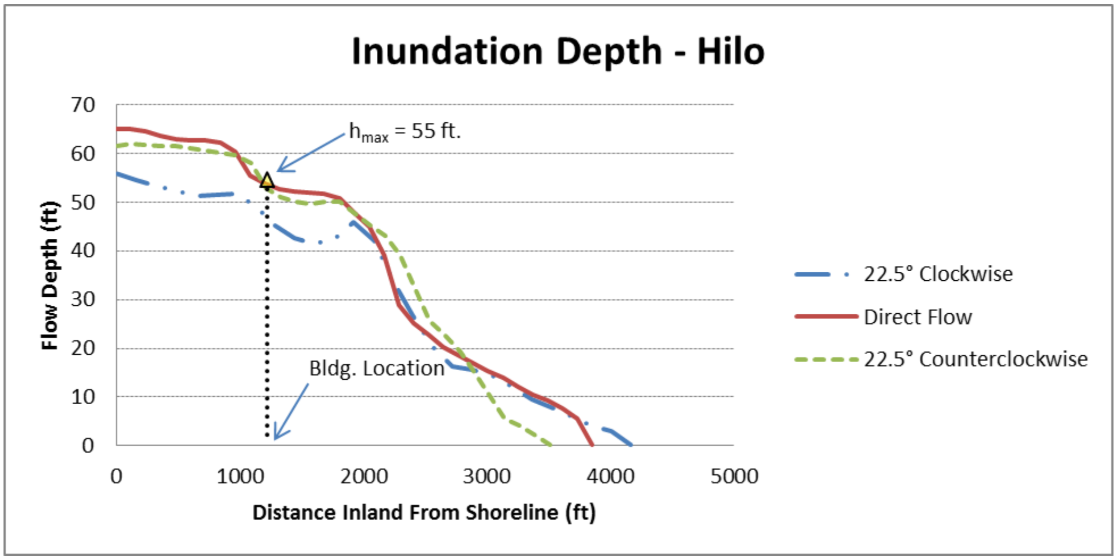

Figure 4: Inundation Depths for Hilo Transect Lines

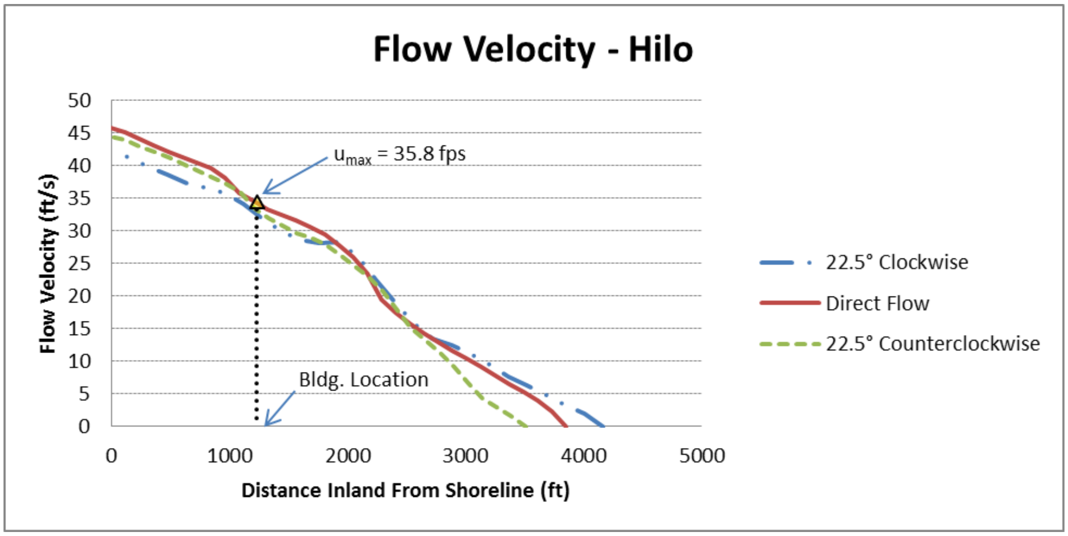

Figure 5 - Flow Velocities Based on Distance from Shoreline for Hilo Transect Lines

5. Tsunami Design for Residential Building at Monterey Site

5.1 Overall Building Forces

ASCE 7-16 requires that three Load Cases be considered in the tsunami design of a coastal building. These load cases will be demonstrated using the residential building located in Monterey, CA. Load Case 1 is a buoyancy check for enclosed spaces or buildings with basements or structural ground floor slabs where hydrostatic pressure can develop in the soil below the slab. For the prototype buildings it was assumed that the ground floor slab is a non-structural slab-on-grade that is isolated from the columns. Hydrostatic pressure below this slab may lead to slab uplift, but will not affect the structural integrity of the building.

