PERFORMANCE OF LIGHT-FRAME WOODEN STRUCTURE (LFWS) SUBJECTED TO COMBINED WIND AND FLOOD HAZARDS

by

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Abstract

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Light Frame Wooden Structures (LFWS), which are more prone to wind damage by virtue of their light weight, sustain severe damage when they are subjected to simultaneous flood and wind. A difficulty in Multi-hazard design for wind and flood is that the load and resistance factor make use of different design philosophies developed by different sub-disciplines. After Multi-hazard events, in most cases, it is complicated to determine the sole damage done by flood or wind alone. The load path of Wooden structure is already ambiguous, the addition of flood characteristics makes the analysis and design process more complicated. It has therefore become a necessity to understand the performance of LWFS subjected to both individual and combined action of wind and flood hazards. Flooding events are well evident even on areas beyond Special Flood Hazard Areas (SFHA) but still there are no requirements for buildings to be elevated and designed for flood loadings in those areas.

This thesis presents comprehensive investigation of the advanced analysis of Light- Frame Wooden structure and concisely developed flood and wind characteristics. Two types of buildings, Slab-on-ground type and elevated type located in 500-year flood plain and 100- year flood plain, respectively is modeled to understand their performance against combined action of flood and wind loadings. Detailed analysis is done to

understand local and global failure mode of each type of building for different flooding conditions, assuming similar wind loading conditions. From the overall performance of Slab-on-grade type building located in high wind region and 500-year floodplain, it was concluded that a non-engineered building constructed complying only the minimum requirement set by building authorities is liable to fail locally at Wall to foundation shear connection.

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Chapter 1

Introduction

1.1 Problem statement

It is a common phenomenon to observe natural disaster involving multiple hazards. Similarly, the effect of wind disaster may not be limited to wind damage, as concurrent heavy rains and flooding often cause additional havoc. There have been plenty of such events when tropical cyclones have worsened coastal erosion and flooding. For instance, the 1999 Hurricane Floyd, Florida to Maine, destroyed more than 7000 and damaged 56,000 homes. Sustained tropical winds and gusts with a maximum speed of 104 mph were recorded as far north as New York. Storm surge and torrential rains caused extensive flood damage in North Carolina where up to 20 inches of rain fell (NOAA, 2017). Hurricane Katrina in 2005 is one of the costliest Hurricane ever, with an estimated damage of $108 billion and death toll of 1,833 (National Hurricane Center, 2011). Despite, severe consequences of flood on human lives and built structures, only 70% of the US counties have adopted building codes to enforce flood-resistant design (FEMA, 2013).

Timber residential buildings, which are more prone to wind damage by virtue of their light weight, sustain severe damage when they are subjected to simultaneous flood and wind. However, limited information is available to take into account the combined effect during design. According to IBC 2015 and ASCE 7 (2010), for any building types in non-coastal A zone and coastal A and V zone, the required Load Resistance Factor Design (LRFD) load combination for wind and flood load is obtained by replacing Wind loads by Eq. (1) and EQ. (2), respectively. Similarly, Allowable Stress Design (ASD) load combination in non-coastal A zone and coastal A and V zone is obtained by adding Eq. (3) and Eq. (4), respectively.

$$0.5W + 1.0F_a \tag{1}$$

$$1.0W + 2.0F_a \tag{2}$$

$$0.75F_a \tag{3}$$

$$1.5F_a \tag{4}$$

Where: W = wind load

F_a = flood load

However, there is no such provision for flood in zones B, C, and X (areas of moderate or minimal hazard from the principal source of flood in the area, often referred as 500-year flood plain), even though no area in the nation is exempt from damage due to flash flood (FEMA, 1999) and study showed that approximately 30% of all flood claims came from buildings outside the 100- year flood plain (The nurture nature center, 2012).

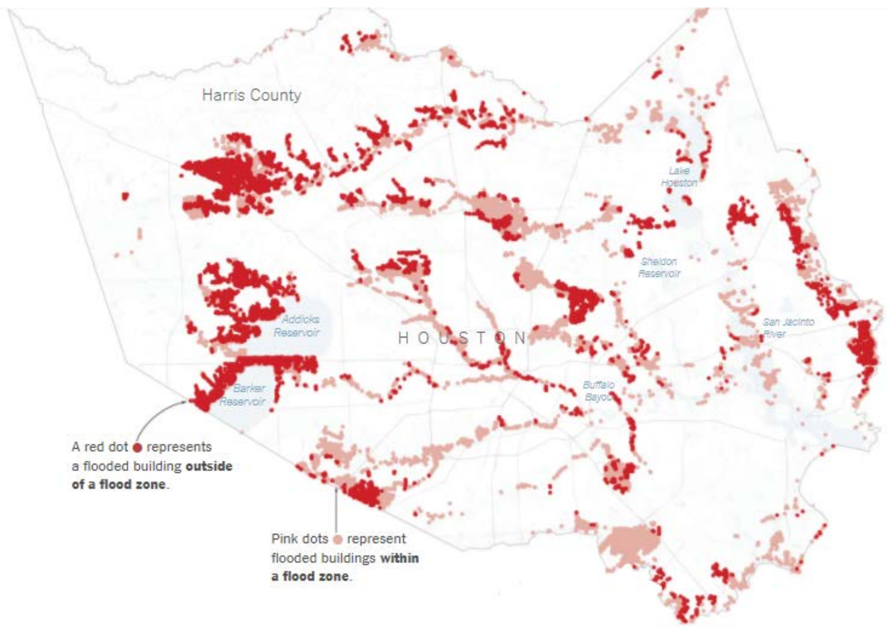


Figure 1.1 Buildings flooded within and outside flood zone in Harris County, Texas (August 2017) (Source: FEMA, 2017)

In Harris county (Figure 1.1), which just received new flood map on January 2017, 40% of the total buildings flooded during Hurricane Harvey were located outside 100-year flood plain (Fessenden et al, 2017). Also, an evaluation of repetitive flood losses from 1978 to 2008 in Harris County, Texas determined over 47% of total losses were located outside the 100-year flood plain (Highfield et al., 2013). Analysis of national sample of 450 jurisdictions determined 25% of flood insurance claims in between 1999 to 2009 were claimed against buildings located outside 100-year flood plain (Brody and Highfield, 2011).

To identify and understand the flood risk and vulnerability of communities under the National Flood Insurance Program (NFIP), FEMA (2011) published Flood Insurance Studies (FIS) and Flood Insurance Rate Maps (FIRMs), which provide information for participating communities to manage and enforce requirements within their designated flood zones. NFIP requires mandatory Flood Insurance and mandatory elevating of lowest floor above Design Flood Elevation (DFE) for all the buildings constructed for residential purpose in Special Flood Hazard Area (SFHA) (FEMA, 2013). However, there are no such requirements for buildings in zones B, C and X since they are outside the Special Flood Hazard Area (SFHA).

It is also common for debris travelling downstream during flood or combined flood and wind event to exert impact load on structure it encounters. This impact load is destructive in nature since the force associated with it may be an order of magnitude higher than the hydrostatic and hydrodynamic forces (Rogers, 2017). However, currently available design codes do not have reasonable provisions for impact load from debris, ice, and any objects transported by flood water striking against the building located in 500-year flood plain. Also, ASCE 7 (2010) does not describe effective measure to account effect of hydrodynamics loads on structure in case the velocity of flood exceeds 10 ft/sec (3.05 m/s) but study of flood events in Korea suggest average velocity of 3.85 m/s (Kang & Kim, 2016), leaving practicing Structural Engineers with option to not incorporate effects of hydrodynamics and impact loading during flood event.

Buildings in SFHA are subject to a greater hazard than those in other types of floodplains. Thus, they are required to be elevated either above the Base Flood Elevation (BFE) or the Design Flood Elevation (DFE). Elevating the building structure can benefit by avoiding seepage into the building and prevent loss of properties and by reducing the hydrodynamic and hydrostatic force due to reduced surface area but during simultaneous flood and wind event, elevated structures with open bottom, either it be on SFHA or Non-SFHA, will be subjected to buoyancy force due to flood which is combined with an uplift force from wind. The negative reactions thus occurring in building's sub structure can result in overturning, sliding and even uplifting of entire superstructure. However, no codal provision exists to take into account this combined effect, leaving practicing engineer without option.

1.2 Scope and Objectives

The main contribution of this thesis is to bridge the knowledge gap as mentioned in the Problem Statement above. The author and project supervisors determined the following project scope. The scope and objectives of this thesis are summarized below:

a) Procurement of all possible Flood and wind loading parameters and study their characteristics.

b) Study of various past flood and wind disasters and their combined effect on timber low rise building.

c) Study of performance of Two-story building designed as per prevalent norms in 500- year flood plain subjected to different intensities of flood and wind against global failure modes.

d) Study of performance of a Single and Two-story building designed as per prevalent norms in 100-year flood plain subjected to different intensities of floods and winds against global failure mode.

e) Study of performance of Two-story building designed as per prevalent norms in 500- year flood plain subjected to different intensities of flood and wind against local failure of Floor to foundation shear connection.

1.3 Outline of Thesis

There is a need for more reliable research that incorporates the effect of multi-hazard in any kind of structure. There are ample of extensive research available to determine the effects of individual natural disasters on buildings, while the study of combined impact due to multiple hazards on the building is in novice stage. Perhaps it is because of the current design procedures in the United States which uses the envelope of individual hazard demands on a structure to ensure safety against multiple hazards (Crosti et al, 2011). A difficulty in multiple hazard design for wind and flood is that the load and resistance factor make use of different design philosophies developed by different sub-disciplines. The type and level of damage caused by various natural disaster might differ, but the socio-economic impacts are equally significant. Flood damage and wind damage, in any structure, are sometimes easily distinguishable but most of the time it's complicated to determine the sole damage done by flood only.

Research can be divided into three phases: defining hazards, model analysis and assessment of results. Hazards definition includes lessons learnt from past experiences, requirement of multi-hazard design, the characteristics of all forms of possible loads and their specification. It also includes study of building bylaws near Coastal areas and other water bodies as required by a National Flood Insurance Policy (NFIP) complying communities and other building design codes. Design basis report preparation comprises of all sort of data accumulation required for analysis, design and detailing of building. Model analysis involves preparation of 3D model of a given building based on design basis report and analyzing it for various possible load cases. Final phase is to assess the results, observe the behavior of structure at different load conditions and conclude the research.

Results from thesis research can be used by practicing engineers and in academics to better understand the combined action of flood and wind on a Timber residential low-rise building. With increasing construction activities and other natural causes, the flood flow can vary in a location, which can result in higher velocity, increment in depth and more or larger debris in the flow. Using the results from thesis research, recommendation can be made on the retrofit of wind designed building for flood resistance. Structural components like walls, columns, piers, piles and beams can be stiffened based on the differences between wind and flood design. Just as importantly, it can also be used by owners and architects to determine requirement for elevating the slab and to select proper site location as per the buildings requirement.

Chapter 2

Literature review

2.1 Introduction

This chapter is a literary overview and introduction to previous research, current design practice and summarize past multi hazard events and lessons learnt from them. It first identifies the growing and never-ending timber building industry, its history, properties and direct and indirect impact of flood and wind on low rise timber residential light frame structure. Previous building collapses and damages either due to individual or combined flooding and wind action are identified and analyzed in order to qualify the various modes of failure. The change in mechanical properties such as reduction of strength and stiffness of timber connection is thoroughly studied. In essence this chapter presents previous literary evidence and recommendations for modeling, analyzing and designing low rise timber residential light frame structure incorporating both direct and indirect impact of wind and flood.

2.2 Timber buildings

2.2.1 Introduction

Wood is a renewable and sustainable resource that costs less to produce than steel and concrete, which have been the primary building materials of last 125 years. When wood is processed for use as a structural material, it is often referred to as timber. In North America and Europe there is a strong demand of timber to construct low rise to multi-story residential buildings. The main reasons for this demand are sustainability of wood, ease in fabrication, cost efficiency due to quality-controlled pre-fabrication, durability, easy transportation, fast on-site erection of the building and elegant looks. It is a method of construction which can be more labor intensive than steel or concrete construction, but which offers a finished appearance demonstrating wood construction and workmanship in an artistic form. Wood is also environmentally responsible because it stores carbon, whereas the production of other man-made construction materials like cement, PVC et cetera requires massive amounts of fossil fuels and harmful emissions. Rising concern of climate change and the carbon-dioxide emissions during the construction process encourage the use of timber.

