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PROTOTYPE BUILDING DESIGNS AND COST IMPLICATIONS OF INCORPORATING TSUNAMI DESIGN

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Abstract

This paper will demonstrate the application of the new ASCE 7-16 Chapter 6, Tsunami Loads and Effects, to prototypical buildings located in various coastal locations in the US. 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 lead author is finalizing a design manual to be published by ASCE in 2020 with extensive worked examples of tsunami design. Highlights of these examples will be illustrated in this paper and presentation.

Two reinforced concrete prototypical buildings were considered at four coastal locations, namely Seaside, Oregon; Monterey, California; Waikiki, Hawaii; and Hilo, Hawaii. The first building is a 6-story office building with reinforced concrete moment resisting frames, while the second is a 7-story residential building with reinforced concrete shear walls. Both buildings were designed for the wind and seismic requirements at each of the four locations, and then subjected to the appropriate tsunami loads specified by ASCE 7-16. For the Monterey, California location, the seismic structural system was adequate to resist the tsunami loads, so only minor strengthening was required for some of the exterior columns. For the Hilo, Hawaii location, the 2500-year design flow depths exceed the fifth floor of the building, leading to the need for more significant increases in both the lateral load resisting system and individual members. The other two locations fall between these two extremes.

The financial consequences of including tsunami loads and effects in the design of these coastal buildings will be illustrated. For mid- to high-rise reinforced concrete and structural steel buildings designed for 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 on 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 are demonstrated for the prototypical buildings, along with the financial implications for the overall building cost.

Keywords: Tsunami Design; ASCE 7-16; Prototypical building designs; Structural Design; Tsunami Loads

1. Introduction

The Indian Ocean Tsunami in December 2004 caused extensive damage to communities around the Indian Ocean and resulted in the loss of over 240,000 lives. This event triggered a significant shift in tsunami preparation, warning systems and research. Prior to 2004, most tsunami research was focused on open ocean propagation of tsunami waves in order to predict better the arrival time and the potential size of the tsunami wave along affected shorelines. The focus was primarily on saving lives in coastal communities through horizontal evacuation to high ground. Primarily because of the experience of communities in low-lying regions such as Banda Aceh, with limited access to high ground near the shoreline, and short warning time for wave arrival, further attention was focused on the potential for vertical evacuation into buildings or other structures designed to resist tsunami loads. This led to the development of FEMA P-646: Guidelines for Design of Structures for Vertical Evacuation from Tsunamis by the Applied Technology Council [1].

The Maule Tsunami of February 27, 2010, and the Tohoku Tsunami of March 11, 2011 caused tremendous damage to many coastal buildings, bridges and port facilities in Chile and Japan, respectively. However, a number of larger concrete and structural steel buildings survived with only non-structural damage, particularly if they had been designed for high seismic conditions. Field surveys following these events and analysis of survivor videos have provided a wealth of information on the tsunami flow characteristics and structural loading that need to be considered in the design of coastal buildings [2, 3]

Starting in February 2011, the ASCE Tsunami Loads and Effects Subcommittee worked for four and a half years to develop a new chapter for inclusion in the ASCE 7-16 Standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures [4]. This new Chapter 6, Tsunami Loads and Effects, provides comprehensive provisions for design of coastal structures for tsunami loads, scour and related considerations.

ASCE 7-16 has since been adopted by the International Code Council for inclusion by reference in the 2018 version of the International Building Code [5] used throughout the United States of America. The tsunami design provisions will apply to all coastal communities in California, Oregon, Washington State, Alaska and Hawaii. A companion design manual has been developed by the first author to explain the new provisions and demonstrate their application to prototypical reinforced concrete and structural steel buildings in coastal communities in the Western USA [6].

The ASCE 7-16 standard only requires tsunami design for Tsunami Risk Category (TRC) III (critical and high occupancy buildings) and TRC IV (essential facilities). Tsunami design is not required for the majority of buildings that fall into TRC II, such as those used for residential, commercial and industrial purposes, for example. However, local jurisdictions are encouraged to consider requiring tsunami design for mid- to high-rise TRC II buildings so as to provide a “refuge-of-last-resort” for those who are not able to evacuate to high ground prior to tsunami arrival. The survival of taller buildings will also enhance the ability of the community to recover after the tsunami, thus increasing their resilience.

The study reported here was performed to establish an understanding of the likely financial consequences of requiring tsunami design for mid- to high-rise TRC II buildings so that local jurisdictions could make informed decisions about whether or not to require tsunami design for taller TRC II buildings in the Tsunami Design Zone.

2. Prototypical Building Design

A reinforced concrete prototypical residential building was developed for use in this study. The initial layout and structural configuration of this building were determined with the assistance of Gary Chock, president of Martin & Chock, Inc., a Honolulu structural engineering consulting company, to ensure that it represents a realistic mid-rise residential building. This Risk Category II building was developed with sufficient height above grade (over 65 feet (20 m) as suggested for severe tsunami hazard regions in the Commentary to ASCE 7 Chapter 6) to provide last-resort refuge for those caught in the tsunami inundation zone without sufficient time to evacuate to high ground, and to increase community resilience by ensuring that larger buildings remain intact except for non-structural components at the lower levels.

The prototypical building was located near the shoreline at four different locations in the U.S., with its broad dimension facing the incoming tsunami flow. Figure 1 shows the Hilo, Hawaii, location in the ASCE

7-16 Geodatabase [7], along with the three transects that must be considered to determine flow conditions at the site. The building is located about 1100 feet (335 m) from the shoreline. The other locations are Seaside Oregon, Monterey California, and Waikiki Hawaii. The latitude and longitude of each of the building locations are given in Table 1. The prototype residential building is a seven story building consisting of flat plate post-tensioned concrete floor slabs supported by gravity load columns. Lateral load resistance is provided by reinforced concrete shear walls.