Load Case 2 assumes that the maximum flow velocity (18 ft/s) occurs when the inundation depth is $2/3h_{max}$ = 2/3 x 13.0 = 8.67 ft. The hydrodynamic drag on the entire building is given by:

$$F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2)$$

Where ρ_s = 1.1 x 2.0 = 2.2 slugs/cuft (seawater density including sediment and debris)
I_{tsu} = 1.0 (Importance factor for TRC II buildings)
C_d = 1.4 (ASCE 7-16 Table 6.10-1 based on B/h_{sx} = 254/8.67 = 29.3)

$$C_{cx} = \frac{\Sigma(A_{col} + A_{wall} + A_{beam})}{Bh_{sx}} = \frac{\Sigma((2' \times 8.67' \times 32) + (2 \times 10' \times 8.67' + 2 \times 28' \times 8.67') + 0)}{254' \times 8.67'} = 0.551 < 0.7$$

Therefore $C_{cx} = 0.7$ controls
B = 254’ overall width of building
h = 8.67’
$u = u_{max}$ = 18.0 fps (for load case 2)

Substitution gives:

$$F_{dx} = \frac{1}{2}\rho_s I_{tsu} C_d C_{cx} B(hu^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 1.4 \times 0.7 \times 254(8.67 \times 18.0^2)/1000 = 769\ kips$$

Load Case 3 occurs when the inundation depth is h_{max} = 13.0 ft. and the flow velocity is $1/3u_{max}$ = 1/3 x 18.0 = 6.0 fps. The resulting lateral load on the building is F_{dx} = 119 kips.

The controlling lateral load (769 kips) is applied as a uniformly distributed exterior pressure on the coastal elevation of the building over a height of 8.67 feet above grade. The lateral force resisting system for the structure at the first floor level is evaluated for this load.

5.2 Evaluation of Lateral Force Resisting System

Because the structure has been designed for Seismic Design Category D, ASCE 7-16 permits the use of $0.75\Omega_o E_h$ to evaluate the lateral force resisting system (LFRS), where E_h is the seismic base shear and Ω_o is the overstrength factor . From the seismic design of this structure, E_h = 3,313 kips and Ω_o = 2.5. Therefore;

$$0.75\Omega_o E_h = 0.75 \times 2.5 \times 3{,}313 = 6{,}212\ kips.$$

For this example, the controlling load case for overall building tsunami lateral load is LC2, with F_{dx} = 769 kips applied over a height of 8.67 ft. The majority of this load will be resisted by the grade beam/foundation system, with only about 238 kips transferred to the second floor slab. Because this is well below the 6,212 kip capacity, the lateral force resisting system has ample capacity to resist the overall tsunami loads.

5.3 Simplified Equivalent Uniform Lateral Static Force

In lieu of performing detailed hydrostatic and hydrodynamic analysis, ASCE 7-16 provides an optional simplified but conservative estimate of the maximum lateral load on the building, given by;

$$f_{uw} = 2.5 I_{tsu} \gamma_s h_{max}^2 = 2.5 \times 1.0 \times (1.1 \times 64.0) \times 13.0^2 = 29.7\ kip/ft$$

With $C_{cx} = 0.7$, the total force on the structure is $F = 0.7 \times 254 \times 29.7 = 5{,}280\ kips$. This lateral load is compared with 0.75 Ω_o E_h = 0.75 x 2.5 x 3,313 = 6,212 kips > 5,280 kips. Therefore the LFRS is adequate to satisfy this requirement and the detailed analysis for LC2 and LC3 shown above is not necessary. The components can also be designed on the basis of this conservative uniform distributed force with the appropriate width *b* dimensions but this would yield very conservative results. This simplified approach in not used here, but rather the detailed loading approach is used for the individual component design in the following section.