Timber buildings, which currently predominate low rise residential building industry can also take over mid-rise construction in near future owing to their environment friendly, aesthetic and economic construction. The 9-Story Stadthaus apartment building in London was completed in 49 weeks (as opposed to 72 estimated for a similar concrete building) and stores 185,000 kg of carbon (opposed to emissions of 125,000 kg with the concrete design), while being comparable in overall cost. The timber structure was just erected in 27 days by four men each working 3 days in a week, while the entire project took 49 weeks. As designer and contractors get more experienced in wood construction, the building cost and project duration will continue to be more competitive (Thompson, 2009).

2.2.2 History of Timber buildings

Not only today, wood shelters were also among the primary materials in the early ages used by mankind to protect themselves from natural hazards. Egyptians and Romans were the first kind to have used timber frames for multiple purposes back in 500 to 1000 B.C. Europe is full of timber- framed structures dating back hundreds of years, including manor and castles, homes and inns, whose architecture and construction technique evolved over the centuries. Similarly, In Asia there are temples and monasteries that were built as early as on Sixth century. In America, timber structures increased in considerable number during the colonization of Europeans.

The development in mass production of steel nails in 1830s led to a revolutionary change in the wood construction industry. Prior to 1830s, mortise and tenon joints were used as connection in heavy timber structures (Wolfe and McCarthy, 1989), but the invention of nails led to the use of smaller, standardized lumber sections which are still in use these days. Until 1940s wood construction practice changed a little, including beginning of wood-frame roof assemblies with wood planking (typically 1-inch nominal thickness) fastened with two or three nails per framing member (Wolfe and McCarthy, 1989). To overcome the demand for housing and construction of war-related infrastructure, during World War II, United States Military propelled more efficient and faster wood construction methods. Plywood sheathing was developed to replace wooden plank in roof assembly. The demand for plywood sheathing escalated because of its lighter weight and ability to quickly install due to fewer nails and larger coverage. In mid-1950s, development of metal plated connection trusses replaced the lumber rafters and board sheathing construction.

In summary, major changes in conventional light-frame wood construction practice occurred approximately 75 years ago with the introduction of nails allowing light frame construction, development of metal plates connecting trusses which allowed larger span construction and manufacture of plywood which reduced the weight of roof assembly and decreased the construction time. From 1950 to 1990, about 69 million homes were constructed which was 65% of the building stock in the US (US Census Bureau 2003). Much of the existing building stock in hurricane-prone region and flood zones was designed and constructed to codes and standards that required far less than current codes to mitigate flood and wind damage or constructed to no codes at all.

2.2.3 Construction of Residential light frame wood structure

In low-rise residential buildings, light-frame wood structures, such as a stud wall covered in sheathing with nails either one sided or both sided transfer all the lateral and vertical loads imposed. Walls are “stick-built” on site and fastened together using nails and constructed of dimensional lumber-nominal 2x4 and 2x6 member are typical. The studs are typically spaced at 16 to 24 inches on center (HUD, 2000). Nails are used for shear and moment connection between plywood sheathing and dimensional lumber. Sheathing attached to walls and roof trusses or rafters, creates diaphragm action to resist in-plane loads. Most residential roof structures today are framed with trusses composed of several lumber pieces connected using metal plate connector.

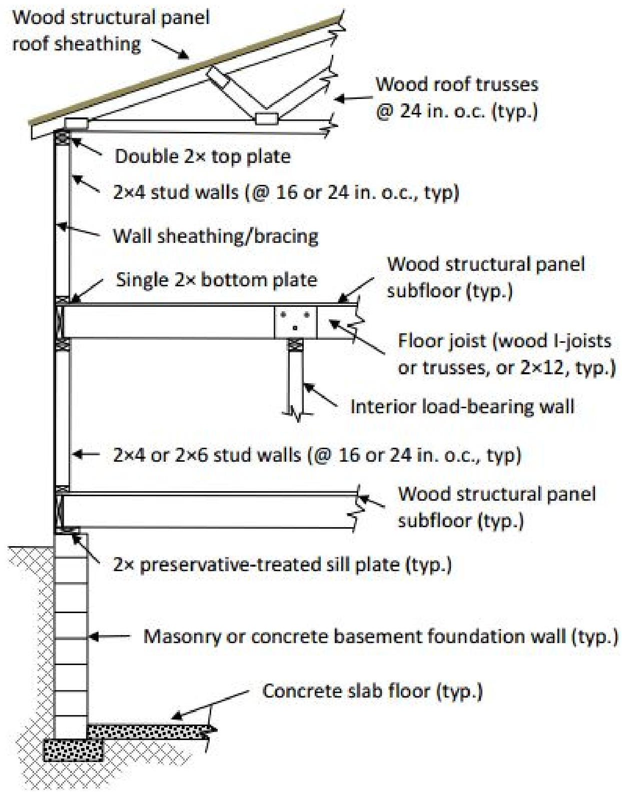


Figure 2.1 Typical modern light frame construction (Source: HUD 2000)

Figure 2-1 provides visual representation of the construction of LFWS, often called “Stick-built” or platform construction. Wood structural panel attached to floor joists or floor truss also creates diaphragm action and distributes the lateral wind and Seismic loads to shear walls or columns. Generally, the diaphragm shall be considered flexible when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story (UBC 1988). However, ASCE 7-10 Section 12.3 allow light-frame diaphragms to be idealized as flexible.

Madsen defines the requirement of an adequate connection as “ability to transfer axial forces, moments, and shear forces from one structural member to another, with acceptable deflections and rotations, and adequate safety at a reasonable cost” (Madsen, 2000). Ideally, connections in the timber would be able to withstand all the forces present in the member, if properly designed as per standard. Building construction in coastal areas and inland construction are substantially different in terms of connections. Construction in noncoastal, non-seismic areas with no high wind hazards must normally support only vertical loads and modest wind forces. In coastal areas, large forces are applied due to wind, hydrodynamic and breaking wave action of flood and debris impact in lateral and upward direction, thus connections should be able to provide considerable lateral and uplift resistance. Connection hardware must be corrosion resistant, but outer galvanized layer of galvanized connector is subject to knocking off when hammer strikes the connector and nail during installation so extra attention is required. To avoid such problem, corrosion resistant connectors that do not depend on a galvanized coating can be used such as stainless steel.

Figure 2-2 depicts the vertical load paths in low rise- light frame wooded structure to resist wind uplift forces on the roof. The wind load acting upwards on the roof sheathing is resisted by nailed connections between the sheathing and truss. The trusses are connected to walls, which transfers uplift force coming from sheathing to wall system. Upper story walls are fastened to floor which is again connected to wall beneath. Similarly, the load is transferred to ground floor and ultimately to ground via piles, thus completing the Structural load path. The connection between the structural elements in load path play vital role thus must be designed to adequately transfer the applied loads from roof to ground. Unlike in moment resisting frame structure, there are multiple load paths available, thus determining the most critical path is very important. The combined action of flood and wind load together makes the process more complicated because these are two different loadings with varying properties derived from different disciplines.

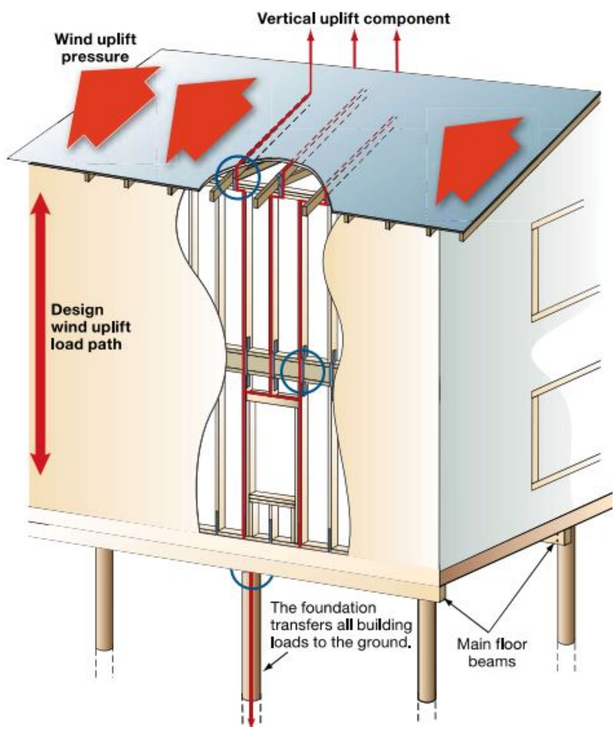


Figure 2.2 Vertical Load paths in low rise, LFWS (FEMA, 2010)

2.2.4 Common type of foundation for non-elevated building- Slab-on-Grade:

Slab-on-grade or floating slab foundations is a common engineering practice whereby the concrete slab that is to serve as the foundation for the building is formed by casting concrete directly on the ground surface, leaving no space in between ground and the structure. It is very common in places where ground freezing, and thawing effect is of very less concern and there is no need for heat ducting underneath the floor. These are also commonly used in residential foundations or any lightweight structure on shrink-swell soils or expansive soils. This type of foundation is suitable where basements are not required to be elevated and when expansive soil conditions extend depths that make pier construction cost prohibitive (Lytton, 2004).

The commonly adopted depth of slab and grade beam for two story Light Frame Wooden Structure is 4 inches and 24 inches respectively. However, for single story 18 inches of grade beam is also common depending on the soil conditions. The bottom of exterior beam is required to be at least below 12 inches from the ground level (Home of Texas, 1995). Internal beam is also required for all the internal framing walls. The requirements for external and internal beams are shown in Figure 2.3 and Figure 2.4, which was published in Technical guide by Home of Texas (1995).

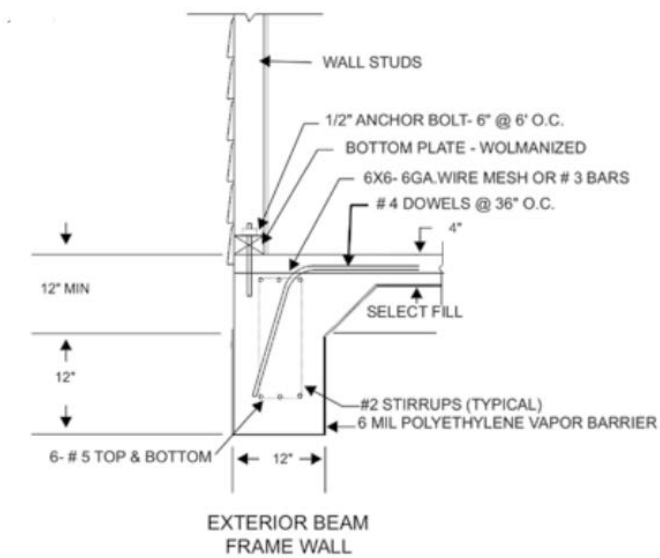


Figure 2.3 Detail of exterior beam of Slab-on-grade foundation (Home of Texas, 1995)

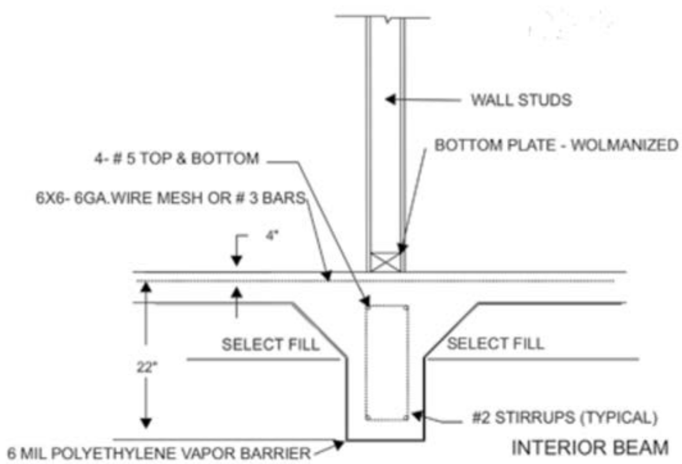


Figure 2.4 Detail of interior beam of Slab-on-Grade foundation (Home of Texas, 1995)

2.2.5 Common type of foundation for elevated building- Pile foundation:

In V-Zones, the National Flood Insurance Program (NFIP) restricts foundation to pile or column foundations and does not permit the use of fill to elevate the building above base flood elevation (BFE) and construction of slab foundation, crawlspace foundations, basements and solid foundation walls. Provided that the fill is prevented against erosion, NFIP allows buildings to be elevated and supported by fill in A-zones. When the buildings are to be elevated above Base Flood Elevation (BFE), pile foundation is commonly used. The minimum lowest floor elevation and the foundation type design for new construction are determined by the Base Flood Elevation (BFE) (FEMA P-499, 2010).