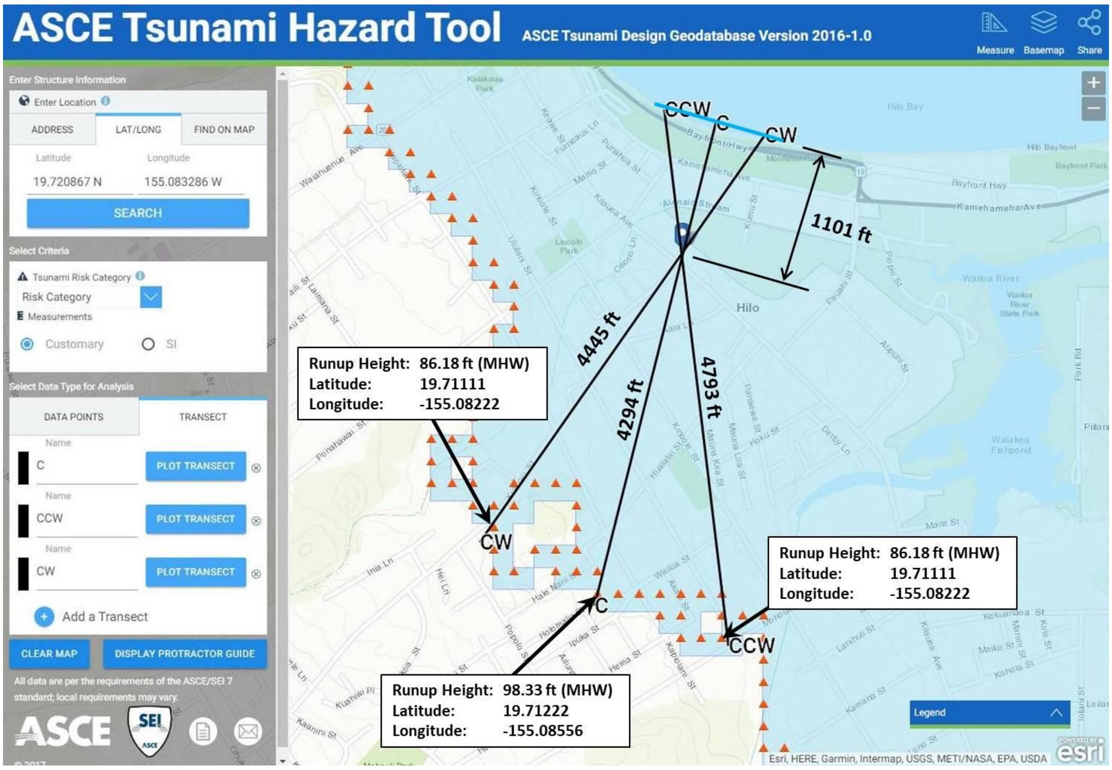

Figure 1: Hilo location for prototype building (1 ft = 0.305 m)

Figure 2 shows the floor plan and section for this building. The Hilo, Monterey and Seaside buildings have special reinforced concrete shear walls at the elevator core, mechanical room and staircases. Typically the elevator and mechanical room would be located in the interior of the building, however here they have been located on the exterior to incorporate the effect of debris impact on the structural walls. Because of the lower seismic demand at the Waikiki location, that building has ordinary reinforced concrete shear walls.

3. Wind and Seismic Design

The prototype buildings were located in four coastal communities with varying wind, seismic and tsunami loading conditions. Table 1 lists the locations with associated latitude and longitude, and design wind speeds according to ASCE 7-16. Table 2 lists the seismic design criteria at each site assuming soil classification D for "stiff soil". The Waikiki site has a lower seismic demand than the other sites, and is classified as Seismic Design Category (SDC) C based on local Honolulu City and County amendments to the ASCE 7 requirements [8].

Table 1: Prototype building locations and associated design wind speeds

Location	Latitude	Longitude	Wind Speed
Monterey, CA	36.6002 N	121.8818 W	110 mph (177 kph)
Seaside, OR	45.9948 N	123.9295 W	110 mph (177 kph)
Hilo, HI	19.7209 N	155.0833 W	130 mph (209 kph)
Waikiki, HI	21.2755 N	157.8255 W	130 mph (209 kph)