6. Component Design

6.1 Drag Force on Components

All structural members below the maximum flow depth must be designed for hydrodynamic drag forces, including the effect of debris accumulation on the exterior of the building. For example, for exterior columns, the tributary width must be taken as $b = C_{cx}$ x column spacing = 0.7 x 28 = 19.6 ft. The controlling load case will be LC2, when the inundation depth is h_e = 8.67 ft and u_{max} = 18.0 fps. The hydrodynamic drag is computed as:

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b(h_e u^2) = \frac{1}{2} \times 2.2 \times 1.0 \times 2.0 \times 19.6(8.67 \times 18.0^2)/1000 = 121\ kips$$

This load is applied to the column as an equivalent uniformly distributed lateral load of 121/8.67 = 14.0 kips/ft over the lower 8.67 feet of the column. The column must be designed for this load combined with gravity loads using the appropriate load combination.

For interior columns, the hydrodynamic drag is applied to the column only, without any debris accumulation. The resulting LC2 load of 10.3 kips is applied to the column as an equivalent uniformly distributed lateral load of 10.3/8.67 = 1.19 kips/ft over the lower 8.67 feet of the column. This load must be combined with gravity loads and the column capacity verified. Similar hydrodynamic loads act on structural walls unless the location is subjected to potential tsunami bores, in which case the hydrodynamic drag is increased by a factor of 1.5 to account for the impulsive load that can develop on relatively wide walls.

6.2 Debris Impact Loads

According to the ASCE7-16 tsunami provisions, if the inundation depth at the building exceeds 3 feet, then exterior structural elements below the flow depth must be designed for debris impact loads. In lieu of detailed debris impact analysis, the member can be designed for the maximum static load given as F_i = 330I_{tsu} = 330 kips. Since the building location is not in an impact zone for shipping containers, ships, and barges, this force can be reduced by 50% to 165 kips. When applied as a static load to the 24" square exterior columns of the Moment Resisting Frame building, the column required no strengthening. However, when this load was applied as a static load to the 20" square exterior columns of the Shear Wall building, the column was inadequate and had to be increased to a 24" square column for the first floor. Figure 6 shows the interaction diagram for the typical exterior gravity load column including the tsunami load combinations.

The debris impact load need not be combined with other tsunami loads and it need not be applied to interior columns. The provisions also allow for a dynamic analysis of this impact loading instead of applying the load statically. Because of the very short impact duration (on the order of a few milliseconds), this can often result in satisfactory performance for smaller columns. Debris impact loads must also be applied to any structural walls or beams located on the exterior of the building. Note that beams built integrally with concrete floor slabs will have significant bracing against lateral loads, while isolated beams may require strengthening to resist the horizontally applied hydrodynamic and impact loads.