Contractor doing construction in coastal areas typically select preservative treated wood piles for pile foundations (FEMA P-499, 2010). Timber piles properly treated with preservatives such as creosote can be very durable with a service life of over 65 years (AWPI, 2002). Wooden piles can be square or round in cross section. Concrete piles and steel piles are less common for residential buildings, however, depending on the pile capacity and elevation requirement concrete piles can be appropriate choice. The ability of wooden piles to be easily cut and adjusted in field and lower cost makes it a popular choice for residential construction without significant elevation requirement (FEMA, 2010). Timber piles are a mainstay of foundation designer, design loads as high as 75 tons have been specified and ultimate loads as high as 235 tons have been carried by timber piles (Timber piling council, 2016). Timber piles are being used in all kind of structures apart from residential buildings, including marine structures, manufacturing plants, processing facilities, commercial buildings, and highway bridges. Foundation of new facilities at JFK Airport in New York, Dulles International Airport in Northern Virginia and infrastructures like residential, commercial buildings, the superdome as well as paved highways in New Orleans are the major examples.

Table 1 Commonly available timber piles tip diameter and length

Foundation piling material	Commonly available tip diameter (inch)	Commonly available length respective to tip diameter (ft)
Southern pine	12,13,14,15	60, 75,70,65
Southern yellow pine	6,7,8,9,10	45,75,70,55,40
Douglas Fir	11,12,13,14,15	45,65,75,100,100

Source: Timber Piling Council (2016)

The total length of pile foundation is calculated based on penetration requirements, erosion and scour potential, Design Flood Elevation (DFE) and uplift force. The minimum diameter of tip of round pile shall not be lesser than 8 inches and similarly the square piles should have a minimum size of 8 inches by 8 inches (FEMA, 2010). However, more than 50 percent of pile cross-section can't be cut for placement of girder which is also the basis for selection of pile dimensions. Pile bracings, as shown in Figure 2.5, are generally used to only address serviceability issue and not strength issues. In other words, Pile bracing should be for comfort of the occupants, but not for stability of the home (FEMA, 2010).

The connection between wood floor joists and girders is usually bearing connection for gravity forces with a twist strap tie for uplift forces. Floor joists of first floor require solid blocking at the end and at spacing of 8-ft on center of the joist where it is supported to prevent from web crushing and buckling unless substantial sheathing (at least 1/2 – inch thick) has been nailed well to the bottom of these joists. As an alternative, bottom flanges of the joists are braced with a small metal strip to prevent the flange from twisting, however, because metal strip is subject to corrosion, solid wood blocking is more effective in coastal or corrosive environments.

Elevated buildings are subjected to large uplift forces from high winds and flood so the connection between wood floor joists and the supporting girders is very critical. The undersides of the elevated structures where the connectors are located is vulnerable to salt sprays; the exposed surfaces are not washed by rain and remain damp longer because of their location. FEMA (2011), recommends application of sheathing to the underside of the bottom floor framing when a building is elevated on an open foundation. The sheathing provides insulation between joist or trusses from wave spray, and thus minimize corrosion of framing connectors and fasteners. It also prevents the floor framing being knocked out of alignment by flood-borne debris passing under the building.

Figure 2.5 Successful pile foundation following Hurricane Katrina (Source: FEMA, 549)

2.2.6 Selection of Panel thickness, stud spacing, nail size and nail spacing

The minimum nominal panel thickness of Wood Structural panel wall sheathing is generally only designed for in-plane shear force; however, it should also comply for moment and shear requirements coming from the out-of-plane wind force. Maximum allowable spacing of Stud, minimum nominal panel thickness, minimum nail size, minimum wood structural panel span rating and panel nail spacing on edges and field are the function of maximum nominal design wind speed, $V_{asd,}$ which is provided in tabular form in Table 2304.6.1 (IBC, 2015).

$$V_{asd} = V_{ult}\,(0.6)^{0.5} \tag{5}$$

Where: V_{asd}= Nominal design wind speed

V_{ult} = Ultimate design wind speeds determined from Figures 1609.3 (1, 2 or 3)

2.3 Flood

2.3.1 Introduction

According to FEMA (2017), flood is a general and temporary condition of partial or complete inundation of normally dry land areas from: 1) the overflow of inland or tidal waters; 2) the unusual and rapid accumulation or runoff of surface waters from any source; 3) mudslide (i.e. mudflows) which are proximately caused by flooding and are akin to a river of liquid and flowing mud on the surfaces of normally dry land areas, as when earth is carried by a current of water and deposited along the path of the current. Most floods fall into three major categories: *riverine*, *shallow* and *coastal flooding*. Alluvial fan flooding is another type of flooding more common in the mountainous western states.

When a channel receives too much water, the excess flows over its banks and into the adjacent floodplain. Flooding thus occurring along a channel is called *Riverine flooding* (FEMA, 1998), which is also known as fluvial flood. Heavy snow melt, ice jams and drainage obstruction due to landslide, ice, debris, or beaver dam can also cause this kind of flood. The dynamics of riverine flooding vary with terrain. In relatively plain areas, land may stay covered with shallow and slow-moving floodwater for several days, while in hilly and mountainous areas, flooding can occur in minutes after a heavy rain. Instantaneously occurring, large depth and high velocities flash floods make this type of floods more dangerous (FEMA,1998).

Shallow flooding is predominately caused by intense rainfall occurring locally over short period, thus are difficult to forecast and prepare for (Falconer et al., 2009). It describes the combined flooding in urban areas during heavy rainfall and pluvial flooding which results from rainfall generated overland flow and ponding before the runoff enters any watercourse, drainage system or sewer, or cannot enter it because the network is full (Falconer et al., 2009). Pluvial flooding often occurs in combination with coastal and fluvial flooding, although typically only a few inches deep, it can cause significant property damage.

Storm surge, produced when high winds from hurricanes and other storms push water onshore, is the leading cause of *coastal flooding* and often the greatest threat associated with tropical storm. Storm surge is defined as the abnormal rise of water generated by a storm, over and above the normal astronomical tide, and is expressed in terms of height above predicted or expected tide levels (National Oceanic and Atmospheric Administration (NOAA), 2013). About 8,651,000 people, or slightly more than 3% of the total US population, live in Zone V and coastal Zone A, spread over 48,406 sq. mi, which is also defined as 1% annual chance coastal flood hazard areas by FEMA (2011) (Crowell et al 2010). Similarly, in the United States, over half of the nation's economic productivity is located within coastal zones (NOAA,2017).

2.3.2 Flash Floods

According to National Weather Service (2006), "*Flash flood is a rapid and extreme flow of high water into a normally dry area, or a rapid rise in a stream or creek above a predetermined flood level, beginning within six hours of the causative event (like intense rain, dam failure, ice jam). However, the actual time threshold may vary in various parts of the country. Ongoing flooding can intensify to flash flooding in cases where intense rainfall results in a rapid surge of rising flood waters*." In the USA flash floods are regarded as having a time to peak up to 6 hours for catchments up to 400 km^2 (Georgakokos and Hudlow, 1984). Flash floods are very threatening and destructive not only because of the force of the water but also due to the impact of debris that is often swept up in the flow. Channel velocities of 9 ft per second (fps) can move 90-pound rock, similarly, major flash floods that occurred in the Big Thompson Canyon in Colorado in 1976, where velocities exceeded 30 fps, moved boulders weighing 250 tons (FEMA, 2006). They can even occur even if no rain has fallen, for instance after a levee or dam has failed, or after a sudden release of water from debris or ice jam like breaking of glacial lake, or heavy rain in location on same river basin but beyond sight.

A flash flood that occurred in 1992, near Cherokke, North Carolina had the crest of flood measured at *ten ft* above the calculated 100-year flood plain for that area (FEMA, 2006). The occurrence of flash flooding is one of top concern in hydrologic and natural hazards science in terms of both the number of people affected globally and the proportion of individual fatalities.

Jonkman and Kelman (2005) examined data from substantial number of flood events over each continent, which occurred between January 1975 and June 2002, showing that flash floods out of that sample caused around 1550 casualties per year. Moreover, the study showed that flash flood mortality (computed as the number of fatalities divided by the number of people affected) is higher than that of other natural hazards. hazards are expected to increase in frequency and severity, through the impact of global change on climate.

2.3.3 Lessons learnt from flooding events

One of the most influential research work by Robertson et al (2007) describes the performance of engineered buildings and coastal bridges when subjected to Hurricane Katrina storm surge. The hydrodynamic uplift on bridges deck were estimated to be 30% more than the self-weight of the bridge for all bridges except one in which case self-weight was 64% greater than hydrodynamic uplift. Most of the spans of those bridges were found displaced from the supporting piers during hurricane apart from the bridge with higher self-weight. Similarly, due to buoyancy all the bridges were found to have less than 28% residual self-weight when submerged except one with residual self-weight of 39.5% of the unsubmerged self-weight, which was the only bridge not damaged by storm surge. The effect of buoyancy in structural failure is very well justified by this research.

A research conducted by Kelman and Spence in 2004 indicated that typical flood loss patterns can be described as follows: rising floodwater or groundwater soaks through the building walls, floors and furniture. They summarized the damage related to hydrostatic flood action can be greatly enhanced by sediments deposits and the loss profile due to mostly hydrodynamic action was found to be influenced by flow velocity. According to them, a building can be buoyed if the force of rising floodwater or groundwater exceeds the counterweight of the building, so buoyancy of a building can be mitigated by intentionally flooding the basement.

Thieken, Muller, Kreibich and Merz in 2004 conducted a research aftermath of a severe flood event in Germany to determine the flood damage and its influencing factors. They determined significant difference in loss ratios of buildings affected by groundwater rise or stagnant flow on the one hand and very high velocities on the other hand. However, the number of valid cases found was comparatively low. Buildings affected due to moderate or high flow velocities both has same level of loss ratio, so they couldn't be distinguished from each other. They also concluded that the influence of flow velocity on loss ratios is not as clear as the influence of water level and flood duration. According to them, different flood events, such as (slowly rising) river floods, flash floods, storm surges, inundation due to levee breaches or fast ground water rise cause various kinds and extents of losses. Finally, they concluded that flood loss estimation should focus on the quantification of flood impact variables, but not on the stillwater depth alone.

Yeh et al. in 2014 observed the failure mode of reinforced concrete buildings due to overturning moment and sliding during storm surge due to deep submersion of building into water and high flood velocity, exacerbated by entrapped air within building and debris impact. Buildings with breakaway walls and breakaway holes were found capable of surviving storm surge due to tsunami because it allowed water to be intruded inside the building and thus intruded water weight acting along gravity helped to stabilize the building. Making use of tsunami inundation data of the 2011 Tsunami in the town of Onagawa, they demonstrated that the buoyancy force calculations are essential to evaluate global building failures, but the development of buoyancy force (if acting through soil pores) follows a process of diffusion, and therefore, does not respond instantaneously to the inundation level when the foundation is deep. They also concluded the building could be stabilized by flooding the interior of building by adding extra body force acting downward to resist from the failure.

Many of the buildings that failed in hurricane Ivan were built in areas that were designated as Zone B, C, or X at the time of construction and were built on shallow foundation because it was not required for buildings to be elevated in those Zones. These areas were exposed to V-zone conditions because of erosion that occurred during that storm and the long-term erosion (FEMA, 2013).

2.3.4 Effect of Water in timber and its products

Timber buildings, owing to their hygroscopic nature, face unique risk from flooding. The structural property of a member depends largely on the moisture content (MC) (Breyer et al 2014). Increment in MC lead to a reduction in the mechanical properties of timber (Rammer and Winistorfer, 2001) as well as a loss of strength in oriented strand board (OSB) (Wu and Piao, 1999). In flood events, buildings are sometimes under water as long as for a week or more, so it is reasonable to assume increment in moisture content in exposed components, which may ultimately lead to temporary or permanent reductions in the mechanical properties of timber structure. Increase in MC will affect the mechanical properties of the nailed or bolted shear connections between the OSB sheathing and timber frame, wall to floor shear connections and floor to foundation shear connection. Apart from weakening the connections, the increment in MC in timber structures (2” to 4” Sawn Lumber) decreases structural properties like bending, shear, compression parallel and perpendicular to grain, Elasticity and minimum elasticity, which is well evident from the reduction factor (C_M) used in National Design Specification for Wood Construction with Commentary (NDS, 2015).

It is very common for a building lying in a flood zone to be flooded several times in its life span. Following flooding it is very important to dry submerged part sufficiently; however, it is important with timber as its frequent or long-term increment in moisture levels either due to submersion in flood or slanted rain hitting structures surface will cause decay in the material (Rammer and Winistorfer, 2001). Below 20% MC the timber is generally considered to be at reduced risk from the rot (Garvin et al., 2005).