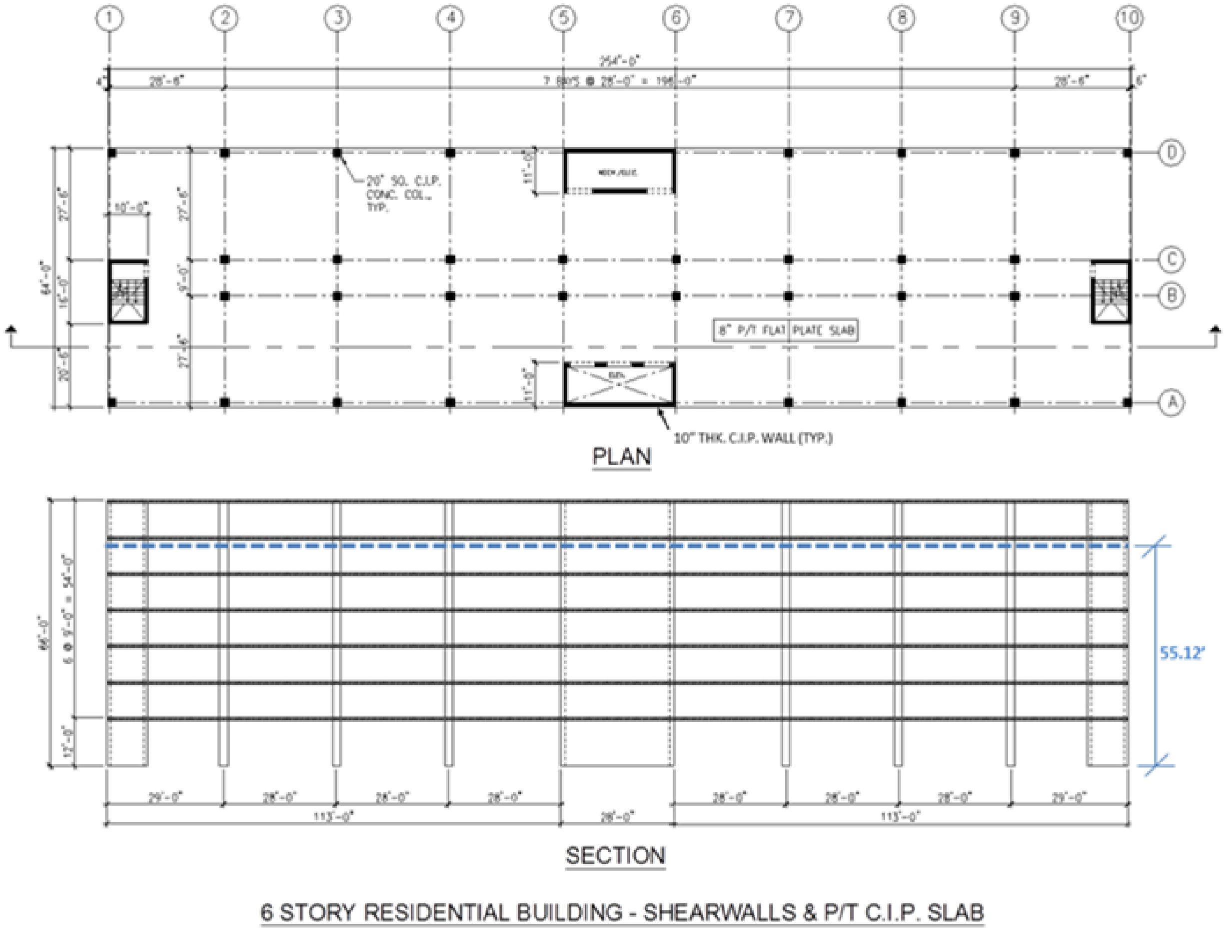

Figure 2: Plan and section views of the prototype 7-story residential building with reinforced concrete shear walls and posttensioned flat slabs supported on gravity columns. The indicated maximum flow depth is for the Hilo, Hawaii location.

Table 2: Prototype building seismic design criteria

Location	Site Class	SDC	S_S	S_1	S_{DS}	S_{D1}
Monterey	D	D	1.513g	0.554g	1.009g	0.554g
Seaside	D	D	1.332g	0.683g	0.888g	0.683g
Hilo, HI	D	D	1.500g	0.600g	1.000g	0.600g
Waikiki, HI	D	C	0.579g	0.170g	0.516g	0.240g

Each prototype building was analyzed and designed for the wind and seismic conditions at their respective locations. They were then evaluated for the tsunami loads required by ASCE 7-16 for all four sites [9]. This paper will demonstrate the tsunami design procedure for the Hilo site, but will report the overall results for all four sites.

4. Tsunami Loading

The Hilo, Hawaii location will be used to demonstrate the tsunami loading on the prototypical residential building. Only a brief overview of the tsunami load calculations will be included here. Additional detailed calculations are presented in [9]. The ASCE 7-16 Energy Grade Line Analysis (EGLA) was used to determine the maximum flow depth (h_{max}) and velocity (u_{max}) at each site using the three transects shown in Figure 1 for the Hilo site. The resulting values are provided in Table 3.

Table 3: Tsunami Conditions at the Prototype Building Sites

Location	Max. Flow Depth h_{max} ft (m)	Max. Flow Velocity u_{max} ft/s (m/s)
Monterey, CA	21.6 (6.6)	31.5 (9.6)
Seaside, OR	31.4 (9.6)	37.9 (11.6)
Hilo, HI	55.1 (16.8)	46.9 (14.3)
Waikiki, HI	22.9 (7.0)	34.7 (10.6)

5. Overall Building Lateral Tsunami Load

According to ASCE 7-16, three load cases must be evaluated in order to determine the overall tsunami hydrodynamic loading applied to the building. Load Case 1 is a check for buoyancy and associated hydrodynamic drag. The flow depth for Load Case 1 is the smallest of: 1) ground floor story height, 2) height to the top of the first story windows, or 3) the maximum flow depth, h_{max}. For this example the top of the first floor windows is 8 ft so for LC1 h = 8 ft. Load Case 2 considers the maximum flow velocity, u_{max}, which is assumed to occur when the flow depth is (2/3)h_{max}. Load Case 3 considers the maximum inundation depth, h_{max}, with a flow velocity of (1/3)u_{max}.

The resulting overall hydrodynamic drag on the building at the Hilo location for each of the three load cases are listed in Table 4.

Table 4: Overall building hydrodynamic drag for Hilo

Load Case	Hydrodynamic Drag Kips (kN)	Flow Depth ft (m)	Distributed Load kips/ft (kN/m)
LC1	1790 (7962)	8.0 (2.44)	224 (3268)
LC2	19744 (87830)	36.75 (11.2)	537 (7835)
LC3	3291 (14640)	55.12 (16.8)	60 (875)

Because the building at the Hilo location was designed for Seismic Design Category D, ASCE 7-16 allows the design to utilize portion of the seismic overstrength to resist the tsunami loads. In order to resist the tsunami loads, the lateral force resisting system (in this case the shearwalls) must be designed for a seismic base shear of at least $E_h = V_{TSU}/(0.75\ \Omega_o)$, where V_{TSU} is the tsunami base shear and Ω_o is the seismic overstrength factor.

An ETABS computer model of the residential building was used to analyze the structural response for this E_h, representing the overall building tsunami demand (Figure 3). The resulting axial load, shear and bending moments in each of the shear walls for the Load Case 2 analysis governed the tsunami systemic loads. These systemic loads will be combined with the gravity loads on the shear walls and they will be evaluated for all inundated levels and modified as necessary.