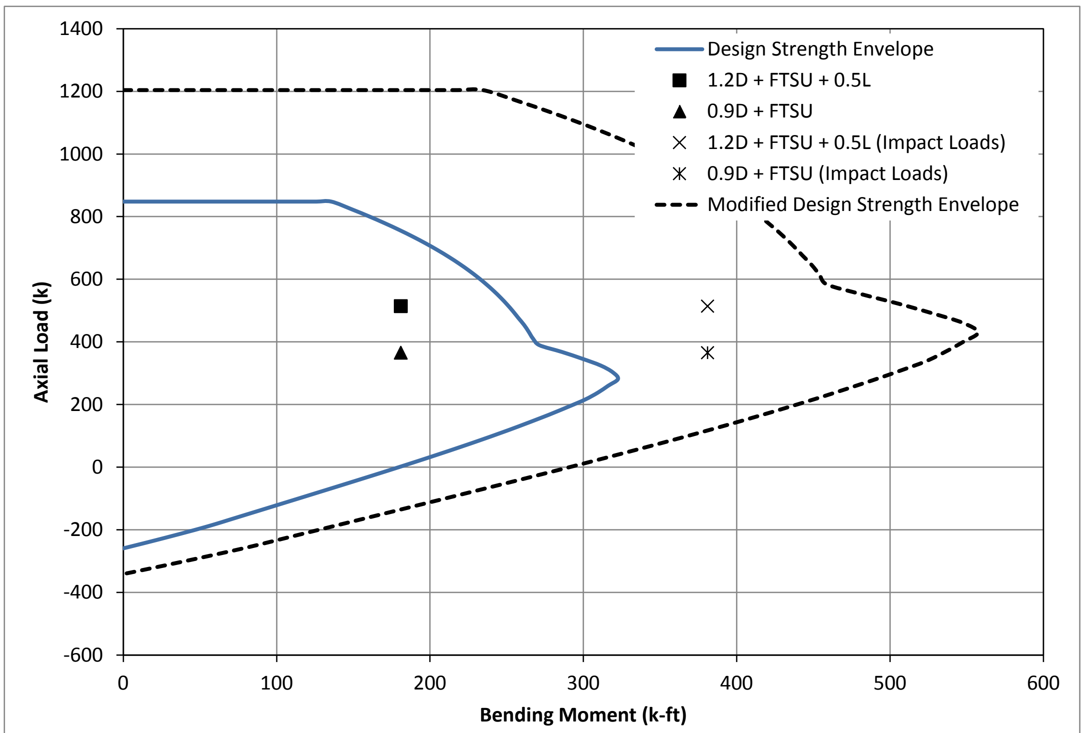

Figure 6: Interaction diagram for exterior gravity load column showing tsunami load combinations

6.3 Scour Effects

Sediment transport and scour around building foundations can result in localized and overall structural failure. Estimation of scour based on sediment transport modeling is not yet at a level where it can be recommended for design of structures. The ASCE7-16 provisions are therefore based on an empirical expression derived from field observations after numerous past tsunamis. The foundation design is required to consider at least two tsunami waves. After the first wave, the predicted scour must be assumed to have occurred around the perimeter of the building foundation. During the second wave, the structural system, including scoured foundations, must be able to resist all tsunami loads described earlier.

The ASCE7-16 Tsunami Loads and Effects provisions also provide requirements for the design of countermeasures to reduce the potential for geotechnical failures. A companion paper provides more detailed information about the design requirements for scour and sediment transport, as well as soil remediation measures to limit the scour potential [12].

6.4 Economic Consequences of Tsunami Design

Based on the material and labor required to meet the tsunami loading requirements, the additional cost for the 6- and 7-story prototypical building designs is estimated to result in an increase of between 5 and 15 percent of the structural system construction cost. Given that the structural system is generally on the order of 25 to 30 percent of the total cost of a building, this implies only a 1 to 5 percent increase in overall building construction cost to incorporate tsunami design. It should be noted that this economic analysis assumed that the buildings were already constructed on robust deep foundations, which did not require upgrading. If the wind and seismic design were such that only shallow footings were required, then additional expense would be incurred in upgrading these foundations for tsunami scour effects. It would also be expected that taller buildings would require less upgrading for tsunami design, while lower rise buildings would require more expensive upgrades.

7. Tsunami Vertical Evacuation Refuge design example.

The Ocosta school district in Washington State is the first location to implement the new ASCE 7 tsunami provisions for design of a Tsunami Vertical Evacuation Refuge at the new Ocosta Elementary School. In 2013 the Ocosta school district passed a bond issue to raise funds to replace their aging elementary/middle school with a new building that would also serve as the community's tsunami refuge. A specific requirement of the bond measure was that the new school building had to serve as a designated tsunami vertical evacuation refuge for the students and staff of the elementary/middle school itself, the adjacent high school, and the local residents. This meant that the structure needed to accommodate 1,000 potential refugees (Figure 7).

Since the new school would be located in a potential inundaton zone from a Cascadia Subduction Zone earthquake and tsunami, the school district retained a design team familiar with tsunami modeling and design. Members of this team were familiar with the concurrent effort by the ASCE 7 Tsunami Loads and Effects committee to develop comprehensive tsunami design criteria, so were able to utilize draft versions of those criteria during design of the Tsunami Vertical Evacuation Refuge structure.

The schematic design phase occurred prior to the completion of the 2,500 year probabilistic tsunami design zone maps so the design team perrformed site specific tsunami inundation modeling using the DOGAMI "L" tsunami as the design event. The school site is located on the inland side of a relatively low level peninsula. The design tsunami overtops the high ground on the peninsula with a flow depth of approximately 3 feet, but then the flow accelerates down the inland side of the peninsula to the school site. There was concern that erosion of the high ground during the tsunami could cause increased flow at the site. The high ground was therefore reduced to approximate the anticipated scour resulting in increased inundation depth at the site. The refuge elevation was then based on the ASCE 7-16 requirements that the calculated inundation elevation be increased by 30% and the refugee elevation be at least 10ft above the resulting inundation elevation.