Post flooding structures can be dried by natural ventilated drying, convection drying and artificial dehumidification (Garvin et al., 2005). They are different from each other in measures of controlling and alternating of temperature and relative humidity. The methods of drying structure play vital role in regaining of Strength and Stiffness. Experience from drying fresh felled lumber suggests that incorrect drying can result in undesirable effects such as warping, cracking or case hardening (Forest Products Laboratory (US), 1999). It is therefore very important to understand the influence of artificial dehumidification and convection heating have on mechanical properties. Removal of moisture using such accelerated method may result in changes to the timber and sheathing that affect their ability to carry design loads safely.

Exposure of timber frame to flood results in weakening of timber in a way that does not happen with other structural materials by the virtue of its hygroscopic nature. Despite the increase in risk of exposure of buildings to flood, there is very little research work carried out to formulate the relationship between drying methodology and recovery of mechanical properties. Escarameia et al. (2007) studied several different wall constructions and investigated drying rate but no strength tests were conducted.

Mtenga et al. (2012) studied the capability of Metal-plated wood truss system for three wetting cases: 1) 4-h shower, 2) alternating cycles of a period of shower wetting followed by a period of no shower, and 3) 24-h soaking in a water tank. Results of tests conducted after wetting and after wet-dry cycles were compared with results of dry control test. The wet joints were found losing over 40% of their load-carrying capacity in all three wetting cases, however upon drying the strength loss was about 10%. Likewise, the stiffness of the joints deteriorated, with stiffness loss ranging from 12 to 37%. In contrast to the load carrying capacities, there were insignificant recoveries in joint stiffness upon drying the specimen.

Similarly, Bradely et al. (2014) also studied a series of drying environments and found that to optimize overall recovery of mechanical properties of a nailed timber to OSB connection after wetting, heating to 38˚C and lowering Relative Humidity to 40% is suggested. Using the optimized drying environment, recovery of 68% of the ultimate strength, 62% of the yield strength and 84% of the stiffness can be expected, which is close enough to the results obtained by Mtenga et al. (2012). Comparison of effect of temperature and relative humidity on recovery of structural properties of a nailed timber to OSB connection indicates the maximum recovery is influenced by relative humidity. Areas with a lot of surface water, such as coastal areas and the great lakes region where floods are more frequent, have high humidity levels due to evaporation, which adversely effect in recovery of ultimate strength, yield strength and stiffness.

2.4 Wind

2.4.1 Introduction

High wind is one of the most significant natural hazard that affects the coastlines and midlands of the United States. High winds can be originated from Tropical storms, hurricanes, typhoons, other coastal storms, and tornadoes. Winds are capable of imposing large lateral and uplift (anti-gravity) load on buildings. When the effect of all possible types of impacts are not considered while designing a residential building, it may result in collapse or unpleasing deflection during high wind events.

2.4.2 Development of Tropical Cyclones

Tropical storms have 1- minute sustained winds averaging 39 to 74 miles per hour (mph). When sustained winds intensify to greater than 74 mph, the resulting storms are called hurricanes, also known as tropical cyclones (in North Atlantic basin or in the Central or South Pacific basins east of the International Date Line) or typhoons (in the western North Pacific basin) (FEMA, 2011). Hurricanes are large storms with diameters on the order of 200 miles or more that are generally originated in the tropics between latitudes 5 and 20. To quantify the strength of cyclones, Saffir-Simpson Hurricane Scale (SSHS) was developed, which has now been replaced by Saffir-Simpson Hurricane Wind Scale (SSHWS), which uses 1-minute sustained wind speed at height of 33 ft over open water as the sole parameter to categorize storm damage potential.

Table 2 Saffir- Simpson Hurricane Wind Scale

Scale Number (Category)	Over water Wind Speed in mph 1-Minute Sustained (3-Second Gust) [a]	Property Damage	Expected Damage to Wood-Frame Houses
1	74-95 (89-116)	Minimal	Poorly constructed house- Loss of roof covering, gable end damage

2	96-110 (117-134)	Moderate	Well-Constructed house- damage to roof shingles, vinyl siding, soffit panels, gutters Poorly constructed house- High chance of roof removal
3	111-130 (135-159)	Extensive	Well-Constructed house- Major roof and siding damage Poorly constructed house- Possibility of total destruction
4	131-155 (160-189)	Extreme	Well-Constructed house- Removal of roof decking and gable end failures Poorly constructed house- Complete Collapse
5	>155 (>189)	Catastrophic	Well-Constructed house- Loss of most of the roof structure and/or exterior walls High percentage of wood-framed homes completely destroyed

Source: FEMA (2011) (a) Peak 1-min wind speed at an elevation of 33 ft over unobstructed exposure

2.4.3 Lessons learnt from High Wind events:

Taher in 2010 recommended building forms, roof shapes and slopes, construction materials and methods, foundation, and sustainability for multi-hazard design. The sections concerning building forms and roof shapes are the most relevant that addresses wind resistance. To design a wind resistant low-rise structure, Taher suggested to use hip roofs with slopes on all four side in hurricane prone areas rather than two-sided gable roofs. She also suggested to use optimal roof slope, which is approximately 30 degrees and construct openings in the negative wind pressure regions on the roof. The use of hip roofs, optimal roof slopes and roof openings reduces the uplift forces due to wind pressures and limits the imbalance of pressure between the interior and exterior faces of the roof. Structural isolation of the two halves of double span roofs is necessary to prevent progressive collapse. Treatment of roof edges as shown in Figure 2.6, developed by the Centre

Scientific et Technique de Bâtiment (CSTB) in France reduces local pressures by distributing air flow at edges and reduction in roof overhangs reduces the uplift pressure acting on the overhangs.

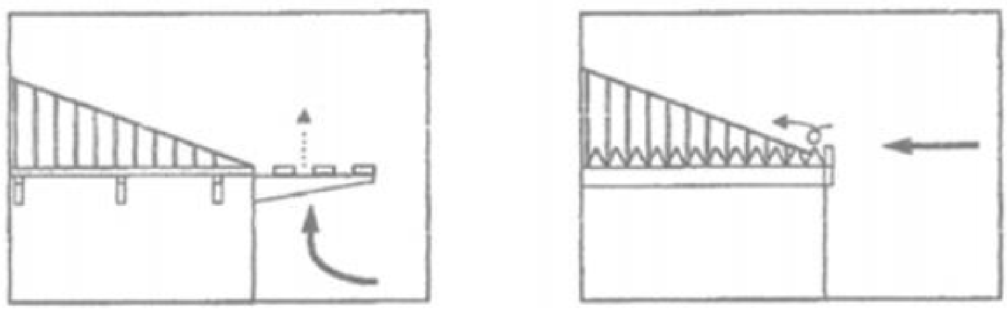


Figure 2.6 Roof Treatment Systems suggested by CSTB; Horizontal Grid overhang (left); Notched Frieze Along Perimeter (right)

As documents by Wood (1976), the high winds associated with Great Atlantic Storm of 1962 (Nor'easter) included peak gusts of up to 84 mph and continued for 65 hours, through five successive tides. The combination of sustained high winds with spring tides resulted I extensive flooding along the coast from the Outer Banks of North Carolina to Long Island, NY. Waves 20 to 30 ft high were reported, which caused severe beachfront erosion, inundated subdivisions and Coastal industrial facilities, toppled beachfront houses and swept them out to sea, destroyed coastal roads and interrupted railway transportation in many areas. In all, the property damage was estimated at half a billion dollars (in 1962 dollars).

Another major example of multi-hazard is Hurricane Floyd, Florida to Maine, 1999. Sustained tropical winds and gusts with maximum speed of 104 mph were recorded as far north as New York. Storm surge and torrential rains causes extensive flood damage in North Carolina where up to 20 inches of rain fell. In North Carolina, over 7000 homes were destroyed and 56,000 were damaged. Hurricane Floyd was also a significant storm in the mid-Atlantic, with up to 14 inches of rain falling in parts of Maryland, Delaware, Pennsylvania, and New Jersey. There were 9 recorded floods in Mid-Atlantic rivers. The rains and high winds causes moderated flash flooding to Coastal and inland communities along the East coast (NOAA 2000).

2.5 National Flood Insurance Program

2.5.1 Introduction

A number of federal agencies have been directly or indirectly involved in identifying flood hazards, but out of all agencies Federal Emergency Management Agency (FEMA) is most directly involved in it and in the regulation of development within flood hazard areas, and in responding to flood disasters (Jones, 2017). Every year, flooding causes hundreds of millions of dollars' worth damage to homes and businesses around the country, however, commonly available standard homeowners and commercial property policies do not cover flood losses. So, to meet the need for this vital coverage, the FEMA administers the National Flood Insurance Program (NFIP) (FEMA, 2015). NFIP was established in 1968 for aiding homeowners and communities by identifying risk, managing flood plains, and providing affordable flood insurance (Brody et al., 2013). The Flood Control Act of 1936 represents the federal government's first attempts to manage flood hazards, which focused on building flood projects, such as river training, building dams, and construction of sea-walls (Pasterick, 1998). The major objective was to construct flood control works to keep flood away from properties and to restrict construction through land use regulations (Pasterick, 1998). Flood insurance was not widely available to individuals before 1968, due to excessive cost of servicing claims. The National Flood Insurance Act of 1968 introduced the National Flood Insurance Program (NFIP) to provide flood insurance to individuals and businesses (Michel-Kerjan, 2010).

2.5.2 The Community Rating System (CRS) program

In 1990, Community Rating System (CRS) program was introduced by National Flood Insurance Program (NFIP) to encourage local flood mitigation activity and NFIP participation. CRS recognizes community efforts beyond those minimum standards as required by NFIP by reducing flood insurance premiums for the community's property owner from 5% to 45% as an incentive for new flood protection activities that can help save lives and property in the event of flood (FEMA, 2015).

To participate in CRS program, a community must first be participant of the NFIP. Participation in this program allows the residents in that community to earn premium discounts on their individual policies. CRS participating undertake flood mitigation activities that exceed the NFIP requirements. Flood mitigation activities may take structural or non-structural form. Structural form is centered on large-scale construction like sea walls, channels, river training if necessary, etc. Non- structural plan addresses educating and training, land use planning tools, flood insurance, and emergency and recovery policies (Highfield and Brody, 2013).

An NFIP community, to be eligible for a CRS discount must do Activity 310, Elevation certificates and Activity 510 (Floodplain management Planning), if community is designated as repetitive loss community. In total, CRS grants credit for 19 different activities that fall into four series. Activities apart from Activity 310 and 510 are optional, however, to qualify for maximum 45% discount in Special Flood Hazard Area (SFHA) and maximum 10% discount in non- Special Flood Hazard Area (Non-SFHA) optional activities also needs to be carried out (FEMA, 2015). Credit points accumulated as per the Table 2.2

is summed up to determine the discount. Credit points totaling more than 4500 points qualifies for 45% discount in SFHA and 10% in non-SFHA.

Table 3 Community Rating System (CRS) Activities

Series	Activity Number	Activities	Maximum Points[(a)] (Average Points) [(a)]
300: Public Information	310	Elevation Certificates	116 (46)
	320	Map Information Service	90 (63)
	330	Outreach Projects	350 (63)
	340	Hazard Disclosure	80 (14)
	350	Flood Protection Information	125 (33)
	360	Flood Protection Assistance	110 (49)
	370	Flood Insurance Promotion	110 (0)
400: Mapping and Regulations	410	Floodplain Mapping	802 (65)
	420	Open Space Preservation	2020 (474)
	430	Higher Regulatory Standards	2042 (214)
	440	Flood Data Maintenance	222 (54)
	450	Storm water management	755 (119)
500: Flood Damage Reduction	510	Floodplain Management Planning	622 (123)
	520	Acquisition and Relocation	1900 (136)
	530	Flood Protection	1600 (136)
	540	Drainage System Maintenance	570 (214)
600: Flood Preparedness	610	Flood warning and Response	395 (144)
	620	Levee Safety	235 (0)
	630	Dam Safety	160 (0)

Source: FEMA B- 573 (2015) (a) Maximum and average points are subject to change

2.5.3 Flood Maps and Flood Management Regulations

FEMA provides participating communities with a Flood Insurance Rate Map (FIRM) and Flood Insurance Study (FIS). Depending on the severity, several areas of hazards are identified on FIRM. Special Flood Hazard Area (SFHA), also know known as base flood area or 100-year flood plain is defined as the area that will be inundated by the flood event having a 1- percent chance of being equaled or exceeded in any given year (FEMA, 2013). It is mandatory to have flood insurance for all building types constructed in all SFHAs. Design flood refers to the locally adopted regulatory flood; if a community regulates the minimum requirement set by National Flood Insurance Program (NFIP), the design flood is identical to the base flood (the 1% annual chance flood or 100-year flood), however, if the community requires to exceed minimum NFIP building elevation requirements, the design flood can exceed the base flood (FEMA, 2011).