6 Component Hydrodynamic Loading

6.1 *Exterior Columns*

Exterior columns must resist the hydrodynamic drag force assuming debris damming equivalent to a minimum of 0.7 times the column tributary width. The resulting distributed lateral load will then be applied to the column to determine the resulting shears and bending moments. The resulting distributed load is applied to the column to determine the shear force and bending moment distributions as shown in Figure 4 for Load Case 2.

6.2 *Interior Columns*

The drag force on the 20” square interior columns is based on the column width, without any effects of debris damming. The resulting load is relatively small and the existing column is adequate for resisting both the bending moments and shears induced by this load.

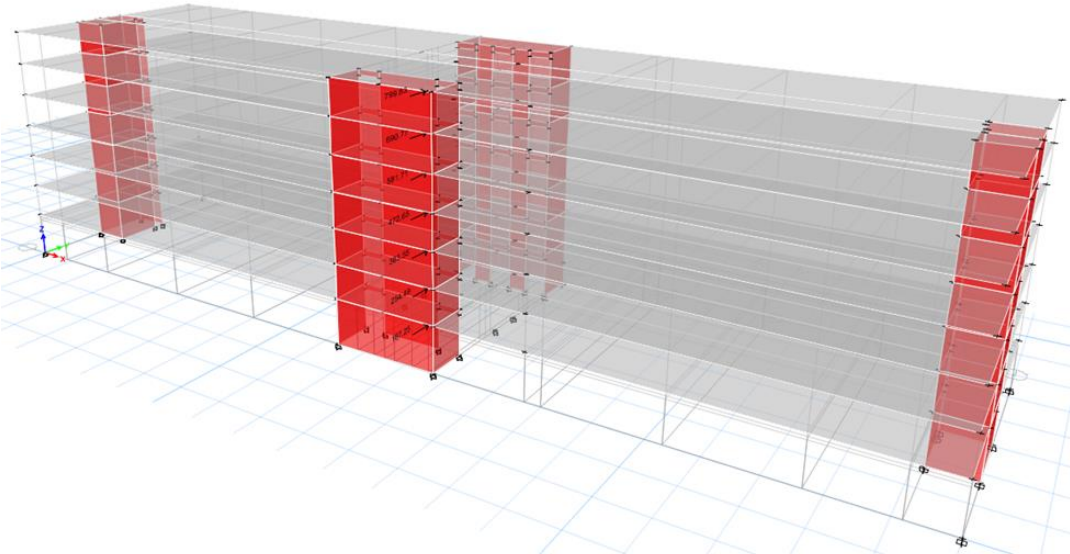

Figure 3: ETABS computer model of residential building subjected to elevated seismic loads to meet the tsunami demand at the Hilo location

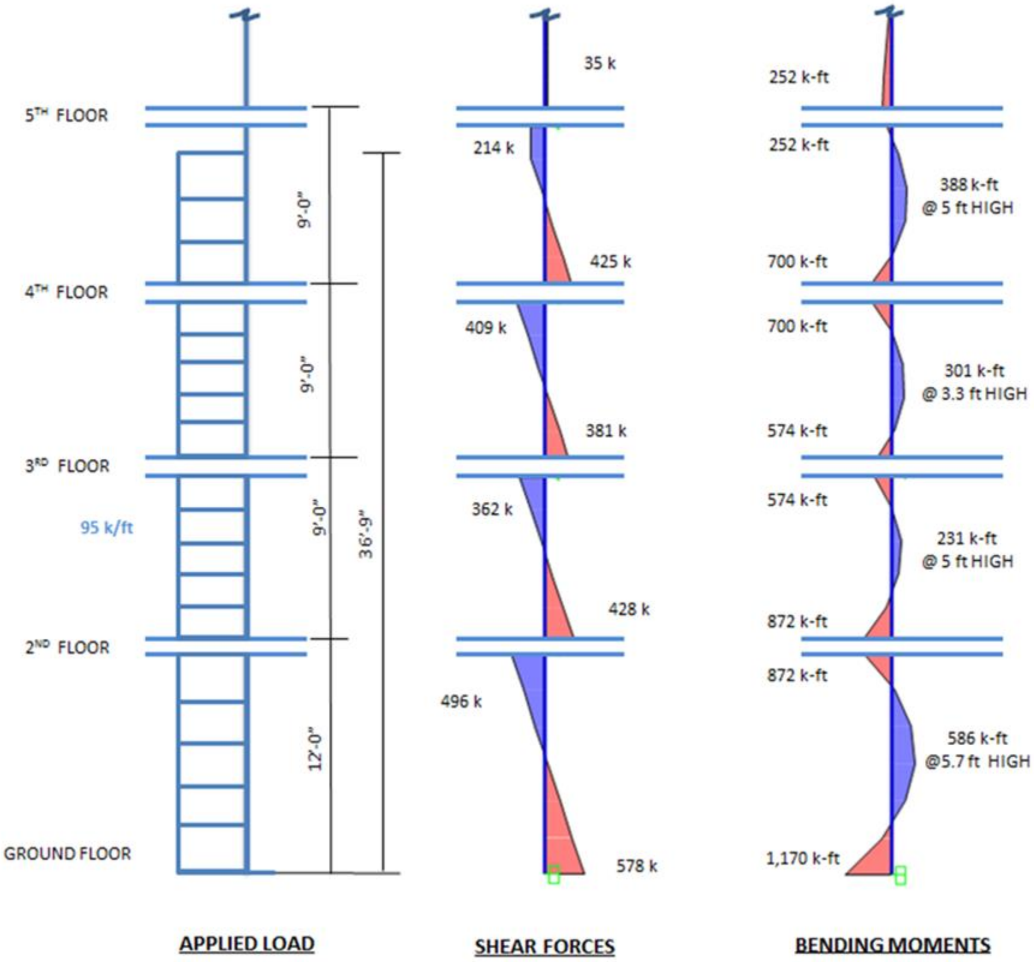

Figure 4: Load Case 2 hydrodynamic loads on individual exterior column including the effect of debris damming (1 ft = 0.305 m; 1 k = 4.45 kN; 1 k-ft = 1.36 kNm; 1 k/ft = 14.6 kNm)

6.3 *Exterior Shear Wall*

The 28 foot (8.75 m) long shear walls at the elevator and mechanical/electrical room are exposed to hydrodynamic and debris impact during incoming and outgoing flow.