In order to meet budget requirements, it was decided that the new school building would consist of two parts. The ground level classroom wing would be constructed of light steel framing while a separate building containing the gymnasium, cafeteria, and music rooms would support the tsunami refuge area on the roof. Because the vertical evacuation structure would accommodate all school occupants, there was no need to design the classroom wing for tsunami loads. A seismic joint separates the two buildings to ensure that collapse of the classroom wing would not damage the refuge structure.

Based on the elevation of the site and the required clear height of a gymnasium plus the expected roof structural depth, no additional height was needed to achieve the target tsunami refuge elevation. Based on consideration of the required ingress/egress requirements for 1,000 refugees to reach the roof refuge during the short warning time for a near-source Cascadia tsunami, it was determined that 4 stair cases would be provided, one at each corner of the rectangular refuge structure. Some of the stairs are accessible from the exterior of the building for students from the high school and residents of the local community to gain access to the refuge. Each staircase is surrounded by reinforced concrete walls to protect the stairs from damage during the tsunami, and to provide a high capacity lateral framing system for both seismic and tsunami loads. Gravity columns supporting the roof framing were structural steel wide flange sections encased in reinforced concrete for the lower 14 feet to provide enhanced capacity to resist the hydrodynamic and debris impact loads (Figure 8).

The rest of the perimeter wall was constructed using lightguage steel framing designed to ‘breakaway’ from the structure so as to reduce the tsunami loads on the overall structure. Debris damming was included in the hydrodynamic drag as required by ASCE7-16, but the resulting tsunami lateral load was still less than the seismic base shear and overturning moments which were driven by the large mass of the roof slab.

The roof was designed as a concrete slab on metal deck supported on wide flange beams spanning the full width of the gynasium. The beams in turn were supported on girder trusses spanning between the shearwalls and supported by intermediate steel columns. Because the site was not located near a port facility or shipping container storage area, the impact of shipping containers was not considered. The design impacts came from large trees and floating vehicles. In order to optimize the concrete encased steel column design, the debris impacts were considered as dynamic loads as permitted by the ASCE7-16 provisions. The column response was calculated using a work-energy approach which reduced the effective impact force by more than 70% from the static load alternative. This approach is reasonable as the column in its deformed shape could still support the design gravity loads. Any damaged columns can be replaced after a tsunami event to restore the original integrity of the refuge structure.

To protect against unanticipated debris loading, the roof structural framing was also designed to prevent progressive collapse in the event of loss of one of the exterior columns. The staircase shearwalls are capable of resisting the extraordinary debris impact loads with the expectation that there may be some localized damage at the impact point. The roof girders are connected to the walls well above the potential impact area so they were not subjected to any tsunami loads. All curtian wall and non-structural connections to the roof were desgined to ‘break-away’ during a tsunami to limit the loads transfered to the structure.

Figure 7 – Rendering of Ocosta School gymnasium with Tsunami Vertical Evacuation Refuge on roof.