According to FEMA (2013), V Zone is defined as the portion of the SFHA that extends from offshore to the inland limit of the primary frontal dune along an open coast, and any other area subject to high-velocity wave action from storms or seismic sources. FEMA (2013) has set minimum requirement for buildings built in Zone V (Coastal High Hazard Areas), which are listed below:

1. The building must be elevated on open type of foundation like pile, post, pier or column foundation such that the direction of flow of water is unobstructed.
2. The building must be adequately anchored to the foundation
3. The building must have the bottom of the lowest horizontal structural member at or above the Base Flood Elevation (BFE). The bracing used brace piles are however not required to be above BFE. The strengthening provided by bracing shall not be considered for Strength design and should only satiate Serviceability requirement.
4. The building design and method of construction must be certified by a design professional.
5. The area below the BFE must be free from obstructions. However, it can be enclosed if the enclosure is made of light weight wood lattice, insect screening, or breakaway walls.

6. Any space below the DFE cannot be used for residential and non-residential purpose, however, it can be used for parking, access and storage (Figure 2.7)

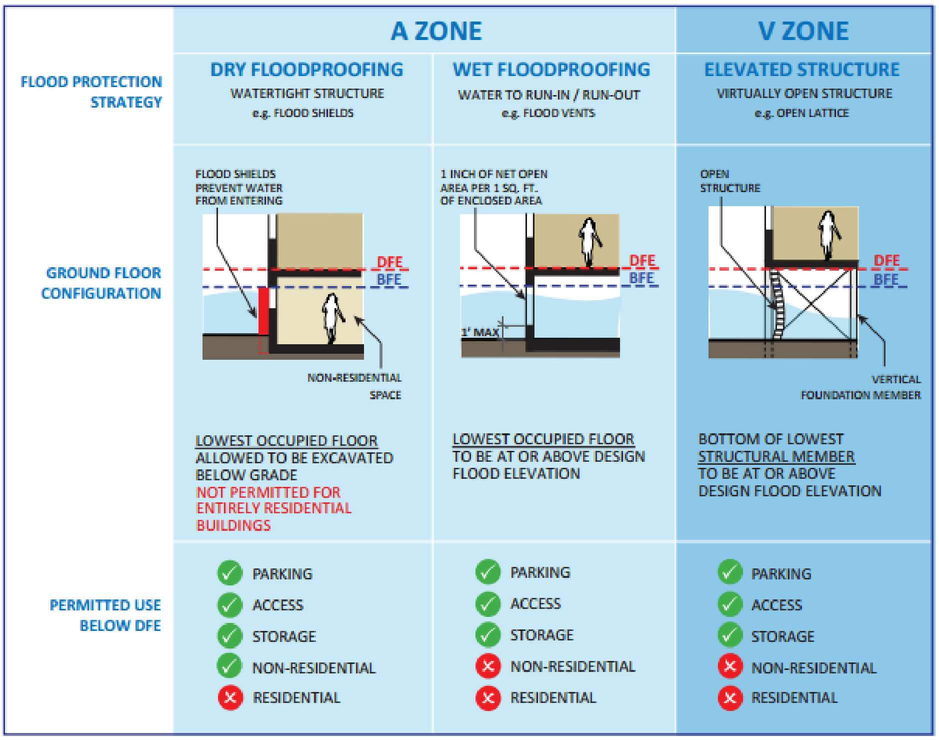


Figure 2.7 Design and Construction requirements for buildings in the SFHAs (Source: Department of Planning, The City of New York)

An “A Zone” is located either inland of a V Zone in Coastal area or adjacent to open water where no V zones are mapped and is defined as the portion not mapped as a V Zone in SFHA (i.e. noncoastal area) (The City of New York, 2013). A zone in coastal areas are subject to wave heights less than 3 ft. According to FEMA (2013), In Zone A, the NFIP requires that the top of the lowest floor of a building must be at or above the BFE; however, there are no standards for foundations other than the overall performance standard that the building be anchored to resist floatation. In an A zone, non-residential buildings can be flood-proofed with their walls made substantially impermeable to the passage of floodwater, however, any floor below DFE cannot be used for residential purpose. As shown in figure 2.7, floors cannot be used for residential and non-residential purpose below DFE, if wet floodproofing is done. However, if dry floodproofing is done, apart from residential purpose, floors below DFE can be used for other purposes like parking, access, storage and non-residential (FEMA, 2013 and The City of New York,2013).

Zone X, B and C are the identified areas outside of the SFHA; Zone B and shaded Zone X (Figure 2.8) identify areas subjected to inundation by the flood that has a 0.2-percent probability (or 500-year flood) of being equaled or exceeded during any given year and the rest (i.e. Zone C and unshaded Zone X) identify areas above the level of the 500-year flood (FEMA, 2013). It is also termed as areas of moderate or minimal hazard from the principal source of flood in the area. NFIP doesn't have any building design and elevating requirement for buildings built outside the SFHA (FEMA, 2013).

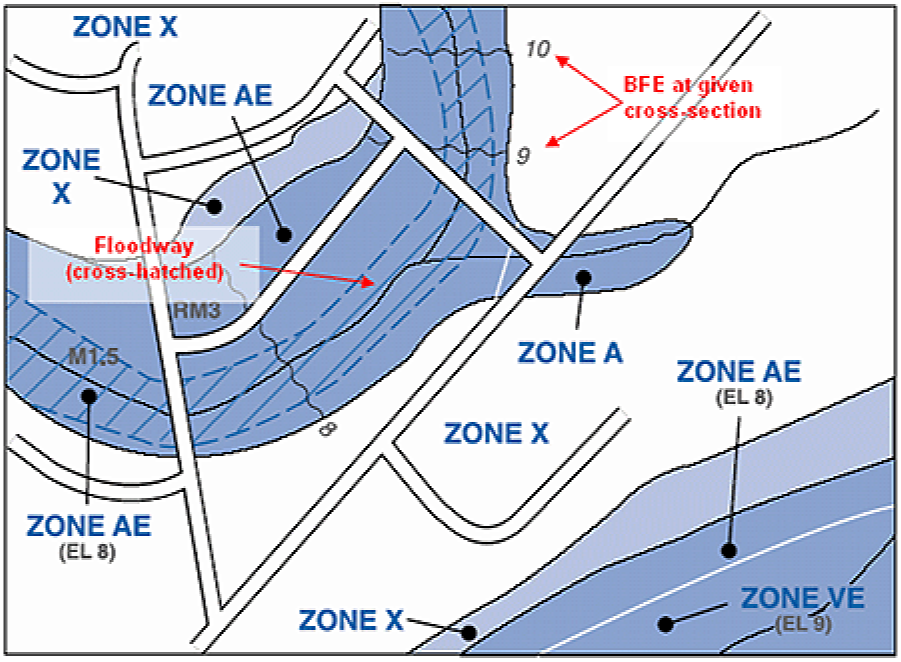


Figure 2.8 Sample Riverine FIRM (Source: FEMA)

Flood hazard maps should be studied carefully for multiple reason, each of which may affect the accuracy of the flood hazard zones and BFEs shown. Although not usually required by communities, making use of updated hydrologic and hydraulic models that has evolved over time and better topographic data now than was used as the basis for the maps may be important for building planning and design in the case of old and out-of-date flood hazard maps (Jones, 2017).

2.5.4 Flood proofing

The Base Flood Elevation (BFE) on FEMA flood maps serves as the threshold to which flood-resistant construction is required using flood damage-resistant materials (The City of New York, 2013). It is necessary to elevate, or flood proofed (where permitted, Figure 2.7) the first occupiable floor to ensure that building remain structurally safe and to protect the contents inside the building. Additional freeboards above the BFE provides margin of safety against the uncertainties of flood modeling and often compensate for the many unknown factors that could contribute to flood heights, including sea level rise (The New York City, 2013). Design Flood Elevation (DEF) is thus obtained by adding the community regulated mandatory freeboard and BFE. The minimum freeboard required is different for various categories of buildings, ranging from minimum requirement for Category I buildings that represent low hazard to life in the event of failure (like storage building) to maximum requirement for Category IV buildings that are designed as essential facilities (like hospital). There are two types of floodproofing allowed as per FEMA standards for buildings in A Zone: Dry Floodproofing and Wet Floodproofing.

Dry Floodproofing is done to make walls watertight up to at least the level of the DFE by using sealant, flood shields and aquarium glasses, by strengthening structural components to resist hydrostatic and hydrodynamics forces from flood waters and protecting utilities from damage (Figure 2.9). A wide variety of materials and devices have been developed to make building walls, floors, openings, penetrations and utilities watertight during flooding. Flood shields, panels, doors and gates are typically used to prevent water flow inside the building. Residential floors below the DFE cannot be dry floodproofed, however, there is no requirement set for dwelling units above DFE.

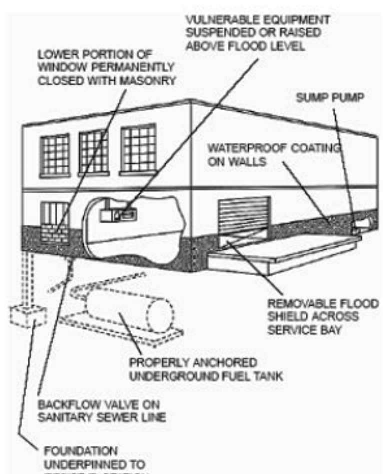


Figure 2.9 Typical- Floodproofing Techniques (*Source*: FEMA)

Similarly, Wet floodproofing in A zone is designed to allow floodwater pass through the building without making use of any mechanical or electrical equipment's. For preventing walls and slabs from collapsing due to differential hydrostatic pressure on interior and exterior side, openings are provided at the ground floor not more than 1 ft to allow water to flow in and out at the appropriate rate. A minimum of two openings must be provided on different sides of each enclosed area, having a total net area of not less than 1 square inch for every square ft of enclosed area subject to flooding; this requirement is waived if openings are engineered and certified (FEMA, 2012).

Results of dry-flood proofing can be pleasing for flood exceeding the base flood by small amount as it helps keeping the floor surface dry by preventing infiltration, however, improper calculations of lateral hydrostatic forces and the vertical buoyancy forces on the structure containing dry-floodproofed walls can expose building to uplift, overturning and sliding mode of failures. As seen in figure 2.10, a light weight building with a dry-flood proofed basement is lifted out of the ground by buoyancy forces during flood (Jones, 2017).

Figure 2.10 Failure of Lightweight building with a Dry- Floodproofed Basement, lifted out of the Ground by Buoyancy forces during a flood (Source: USACE)

2.6 Modes of failure

2.6.1 Introduction

Yeh et al (2014) categorized building failure subjected to lateral flood loads into three classes. First, in the case of global stability failure, a building with lateral resisting system overturns and/or slides as a rigid body. In the second class, failure of one or more individual structural components (e.g. beams or columns) leads to the collapse of the building. The impact of debris on any particular component can lead to the failure of individual structural component. The third class of failure is associated to foundation failure due to soil instability and scouring. The scope of the research is to study the performance of building subjected to global stability failure.

2.6.2 Overturning

Overturning is a global stability failure, wherein building overturns as a rigid body about a point at the level of foundation as shown in Figure 2.11, typically with the lateral resisting system. The net overturning moments are determined by establishing the pivot point for analysis, multiplying individual forces (wind, flood and weight) by their respective moment arms, and summing all moments due to individual forces. The most likely overturning direction is along the short dimension of the building caused by lateral forces. To prevent building from overturning, the resulting moment must be less than or equal to zero, where anticlockwise moment due to effective dead load is considered negative and clockwise overturning moment due to lateral load is considered positive (Equation 12). If the resulting moment is more than zero, pile uplift frictional resistance provides resistance against overturning, thus while designing pile foundation of a building it is necessary to understand the significance of overturning moment. Only the portions of the roof and floor live loads that are part that cantilevers out past the leeward pier will contribute to the overturning. The most conservative load combination to determine the capability against resisting overturning moment is the one which considers minimum dead weight along with uplift forces. The worst case horizontal load combination is load combination 7: $0.6D + 0.6W + 0.75F_a$.