For debris impact, the ASCE 7-16 assumes that a width of wall equal to half the clear story height contributes to resisting the impact load. This results in an effective wall width of 5.67 ft (1.73 m). For ease of comparison, the hydrodynamic load effects on the wall were also considered over a 5.67 ft (1.73 m) width.

7 Component Debris Impact Loads

ASCE 7-16 requires that structural elements below the maximum tsunami flow depth be able to resist impact loads due to various types of debris including cars, poles, rolling boulders, shipping containers, and larger debris depending on the site location. All four of the sites selected for this study fall outside of the large debris regions defined by ASCE 7-16 adjacent to ports, harbors and shipping container storage yards. Therefore, the governing debris impact load is due to a floating pole or log. For all locations considered in this study, the impact force was limited to the log crushing strength, given by ASCE 7-16 Section 6.11.1 as:

$$F_i = 0.5\text{x}330\, C_o\, I_{tsu} = 0.5\text{x}330\text{x}0.65\text{x}1.0 = 107.25 \text{ kips (477 kN)}$$

where: C_o = 0.65 is the debris orientation coefficient,
and I_{tsu} = 1.0 is the importance factor for TRC II buildings.

7.1 *Exterior Column*

The controlling log impact load of 107.25 kips (477 kN) must be applied at any location along all submerged structural members. For column design the load must be applied at the ends of the column, but also outside the ductile hinge region which has greater shear strength due to increased confinement reinforcement.

7.2 *Exterior Shear Wall*

For debris impact loading on the exterior shear wall, the equivalent static load of 107.25 kips resulting from a log strike acts over an effective wall width of 5.67 ft, at a point just below and above the slab at each inundated floor for maximum shear and at the mid-height of the clear wall height for maximum bending moments. Punching shear capacity for 107.25 kip impact load acting on a 1 ft diameter contact area should also be assessed.

8 Design for Tsunami Loads

8.1 *Exterior Column*

A typical first floor exterior gravity load column is shown in Figure 5. The 20”x20” (508mm x 508mm) column with concrete compressive strength f_c' = 4000 psi (27.6 MPa) is reinforced with (8) #7 (22mm) longitudinal bars and 3-leg #4 (13mm) hoops at 4” (102mm) on center for the plastic hinge regions at the top and bottom of the column. The ties reduce to 3-leg #3 (10mm) ties at 5” (127mm) on center over the rest of the column.

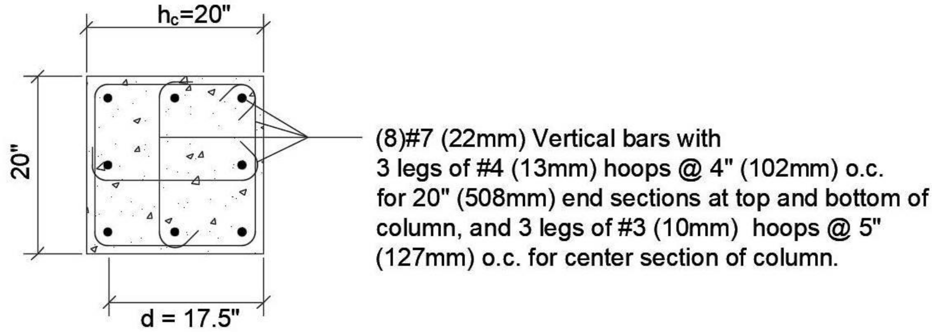

Figure 5: Original exterior gravity load column (1” = 25.4mm)

Figure 6 shows the interaction diagram for this column. The column is able to resist the bending moments due to the log debris impact load. However, the hydrodynamic bending moment far exceeds the capacity of the original column. The column must be strengthened in order to resist the combined tsunami loading. Transverse reinforcement must also be increased to prevent premature shear failures in the column. The resulting 28” (711mm) square column cross-section is shown in Figure 7.

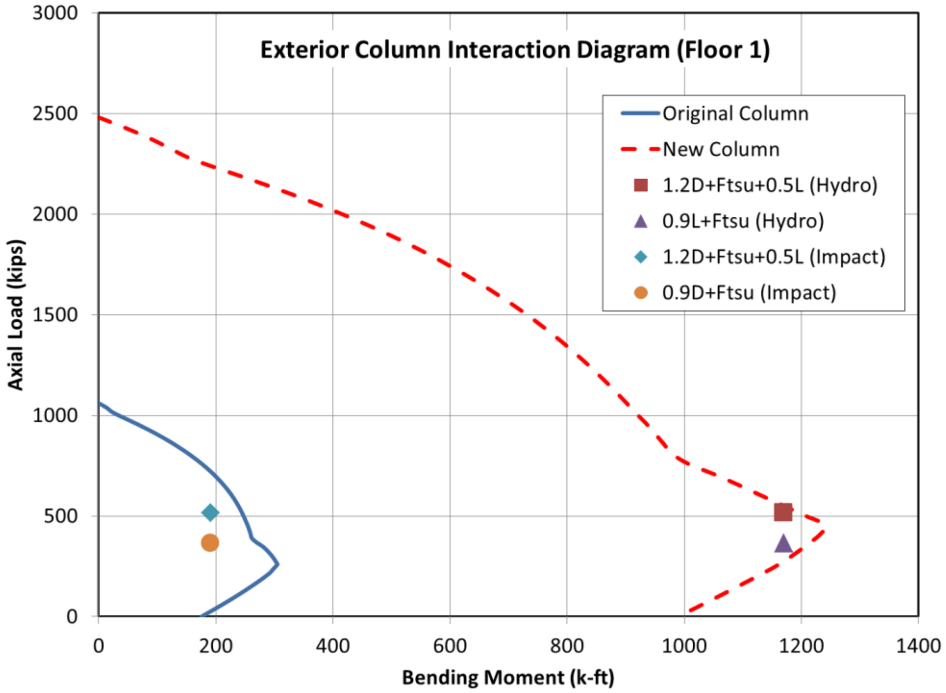

Figure 6: Interaction diagram for typical ground floor exterior column showing tsunami load combinations for debris impact and hydrodynamic drag (1 kip = 4.45 kN; 1 k-ft = 1.36 kNm).