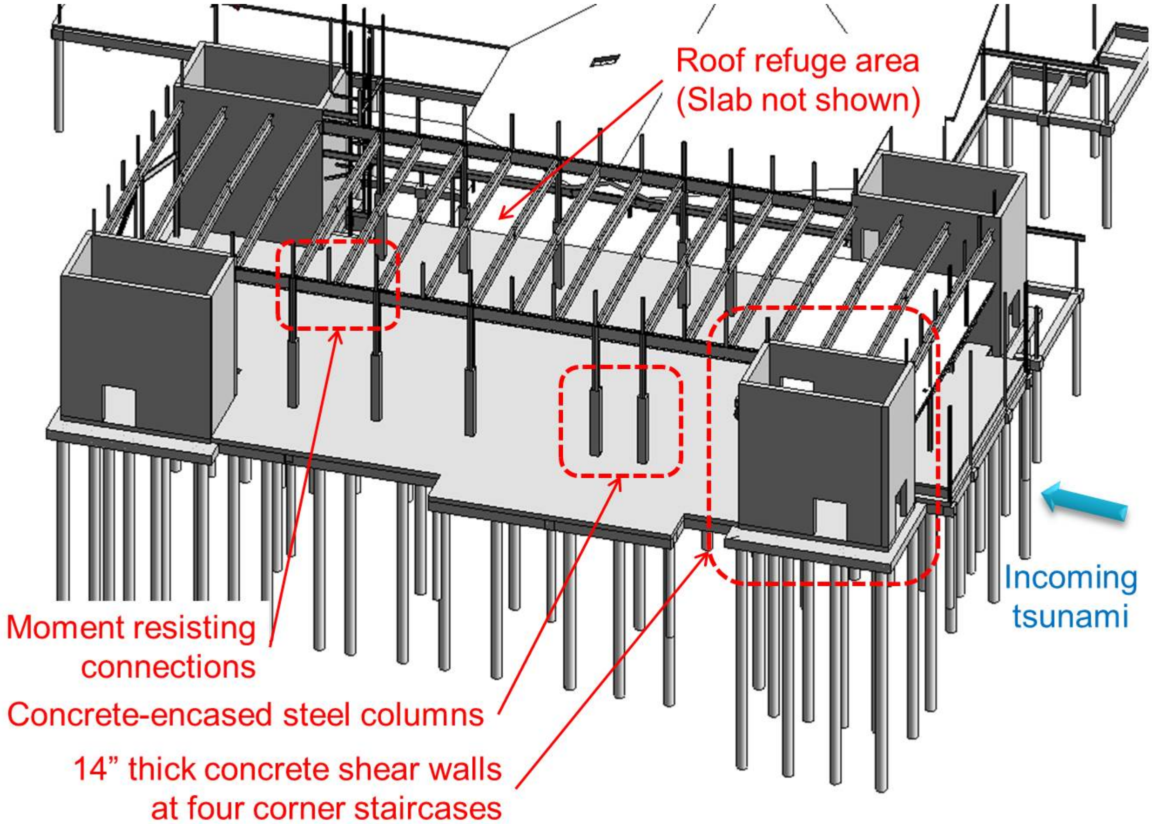

Figure 8 – Structural framing for Ocosta School Tsunami Vertical Evacuation Refuge

The shearwalls are supported on large pile caps with an arrangement of deep piles reaching through the liquefiable layers of soil. Piles would have been required for the seismic design, but additional piles were required to provide lateral support for the shearwalls. In addition piles were required to resist net uplift due to buoyancy when evaluating Load Case 1. The slab on grade between the pile caps was designed as a structural slab with intermediate piles to support the slab during earthquake induced liquefaction. This structural slab also ties the pile caps together laterally and helps to shield most of the piles from scour. However, scour must still be considered for the exterior piles, especially at the building corners where the shearwall pile caps are located. As a conservative assumption, the piles at the edge of the building that are exposed due to scour were ignored when designing for the tsunami load cases.

The addition of a tsunami vertical evacuation structure increased the cost of the project by about 25 percent. This is primarily because the gymnasium roof would have been a much lighter structure with only minimal stair access for maintenance. The four large staircases, heavy roof structure and additional foundation piles were major contributors to this increased cost.

8. Summary and Conclusions

This paper presents an overview of a number of tsunami design examples applied to prototypical reinforced concrete multi-story buildings located within the Tsunami Design Zone in Monterey, CA, Waikiki and Hilo, HI. The first reinforced concrete prototypical building consisted of a 6-story office building with a Moment Resisting Frame lateral system around the exterior of the building, and interior gravity load columns supporting flat slab floors. The second reinforced concrete building was a 7-story residential structure consisting of shear walls to provide the lateral load resistance, and gravity columns supporting flat slab floors.

When designed for high seismic conditions, as are typical of all five US western states, these buildings performed very well during tsunami loading. For example, the MRF buildings required no structural enhancement when located in Monterey, CA, or Waikiki, HI, and required only minor upgrades if located in Hilo, Hawaii. The shear wall buildings were adequate for overall lateral loading considerations, but required upgrading of the exterior gravity load columns to resist the effects of enhanced hydrodynamic drag due to debris damming. The exterior columns and any structural walls located on the exterior of the building would also require upgrading for debris impact, especially if the building is located within close proximity to a shipping container handling or storage facility due to increased shipping container impact forces.

The ASCE 7-16 tsunami design provisions were also used to design the first Tsunami Vertical Evacuation Refuge in the United States at Ocosta Elementary School, Washington State. Because this building is a single story structure with the flat roof designated as the tsunami refuge area, the cost of adding tsunami design was more significant than for the 6- and 7-story prototypical structures. Nevertheless, the surrounding community was willing to support the additional cost because of the added tsunami evacuation option it provides for students of the Ocosta Schools, as well as neighboring residents.

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