2.6.3 Sliding

Sliding failure is a lateral force phenomenon. Lateral forces are resisted by the walls of the structure, buried fters, and the slab, in other words, horizontal resistive force is the sum of foundation resistive force and resistive force from structure self-weight. According to FEMA (P-259, 2012), Foundation resistive force, (*r*) and resistive force from the structure self-weight is obtained using following equation:

$$resistive\ force_{founation}, r = \left(k_p\right)(\lambda_{soil})\frac{d^2}{2} \text{ in lb./ft.} \tag{6}$$

$$resistive\ force_{self-weight} = (coefficient\ of\ friction)(Dead\ load) \tag{7}$$

Where, $k_p = tan^2\left(45^\circ + \frac{\phi^\circ}{2}\right)$; ϕ is the soil angle of internal friction

λ_{soil} is the density of soil and d is the depth of soil from top of soil to top of fting.

The worst case horizontal load combination is load combination 7: $0.6D + 0.6W + 0.75F_a$

2.6.4 Uplift

When the effective dead load of building, obtained from Equation (9) is negative, the structure is incapable of resisting buoyancy forces. The dead weight of the building must resist the uplift forces imposed by wind and buoyancy on the building. The most conservative load combination to determine the capability against resisting uplifting forces is the one which considers minimum dead weight along with uplift forces. Load combination 7 from Table 5 is the most conservative load combination. FEMA (P-259, 2012) suggest the following equation must be satisfied to resist uplift and buoyancy.

$$0.6D + 0.6W + 0.75F_a > 0 \text{ to resist uplift and buoyancy} \tag{8}$$

If the buoyancy forces are greater than the resisting force of the slab, it might cause the slab to crack or even rise out of the ground. So, it is necessary to check that the slab is capable to resist vertical and horizontal flood forces due to negative shear and bending moment acting upward.

2.6.5 Formulation of Global Structural failure assessment

According to Yeh et al (2014), It is important to recognize the role played by effective gravitational body force to stabilize the structure against the lateral external forces causing the instability of a structure leading to failure. The effective body force (D_e) consists of the weight of building (*D*), and the negatively acting buoyancy force (F_{buoy}) on the underneath of the foundation and Wind uplift force (W_{uplift}), mathematically,

$$D_e(t) = D - F_{buoy}(t) - W_{uplift} \tag{9}$$

The effective body force is obtained by adding the weight of building (*D*), which is obtained by adding weight of superstructure and substructure of the building. Weight of Substructure in case of LFWS Slab-on-Grade type is obtained by manually calculating the weight of reinforced concrete slab as described in Chapter 4.2.1 and computed as in Appendix A.6. The weight of superstructure is obtained from 3D model prepared in RISA 3D after designing and detailing for gravity and lateral loads and checking it against serviceability requirement. The buoyancy force is calculated as described in Chapter 3.3.1. Figure 2.11 shows the general model considered for assessment of global stability.

Two stability conditions are considered: 1) incipient sliding, and 2) incipient overturning. Incipient sliding is the case that occurs when sum of lateral forces due to flood and wind exceeds the resisting force. Incipient sliding occurs when the sum of horizontal forces (F_H) is greater than zero,

$$\sum F_H\,(t) = F_X(t) - R_X(t) \tag{10}$$

Where, $F_X(t)$ = sum of external lateral forces acting on the building (lateral component of wind and flood)

$$R_X(t) = \mu D_e(t) - (F_A - F_P) \tag{11}$$

Where, $\mu = \tan(\phi)$ is the friction coefficient (ϕ *is the friction angle*)

F_A & F_P are the active and passive soil pressure forces

Similarly, incipient overturning results when the sum of clockwise moments about reference pivot "point" becomes greater than zero,

$$\sum M_{point}(t) = \left(\frac{1}{2}d_s + h_f\right)F_{a\,(dynamic)} + \left(\frac{1}{3}d_s + h_f\right)F_{a\,(static)} + \sum W_X(h) - \frac{1}{2}aD_{e\,(t)} \tag{12}$$

Where, "a" is the dimension of building in the direction of flow, h_f is the depth of embedment of pivot point, h is the height of diaphragm with respect to pivot point, d_s is the stillwater depth, F_a is the lateral flood load hydrodynamic and static in nature and W_X is the wind force acting on diaphragm in the direction of flow and M_{point} is the moment about pivot point.

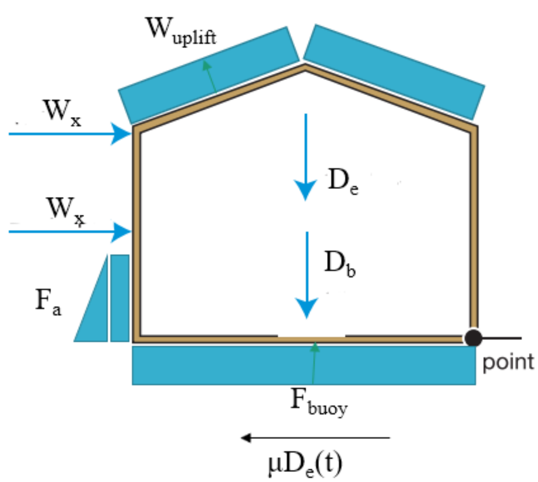


Figure 2.11 Free body diagram used in global stability analysis

Chapter 3 Characteristics of Hurricane and Floods

3.1 Introduction

To know the performance of any structure subjected to multiple forms of hazards, it is very important to be able to understand the characteristics of every individual hazards. Hazards histories are often the basis of learning, so it also equally important to know the past hazard events. This chapter describes the factors that influence wind and flood hazards and combination of wind and flood hazards. This chapter also summarize the combination of various forms of flood loads depending on the locality.

Persistently acting high winds cause unacceptable amount of lateral displacement in mid and high-rise buildings causing occupant discomfort. Thus, serviceability limit state is often controlling condition for wind design. In low-rise Timber residential buildings, when the roof remains intact and connected to walls, the entire structure has been found able to perform satisfactorily in both Ultimate and Serviceability Limit state (Reed et al., 1997). Diaphragm keeps walls intact to each other and unit them to function against lateral load. However, if roof fails there will be no combined lateral resisting system and thus the chances of failure increases.

Flood damages can be a factor for both Ultimate and Serviceability Limit state depending on the intensity of flood and building construction methodology. Smaller depth and slow-moving flood are not detrimental in nature but the rising flood water soaks building walls, floor and furniture. However, even smaller depth flood can result in uplift failure due to the developed buoyancy force that exceeds the counter weight of the building. Buoyancy of a building can be prevented by flooding of the basement on purpose (Thieken et al, 2005). Also, other intangible damages like deterioration of timber connections strength and slow undermining of foundation can be caused by slow moving and lower depth flood (Bradely et al. 2014). High velocity flood, if occurring with larger depth can cause devastating damage because of imposing hydrodynamics and the impact action caused by debris contained in high speed flood.

3.2 Wind Load characteristics

Gust factor approach based on the quasi-steady theory is adopted to predict the alongwind response; alongwind motion primarily results from pressure fluctuations on windward and leeward faces and follows fluctuations in the approach flow. Equivalent static loading is determined from a gust factor, which represents the most probable or mean extreme wind velocity value or resulting load effect (Kijewski, T and Kareem, A., 2001).

The effect of wind loads in a building depends heavily on multiple factors like air density, wind velocity, wind direction, structure shape, and structure stiffness, which are either considered explicitly in design code or included implicitly through scaling factor (Yang, 2006). All five factors that influences design wind loads are incorporated into the code using coefficients, either in the velocity pressure (q_z) *or* through the gust effect factor (G). Of the five factors, as defined by Yang (2006), air density is the only one that is considered as constant across all structures, which is included in the wind design calculation using Bernoulli's equation for fluid flow as in the equation below.

$$q = \frac{\rho V^2}{2} \tag{13}$$

Where,

q= static wind pressure

ρ = mass density of air

V = wind velocity

ASCE 7-10 (ASCE Standard, 2010) defines the static wind pressure (q_z) making use of Bernoulli's equation that takes air density as a constant. To determine a static wind pressure in pound per square ft (psf) with input wind velocity in miles per hour, the coefficient $\frac{1}{2}\rho$ of the Bernoulli's equation is 0.00256, which fluctuates with temperature, humidity and altitude but the changes are small enough that no further correction factors are required (Yang, 2006). Below is ASCE equation for velocity pressure, q_z (lb/ft^2) which varies along the height of the structure (z).

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \tag{14}$$

Where,

K_z = velocity pressure exposure coefficient at height z

K_{zt} = topographic factor

K_d = wind directionality factor

V = wind velocity, miles per hour (mph)

I = Importance factor

The determination of static wind pressure involves computation of multiple coefficients that are used to adjust for differences in exposure condition, topography, direction, and the building importance, as in Equation (14). It has been recognized that for many tall buildings, the crosswind and torsional responses may exceed the alongwind response in terms of both limit state and serviceability, however, most existing codes and standards ignores crosswind and torsional responses due to its complexity (Kareem, A., 1985). The crosswind motion is attributed by fluctuations in the separating shear layers and the torsional motion is due to the imbalance in the instantaneous pressure distribution.

It is obvious that higher wind velocities result in great loads because the kinetic energy of the moving air is directly proportional to the square of its velocity. Wind direction is generally unpredictable. ASCE 7-10 (2010) accounts for wind velocity and direction making use of multiple coefficients and factors. The magnitude of wind velocity is governed by multiple elements including geographic location, topography, and height of the building. Geographic location of the building is the principal element to determine the magnitude of wind velocity. In ASCE 7-10, basic wind speed (V), which is the nominal design 3-second gust wind speeds in miles per hour is determined using maps at 33 ft ground for exposure C category and different occupancy category of buildings and other structures. It corresponds to approximately a 3% probability of exceedance in 50 years. Similarly, building exposure is one of the element that controls wind direction.

Similarly, a topographic factor (K_{zt}) accounts for differentiation of wind velocity based on the location of building, like wind velocity increases for building on isolated hills and ridges. It is applied to accommodate the wind speed-up effect. Building height also influences the magnitude of wind velocity, as it increases with increment in height as shown in Figure 3.1.

$$K_{zt} = (1 + K_1 K_2 K_3)^3 \tag{15}$$

Where, K_{zt} is a topographic factor, K_1 is the factor to account for shape of topographic feature and maximum speed-up effect, K_2 is the factor to account for reduction in speed-up with distance upwind or downwind of crest and K_3 is the factor to account for the reduction in speed-up with height above local terrain.

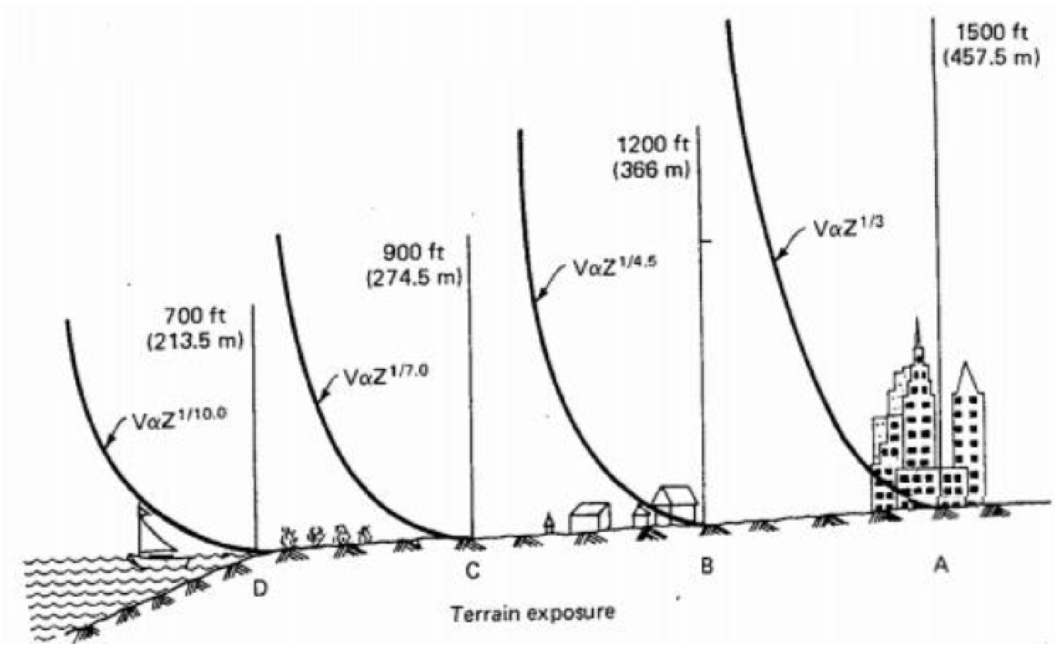


Figure 3.1 Variation in wind velocity for different exposure condition and height (Yang, 2006)

For the calculation of basic wind load, the exposure condition of the surrounding plays significant roles and to take this factor in consideration ASCE 7-10 classify the exposure condition in four categories: Exposure A, Exposure B, Exposure C and Exposure D, as shown in figure 3.1. These categories range from flat and unobstructed areas in Exposure D to suburban and urban areas in Exposure B and A, respectively. The effect of varying exposure condition is implied in the wind load calculations using different pressure exposure factors (K_z). Similarly, the directionality factor (K_d) accounts for the varying direction of wind for different building types.