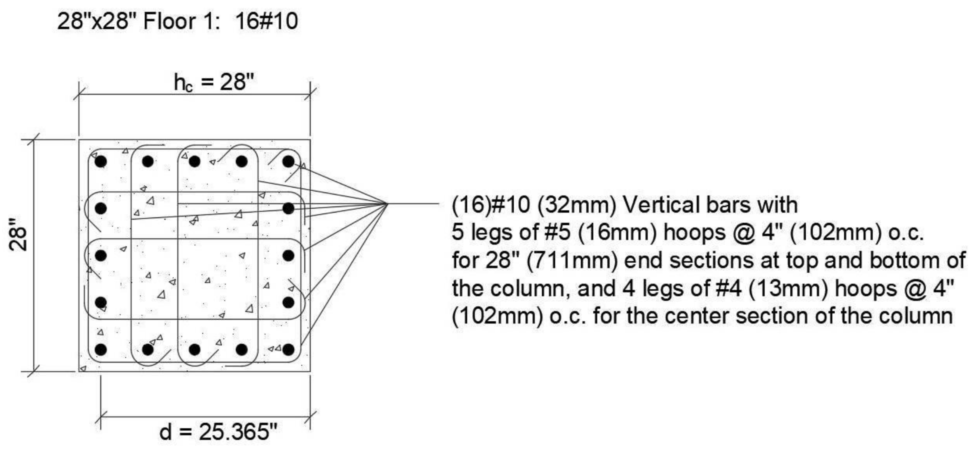

Figure 7: Exterior first floor column including tsunami design (1” = 25.4mm)

8.2 Exterior Shear Wall

The original wall reinforcement required for seismic design is shown in Figure 8. The existing wall is not adequate to resist the bending moments applied by the debris impact or the hydrodynamic load. The wall thickness was increased from 10” (254mm) to 14” (356mm) and the reinforcement in the wall was increased to satisfy all critical load combinations. The wall was also checked for both beam shear and punching shear when impacted by a water-borne log. Although adequate for punching shear, the 5.67 foot (1.73 m) wall width was not sufficient to resist the beam shear on the inundated floors. As a result, headed studs were added to the exterior shear walls on the inflow and outflow faces of the building as shown in Figure 10. The headed studs need to be provided throughout the full height of the exterior walls up to the maximum flow depth.

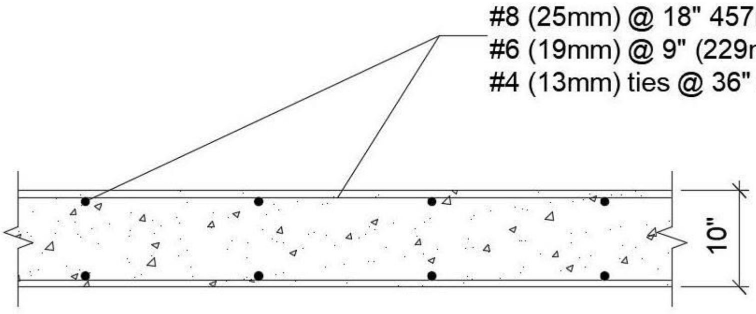

Figure 8: Original reinforcement in shear wall at ground floor (1" = 25.4mm)

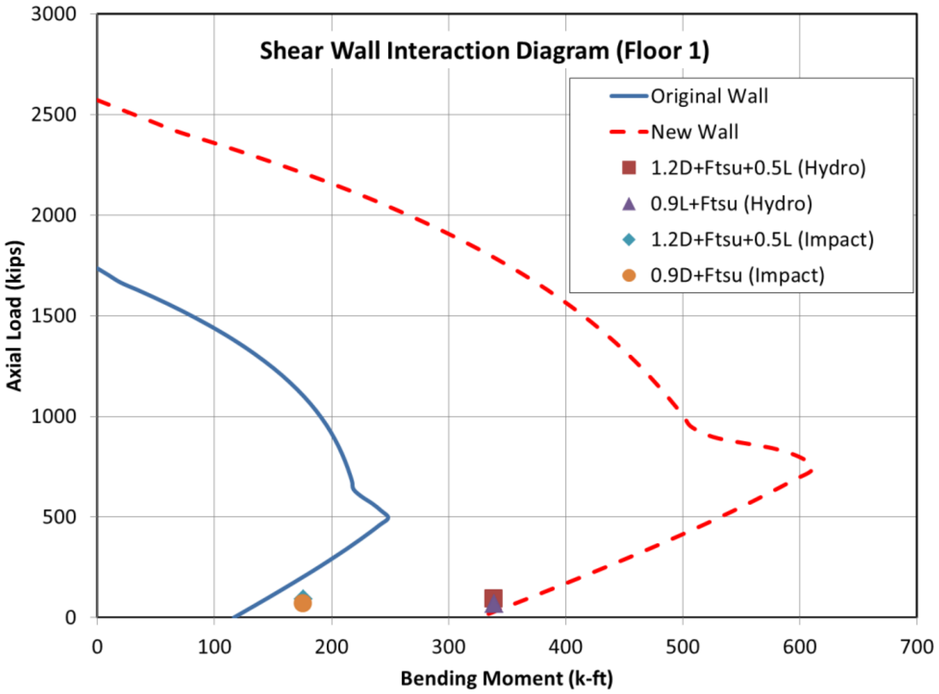

Figure 9: Interaction diagram for typical ground floor exterior wall segment showing tsunami load combinations (1 kip = 4.45 kN; 1 k-ft = 1.36 kNm).

8.3 Elevator Shear Wall

The overall shear wall cross-sections must be designed for the bending moments and shear determined by the ETABS analysis for the enhanced seismic loads required to resist the tsunami hydrodynamic loads on the entire building. The original elevator wall designed for seismic loads is shown in Figure 10. This wall is adequate for all seismic load combinations as shown in the interaction diagram in Figure 11. However, the enhanced loads required for tsunami design require strengthening of the wall (Figure 11) resulting in the new wall reinforcement shown in Figure 12.

9 Cost Implications of Adding Tsunami Design

As demonstrated in the Hilo example shown above, the exterior gravity load columns require increases in cross-section size and in reinforcing steel at the first and second floors in order to resist the tsunami loads. In addition, all of the shear walls need to be strengthened to resist the hydrodynamic drag on the overall building. In addition, the shear wall panels located on the exterior of the building are exposed to debris impact which requires the addition of headed studs to improve the out of plane shear capacity.

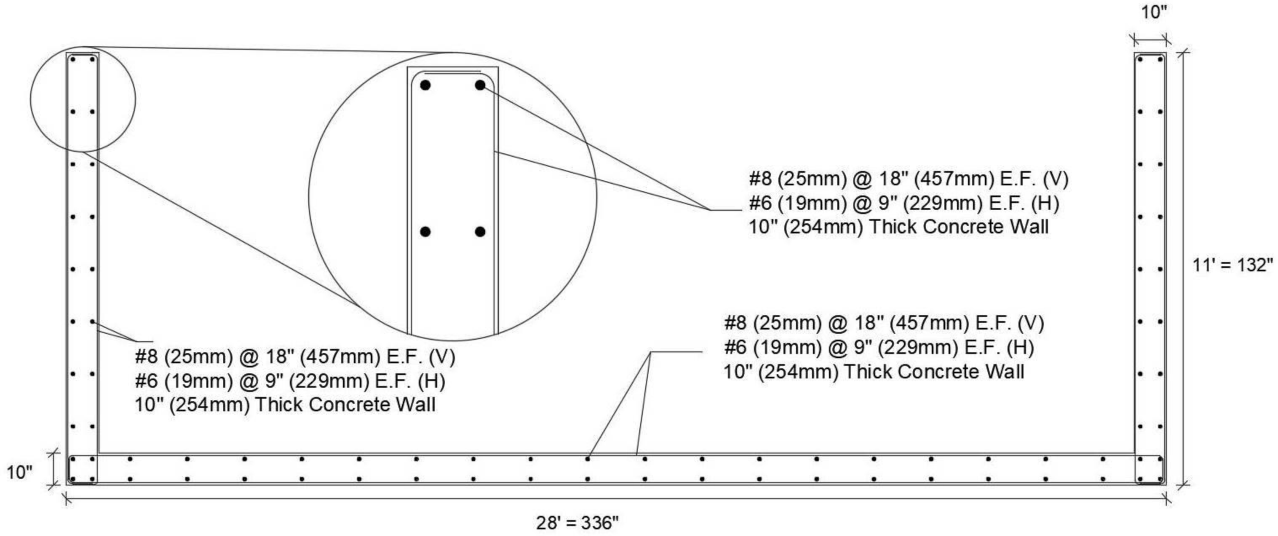

Figure 10: Original elevator shear wall cross-section at the ground floor level (1" = 25.4mm)

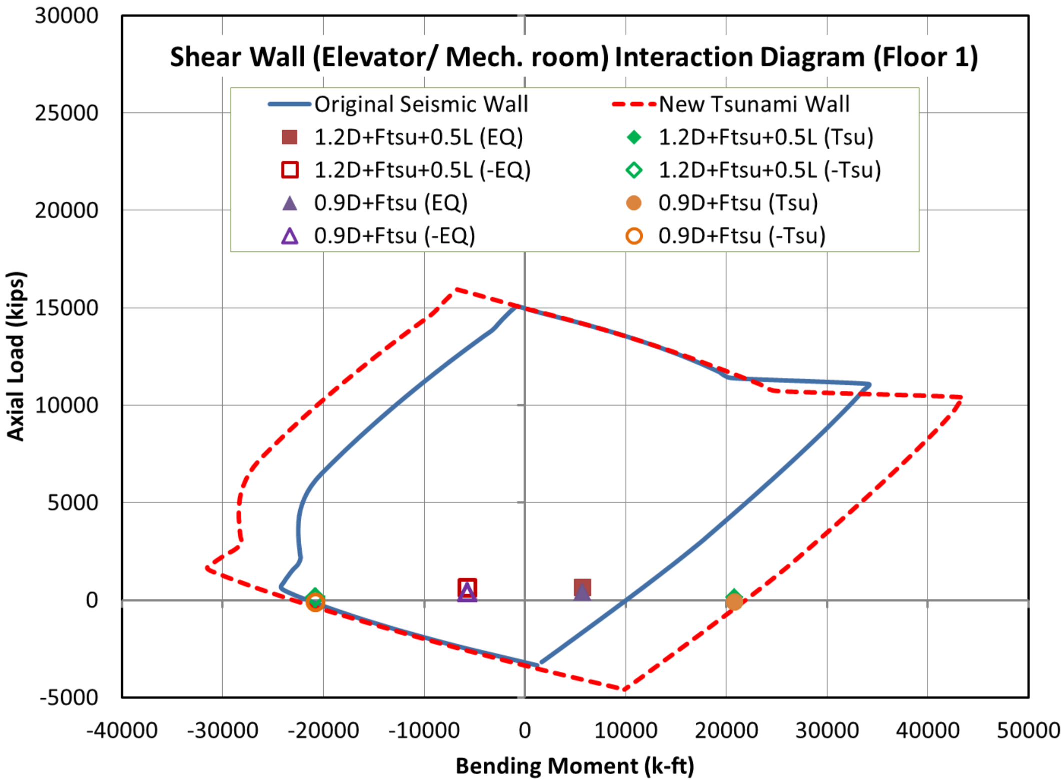

Figure 11: Interaction diagram for ground floor elevator shear wall showing seismic and tsunami load combinations (1 kip = 4.45 kN; 1 k-ft = 1.36 kNm).