The previously computed gust effect factor incorporates the characteristics of the structure like building stiffness and shape and its dynamic response into the design wind load. All these factors are accounted in ASCE 7-10 (2010) making use of the natural frequency of a structure, background response factor (Q), resonant response factor (R) and dimensions in the gust effect factor (G).

According to ASCE 7-10 (ASCE Standard, 2010), For flexible or dynamically Sensitive Buildings, an expression to determine the gust factor, G_f, is defined as:

$$G_f = 0.925\left(\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2 - g_R^2R^2}}{1+1.7g_vI_{\bar{z}}}\right) \tag{16}$$

$$\text{Intensity of turbulence at height } \bar{z},\ I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{\frac{1}{6}} \tag{17}$$

Where, $\bar{z}$ = equivalent height of structure, in ft

Peak factor for background response, g_Q = 3.4

Peak factor for wind response, g_v = 3.4

Peak factor of resonant response, g_R is given by

$$g_R = \sqrt{2\,ln(3600n_1)} + \frac{0.577}{\sqrt{2ln(3600n_1)}} \tag{18}$$

Q, background response factor, is given by

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \tag{19}$$

$L_{\bar{z}}$, the integral length scale of turbulence at the equivalent height is given by

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \tag{20}$$

R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_l)} \tag{21}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{\frac{5}{3}}} \tag{22}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \tag{23}$$

$$R_l = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \; for \; n_1 \, \eta > 1 \tag{24}$$

$$R_l = 1 \; for \; \eta = 0 \tag{25}$$

The subscript l shall be taken as h, B and L which means mean roof height of building, in ft, horizontal dimension of building measured normal to wind direction, in ft, and horizontal dimension of a building measured parallel to the wind direction, in ft respectively.

n_1= fundamental natural frequency

$$R_l = R_h \text{ setting } \eta = 4.6 n_1 h / \bar{V}_{\bar{z}} \tag{26}$$

$$R_l = R_B \text{ setting } \eta = 4.6 n_1 B / \bar{V}_{\bar{z}} \tag{27}$$

$$R_l = R_L \text{ setting } \eta = 15.1 n_1 L / \bar{V}_{\bar{z}} \tag{28}$$

$$\bar{V}_z = \bar{b} \left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} \left(\frac{88}{60}\right) V \tag{29}$$

$\beta = \text{damping ratio, percent of critical (i.e. for 2\% use 0.02 in the equation)}$

$\bar{V}_z = \text{mean hourly wind speed } \left(\frac{\text{ft}}{\text{second}}\right) \text{at height } \bar{z}$

$\bar{b}$, $\bar{\alpha}$, c, l and $\bar{\epsilon}$ are constants that depend on exposure category

V is the basic wind speed in miles per hour

Similarly, for rigid buildings, the gust-effect factor shall be taken as 0.85 or calculated as below:

$$G = 0.925\left(\frac{1 + 1.7\ g_Q I_{\bar{z}} Q}{1 + 1.7\ g_V I_{\bar{z}}}\right) \tag{30}$$

Importance factor (I) plays essential role in scaling the design wind load and acts as additional factor of safety based on the purpose of buildings of different nature of occupancy and use to achieve desired level of safety against risk.

3.3 Flood load characteristics

Flood-resistant design is different than design for other types of environmental loads, as flood resistance depends more on avoiding the hazard and not just resisting the hazards; while all structural elements below the Design Flood Elevation (DFE) must be able to resist the flood loads, the consequences of flood level exceeding the flood proofed elevation even by small amount can be extreme, particularly in coastal and riverine areas with heavy debris loads (Jones, 2017). Flood loads are usually characterized by flood source, depth, velocity, duration, direction, rate of rise and fall, wave heights, flood-borne debris, scour and erosion. It is easy to determine the flood depth making use of Flood Insurance Rate Maps (FIRM) and Flood Insurance Survey (FIS) and the topographic condition around but flood velocity, duration, rate of rise and fall, potential debris loads and scouring, and erosion is difficult to quantify.

The duration of riverine flooding depends on watershed size and longitudinal slope of the valley. For a location with higher slope water drains faster. Larger and shallow watersheds adjacent to large rivers may be flooded for weeks or months. Longer inundation is more likely to cause more damage to structural members than short period flooding because it may make mitigation measures such as dry floodproofing inappropriate because of chances of seepage into the building increases with time.

Floodwater level may fluctuate more often in smaller watersheds. Steep topography with small drainage areas may be subjected to flash flooding, the velocity of the flood is the function of slope of topography and cross section of natural or artificial drainage available. For locations susceptible to flash floods due to insufficient warning time it becomes difficult to relocate temporarily, install shields on windows, doors, and floodwalls and activate pump system. The only possible floodproofing measure that might be effective during the flash flood is active floodproofing measure, which involves human intervention to divert or control the flood. More importantly, rapid rise and fall of flood can also lead to unequal hydrostatic pressures on a building, which is worsen if the buildings exteriors are designed to be watertight.

According to FEMA (FEMA P 55, 2011), for engineering estimates, hydraulic forces imposed on building during flood can be classified as: 1) Hydrostatic load including lateral loads from standing water, slowly moving water and non-breaking waves and uplift (buoyancy) effect, 2) Breaking wave load, 3) Hydrodynamic load from rapidly moving water, including broken waves, and 4) Debris impact load from waterborne objects.

3.3.1 Hydrostatic loads

stillwater or slowly moving water induces horizontal hydrostatic forces against a structure, especially when floodwater levels are different on interior and exterior side. These pressures are always perpendicular to the building surface and increase linearly with head of water below the surface of water. Substantial difference in head in internal and external side of building can cause severe deflection or displacement of building or its component.

Usually during a slow rising flood, there is no difference in floodwater level in interior and exterior side of non-dry-floodproofed building, which cancels the hydrostatic forces acting in a member from opposite side. However, during riverine flash floods or coastal surge, the floodwater level increases on exterior side rapidly but it takes considerable time for flood water to infiltrate inside the building, causing floodwater level difference in interior and exterior side. This will result in hydrostatic force acting inward against the walls of building. Rare but possible, generally after slow rising flood with equal interior and exterior floodwater depth, if there is rapid drawdown in floodwater level in exterior side, it may result in outward pressure on the walls of a building as the retained indoor floodwater tires to escape.

In flooding events with different floodwater elevation in interior and exterior side, the uplift forces acting is due to buoyancy forces, which is equal to the amount of water displaced by building. When the uplift force associated with the flood exceed the weight of the building component, the building will have negative self-weight, resulting floatation of the building. For a building with slab-on-grade type of foundation, the buoyancy force acting on the slab from downward may result in uplift along with slab. In case of pier-and-beam or crawlspace foundation, the uplift force acting on the bottom might result in failure of floor system due to negative bending moment and shear force acting from downward. As seen in Figure 3.2, the foundation and superstructure framing remain intact, however, the floor system received damage because of local failure of deck or joist, either due to inappropriate connection or inadequate section size to resist moment and shear acting from downwards. Similarly, connection failure in between floor system and foundation is also experienced by pier-and-beam or crawlspace foundation due to differential floodwater level in interior and exterior side, which is exacerbated by the uplift force during high wind event.

Figure 3.2 Failure of floor system but major framing system remains intact (Source: FEMA P-55 II, 2011)

According to FEMA (P-55, 2011), Lateral hydrostatic load and vertical buoyancy force can be calculated as following:

$$f_{sta} = \frac{1}{2}\gamma_w d_s^2 \tag{31}$$

$$F_{sta} = f_{sta}(w) \tag{32}$$

$$F_{buoy} = \gamma_w(Vol) \tag{33}$$

Where,

f_{sta} = hydrostatic force per unit width (lb/ft) against vertical element

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water, 64 lb/ft^3 for sea water)

d_s = design stillwater flood depth (ft)

F_{sta} = total equivalent lateral hydrostatic force on a structure (lb)

F_{buoy} = vertical hydrostatic force (lb) resulting from the displacement of a given volume of flood water

Vol = volume of floodwater displaced by a submerged object (ft^3)

w = width of vertical element (ft)

3.3.2. Wave loads

The breaking wave forces acting on wall for brief duration (0.1 to 0.3 second after the wave breaks against the wall) are much higher than the typical wind forces that act on a coastal building, even wind pressures that occur during a hurricane or typhoon (FEMA P-55, 2011). The height of waves and wave crest elevations is the major factor to be estimated to calculate the wave loads, which is estimated using Wave Height Analysis for Flood studies (WHAFIS) model prepared by FEMA. FEMA (2011) categorize wave forces into four categories:

- From nonbreaking waves which are usually computed using hydrostatic forces against walls and hydrodynamic forces against piles
- From breaking waves which has short duration but has large magnitude
- From broken waves which are like hydrodynamic forces caused by flowing or surging water
- Wave slam, where just the top of a wave strikes a vertical wall

According to FEMA (P-55, 2011), The breaking wave on a pile and vertical walls can be assumed to act at the stillwater elevation and is obtained as:

$$F_{brkp}\ (piles) = \frac{1}{2} C_{db} \gamma_w D H_b^2 \tag{34}$$

$$f_{brkp}\ (enclosed\ dryspace\ behind\ wall) = 1.1 C_p \gamma_w d_s^2 + 2.4 \gamma_w d_s^2 \tag{35}$$

$$f_{brkp}\ (equal\ stillwater\ elevation\ on\ both\ sides) \tag{36}$$

$$= 1.1 C_p \gamma_w d_s^2 + 1.9 \gamma_w d_s^2$$

$$F_{brkp}\ (wall) = f_{brkp}(w) \tag{37}$$

Where,

f_{brkw} = total breaking wave load per unit length of wall (lb/ft) acting at the stillwater elevation

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water, 64 lb/ft^3 for sea water)

C_{db} = breaking wave drag coefficient

Recommended value, 2.25 for rectangular and square pile

1.75 for round pile

D = pile diameter (ft) for a round pile or 1.4 times the width of pile or column for a square pile (ft)

H_b = breaking wave height (=0.78 d_s)

F_{brkp} = total breaking wave load (lb) acting at the stillwater elevation

C_p = dynamic pressure coefficient

[=1.6 for buildings that impose low hazard to human life or property during failure

=2.8 for Coastal residential building

=3.2 for buildings that impose substantial risk to human life during failure

=3.5 for buildings designated as essential facilities]

w = width of wall (ft)

d_s = design stillwater flood depth (ft)

In determining the forces above, it is assumed the vertical wall causes a reflected or standing wave against the waterward side of the wall with the crest of the wave at a height of 1.2d_s above the sill water level. If free water exists behind the wall, commensurate reduction in hydrostatic pressure must be done.