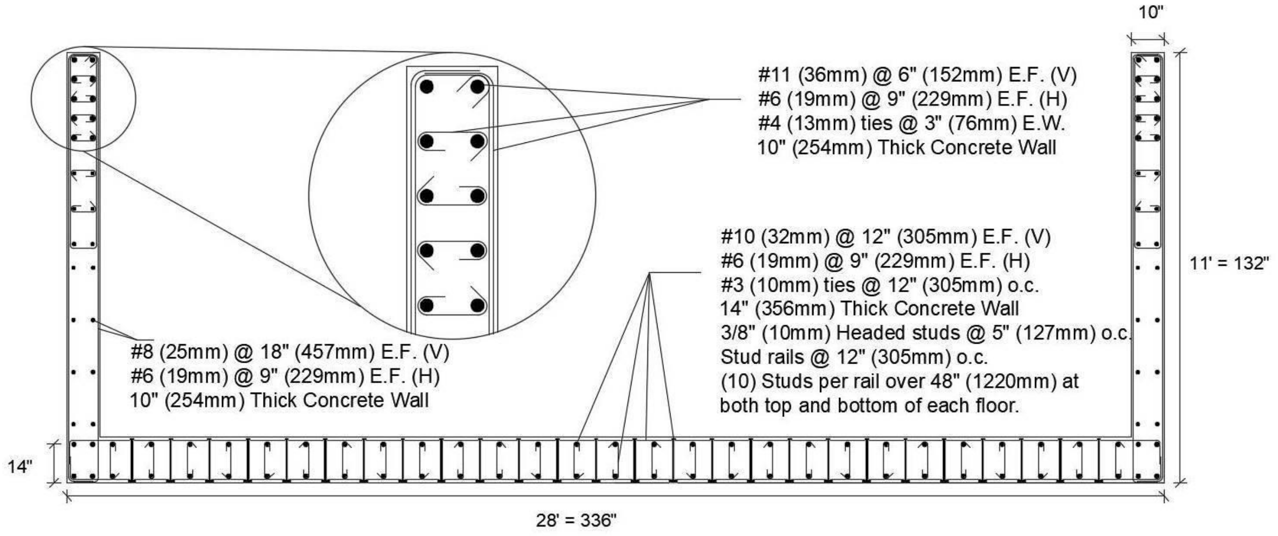

Figure 12: New Elevator shear wall cross-section at the ground floor level

Similar observations were noted at the other three locations. At the Seaside OR, location the tsunami maximum inundation depth is 31.4 feet (9.6 m), resulting in increases in both column and shear wall size and reinforcement requirements for the first two floors of the building. Because of the lower tsunami loading at Monterey CA, there was very little increase in the column reinforcement. At the Waikiki HI, location, the flow conditions are only slightly greater than those at the Monterey CA, location. However, the lower seismic design requirements (SDC C) result in less robust shear walls requiring an increase in both wall thickness and reinforcement, while the gravity load exterior columns require strengthening of the first two floors similar to the Seaside building.

Based on a prior study by Nitta and Robertson [11] the overall construction cost of the prototypical buildings are $23,305,700 for the special shear wall building at the Seaside, Monterey and Hilo locations and $22,021,300 for the ordinary shear wall building at the Waikiki location. This includes the structural frame and all non-structural components of the building. Note that this study ignores the variation in construction costs between these four locations so as to provide an unbiased comparison of the premium paid for incorporating tsunami design.

The resulting increases in concrete materials (incl. formwork, labor, etc.) and reinforcing steel (incl. fabrication and installation) for the prototypical building at each location are shown in Table 5.

Table 5: Cost Increases due to Tsunami Design

Location	Concrete	Reinforcement	Total	Incr. (%)
Monterey,CA	$ 19,035	$ 36,283	$ 55,318	0.24
Seaside, OR	$ 52,238	$ 57,864	$110,102	0.47
Hilo, HI	$178,127	$117,335	$295,462	1.26
Waikiki, HI	$ 52,460	$ 31,717	$ 84,177	0.38

10 Conclusions

Based on the prototypical 7-story reinforced concrete residential buildings considered in this study, the following conclusions were drawn:

- The increase in building cost to include tsunami design is less than 1% for all locations except Hilo, Hawaii, where the flow depth and velocity corresponding to the design tsunami are extremely high. Other locations with bathymetry and topography that tend to focus the tsunami energy may also experience greater than 1% cost increases, but it is unlikely ever to exceed 2% of the overall building construction cost.

- Locations along the California coast south of Mendocino, (represented by the Monterey location) are less directly exposed to the Cascadia Subduction zone. This results in very small, if any, premium for including tsunami design in special shear wall concrete buildings over 7-stories high.
- Locations exposed to the Cascadia Subduction Zone (represented by the Seaside location) would expect cost increases up to 0.5% of the overall building cost.
- Locations with ordinary shear walls (represented by the Waikiki location) will require additional strengthening because of the original lack of high-seismic design.
- The cost premium for incorporating tsunami design will decrease for buildings that are taller than 7 stories. The wind and seismic design of taller buildings will require more robust members at the lower floors, which in turn will require less strengthening to resist the tsunami loads.
- The cost premium will also reduce for buildings located further inland since the tsunami flow parameters decrease with distance from the shore-line. The sites selected for this study were purposely located close to the shoreline to determine the likely maximum tsunami design cost premium for each location.
- The cost premium for incorporating tsunami design will increase for similar buildings that are shorter than 7 stories because of the lower wind and seismic load design for these buildings.

8. Copyrights

9. References

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