The action of wave crest striking the elevated portion of a structure is known as "wave slam". Wave slam introduces lateral loading on the lower portion of the elevated structure and are only computed for buildings that are elevated on piles or columns.

$$F_s = f_s w = \frac{1}{2}\gamma_w C_s d_s h w \tag{38}$$

Where, F_s = lateral wave slam (lb)

f_s= lateral wave slam (lb/ft)

C_s= slam coefficient incorporating effects of slam duration and structure stiffness for typical residential structure (recommended value is 2.0)

γ_w= unit weight of water (62.4 lb/ft^3 for fresh water and 64 lb/ft^3 for sea water)

d_s= stillwater flood depth (ft)

h= vertical distance (ft) the wave crest extends above the bottom of the floor joist or floor beam

w= length (ft) of the floor joist or floor beam struck by wave crest

3.3.3 Hydrodynamic loads

Determining design flood velocities, either in case of coastal or riverine flooding is subject to considerable uncertainty. Flood velocities should be estimated conservatively by assuming the most critical direction of flow relative to the site and by assuming flow velocities can be high. For design purpose, flood velocities in coastal areas should be assumed to be in between the lower bound and upper bound as given below. Flood velocity shall be selected considering numerous factors like flood zones, topography and its slope, distance from the source of flooding, proximity to other buildings or obstructions and possibility of channel formation in between engineered buildings. High velocity flows can be exacerbated by the presence of manmade or natural obstruction along the direction of flow and formation of narrow access path such as channel formed by large engineered buildings. The lower and upper bound velocities can be determined, as per FEMA (P-55, 2011), provided below:

Lower bound $V = \frac{d_s}{t}$ (39)

Upper bound $V = (gd_s)^{0.5}$ (40)

Where, V = velocity of water (fps); d_s= design stillwater flood depth (ft); t= 1 sec; g= 32.2 ft/sec^2

Hydrodynamic force is a function of flow velocity and structural geometry of building and is similar to, but not exactly the same as, the drag force used in the field of fluid dynamics. If water velocities do not exceed 10 ft/s, dynamic effects of moving water are permitted to be converted into equivalent hydrostatic loads by increasing the Design Flood Elevation (DFE) for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only (ASCE, 2010).

$$d_h = \frac{aV^2}{2g} \tag{41}$$

Where,

V = average velocity of water in ft/s < 10 ft/s

g = acceleration due to gravity, 32.2 ft/s^2

a = coefficient of drag or shape factor (not less than 1.25)

FEMA (P-55, 2011) also has similar approach to quantify the hydrodynamic force, which can be calculated as follows:

$$F_{dyn} = \frac{1}{2} C_d \rho V^2 A \tag{42}$$

Where, Horizontal drag force (F_{dyn}) acts at a stillwater mid-depth i.e. halfway between the stillwater elevation and the eroded ground surface, which is applied to obstruction normal to flow having surface area, A. The drag coefficient (C_d) is a function of the shape of the object blocking the path of flow. When an object is round and square, or rectangular pile, the recommended values for drag coefficient is 1.2 and 2, respectively. For any object other than a round, square, or rectangular pile, the coefficient is determined by one of the following ratios (see Table 4). Mass density (ρ) of water depends on the source of water, 1.94 slugs/ft^3 for fresh water and 1.99 slugs/ft^3 for salt water. Velocity of water (V), is assumed to be constant (i.e., steady-state flow) and should fall in between lower and upper bound values as obtained from Equation (44) and (45).

Table 4 Drag coefficients for ratios of width to depth ($\frac{w}{d_s}$) and width to height ($\frac{w}{b}$)

Width-to depth ratio $\frac{w}{d_s}$ or $\frac{w}{b}$	Drag coefficient ($\boldsymbol{C_d}$)
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

3.3.4 Debris load

Buildings are subjected to impact load when debris, ice, and any objects transported by floodwater strikes against the buildings and structures. Debris impact load is very difficult to predict because it is influenced by where the building is located in the potential debris stream. Theoretically, debris impact forces can be determined making use of impulse-momentum principle, which implies, the impulse of the resultant force acting for an infinitesimal time is equal to the change in linear momentum; however, due to uncertainty in the determination of impact time duration, the theory can't be applied in practice (Yeh et al, 2014). The effect of debris load can be quantified to some extent with the knowledge of size, shape and weight of potential waterborne debris, design flood velocity, velocity of potential debris with respect to flood, the duration of impact and location of building. Unlike other forces, debris impact forces occur locally at the point of contact with the surface of building, if the debris is smaller than the building.

Debris impact load is quantified by FEMA (P-55, 2011) making use of equation provided in ASCE 7-10, *Commentary* (2010) as follow:

$$F_i = WVC_DC_BC_{Str} \tag{43}$$

Where, F_i = impact force acting at the stillwater elevation (pound)

W= weight of the object (pound); in absence of information about the nature of potential debris, a weight of 1000 pounds is recommended

V = velocity of water (fps); d_s= design stillwater flood depth (ft); *t*= 1 sec; *g*= 32.2 ft/sec^2

$$\textit{Lower bound } V = \frac{d_s}{t} \tag{44}$$

$$\textit{Upper bound } V = (gd_s)^{0.5} \tag{45}$$

Depth coefficient (C_D) accounts for reduced debris velocity as water depth decreases. The highest value being 1 for floodway or zone V and Zone A with $d_s \geq 5\ ft$

C_D = 1 for floodway or Zone V and Zone A with $d_s \geq 5\ ft$; "0.75" for Zone A with $d_s = 4\ ft$; "0.375" for Zone A with $d_s = 2.5\ ft$; "0.00" for Zone A with $d_s \leq 1\ ft$.

Blockage coefficient (C_B) is used to account for the reduction in debris velocity expected to occur because of the screening provided by trees and other structures located on upstream of structure on which impact load is being calculated. The maximum value being 1 for no upstream screening with flow path wider than 30 ft and minimum value being 0 for dense upstream screening with flow path lesser than 5 ft wide. For limited upstream with flow path 20 ft wide, it is considered to be "0.6". Similarly, for moderate upstream screening with flow path 10 ft wide, it is considered to be "0.2".

Building structure coefficient (C_{str}) is directly proportional to the importance *(C_I)* and orientation coefficient *(C_o)* of building and maximum response ratio. Maximum response ratio (R_{max}) is obtained assuming approximate natural period, T = 0.75 sec for timber pile and masonry column; T= 0.35 sec for concrete pile or concrete or steel moment resisting frame; T=0.2 sec for reinforced concrete foundation wall. Similarly, it is indirectly proportional to duration of impact *(Δt)* which is assumed to be 0.03 sec. It can be obtained as:

$$C_{str} = \frac{3.14\, C_I C_o R_{max}}{2g\Delta t} \tag{46}$$

For timber pile and masonry column supported structures 3 stories or less in height above grade, C_{str} = 0.2

For concrete pile or concrete or steel moment resisting frame supported structures 3 stories or less in height above grade, C_{str} = 0.4

For reinforced concrete foundation walls supported structures, C_{str} = 0.8

Furthermore, the debris damming forces due to the jamming effect of debris on a structure, increases the hydrodynamic forces by increasing the surface area exposed to flow. This force is followed by initially imposed impact force due to debris and is obtained by replacing the width of the structure by the width of the jammed debris or summation of width of portion exposed to only building and width of jammed debris.

3.3.5 Localized scour

Waves and currents during flood conditions create turbulence around foundation and cause scouring around the element. Formation of scour, unlike other already described characteristics doesn't have direct impact on building but it can lead to the failure of foundation due to loss in either bearing capacity or anchoring around the foundation. The determination of localized scour condition requires the knowledge of flood conditions, soil characteristics, and more importantly foundation type.

According to FEMA (P-55, 2011), The localized scour depth around single pile or column is evidently found to be approximately 1 to1.5 times the pile diameter, recommended value being 2 times the pile diameter.

$$S_{max} = 2.0\ a \tag{47}$$

Where, S_{max}= maximum localized scour depth (ft)

a=diameter of a round foundation element or maximum diagonal cross-section dimension for a rectangular element

However, for group of piles determination of total scouring depth (S_{TOT}) is a complex process and is a dependent on flow characteristics (depth, velocity, and direction), wave characteristics (height, period, and direction), structural characteristics (pile diameter and spacing) and finally soil characteristics (Sumer et al., 2001).

$$S_{TOT} = 6a + 2\ (\text{if grade beam and/or Slab} - \text{On} - \text{Grade present}) \tag{48}$$

$$S_{TOT} = 6a \qquad (\text{if no grade beam and Slab} - \text{On} - \text{Grade present}) \tag{49}$$

3.3.6 Requirement for a successful flood resistant building

According to Jones (2017), A successful flood resisting building should be able to exhibit following characteristics:

1. The foundation shall remain intact and be able to function fully following design level flood
2. Breakaway enclosures below the Design flood elevation (DFE) should break free without causing damage to the superstructure and substructure

3. The building envelope shall remain sound and must be accessible and usable after a design level flood
4. If any, flood damage shall be minor and repairable and utilities connection such as Mechanical, Electrical and Plumbing (MEP) shall be intact or easily repairable after a design level flood

3.6 Load combination

The flood loads are added to wind load in the load combinations to account for the strong correlation between flood and winds in hurricane prone regions. Load combination including flood loads in "V or Coastal A zones" and "Non-Coastal A zones" for Allowable stress design and Strength design is done as in Table 5 and Table 6, respectively. Whichever load listed in respective tables produces the most unfavorable effect in the building, foundation, or structural member shall be considered for design.

Table 5 Allowable stress design Load combinations including flood (ASCE, 2010)

Load combination	V or Coastal A zones	Non-Coastal A zones
1	D	D
2	D + L	D+L
3	D + (L_r or S or R)	D + (L_r or S or R)
4	D + 0.75L + 0.75(L_r or S or R)	D + 0.75L + 0.75(L_r or S or R)
5	D + 0.6W + 1.5F_a	D + 0.6W + 0.75F_a
6	D + 0.75L + 0.75(0.6W)+ 0.75(L_r or S or R) + 1.5F_a	D + 0.75L + 0.75(0.6W)+ 0.75(L_r or S or R) + 0.75F_a
7	0.6D + 0.6W + 1.5F_a	0.6D + 0.6W + 0.75F_a

Table 6 Strength Design Load combinations including flood load (ASCE, 2010)

Load combination	V or Coastal A zones	Non-Coastal A zones
1	1.4D	1.4D

2	1.2D + 1.6L +0.5 (L_r or S or R)	1.2D + 1.6L +0.5 (L_r or S or R)
3	1.2D + 1.6 (L_r or S or R) + (L or 0.5W)	1.2D + 1.6 (L_r or S or R) + (L or 0.5W)
4	1.2D + 1.0W +2.0F_a + L + 0.75(L_r or S or R)	1.2D + 0.5W +1.0F_a + L + 0.75(L_r or S or R)
6	0.9D + 1.0W +2.0F_a	0.9D + 1.0W +1.0F_a

Flood loads has various characteristics and not all types of flood loads will act together at certain locations or certain building types. Thus, it is necessary to understand the characteristics of flood based on the location of a building before applying flood loads in a building. Wave loading is not a function of velocity of flow but depends on stillwater depth Equation (34), (35) and (36) and dynamic pressure coefficient; a building prone to shallow and slow rising floods located in noncoastal areas might not be subjected wave hazards. Similarly, hydrostatic loads due to variation in stillwater depth in interior and exterior side can cause overturning moments, however, for slow rising flood building might also be subjected to equivalent counteracting hydrostatic forces from opposite direction. The hydrodynamic forces applied during flooding event depends on stillwater depth and velocity of flow. The upper bound velocity as determined using Equation (39) and (40) may not represent some flood scenarios like flash floods in steep terrains where flood velocity can reach as high as 30 ft per second (fps) (FEMA, 1998). The upper bound velocity for flood water depth 2 ft, 3 ft and 4 ft is equal to 8 fps, 9.8 fps and 11.35 fps, respectively. Thus, it is necessary to study flood characteristics thoroughly for several types of buildings and their location.

According to floodplain management regulations enacted by communities participating in the NFIP, the construction of solid perimeter wall foundation is prohibited in V zones but are allowed in A- zone. It is also unrealistic to assume the impact loads in all piles at the same time as breaking waves or hydrodynamic loads, thus it is necessary to evaluate the strategic locations such as corner of the building. Similarly, it is also unrealistic to assume that all rows of piles will be subjected to breaking wave load at once.

According to FEMA (P-5, 2011), When a pile or open foundation is to be constructed in V zone, following load combination is necessary to be considered for efficient foundation design. It is also recommended to design a pile or open foundation in Coastal A zone is for same load combinations.

$$F_{brkp}\ (on\ all\ piles) + F_i(on\ one\ corner\ or\ critical\ pile\ only) \tag{50}$$

Or

$$F_{brkp}\ (on\ front\ row\ of\ piles\ only) + F_{dyn}(on\ all\ piles\ but\ front\ row) + F_i(on\ one\ corner\ or\ critical\ pile\ only) \tag{51}$$

Where, F_{brkp} is total breaking wave load (lb) acting at the stillwater elevation

F_i is impact force acting at the stillwater elevation (pound)

F_{dyn} is hydrodynamic force acting at mid-height in between stillwater elevation and the eroded ground surface (pound)

Not recommended in practice but in cases where solid or wall foundation is required to be constructed in Coastal A zone, load combination is obtained by adding total breakup wave load (F_{brkp}) on walls facing shoreline including hydrostatic component and hydrodynamic force (F_{dyn}). A high degree of redundancy is mandatory, one way is to assume one corner is destroyed by debris impact force. For solid (wall) foundation in Non-Coastal A-Zone, load combination can be obtained by adding hydrostatic forces (F_{sta}) and hydrodynamic forces (F_{dyn}).

Chapter 4

Model Development

4.1 Introduction

To examine the effect of combined action of flood and wind in a Light Frame Wooden Structure (LFWS) located in Zone A (100-year flood plain) two detailed analytical models are developed, one for single story and next is for two-story. Similarly, to study the performance of a building designed for B, C and X Zone (500-year flood plain) as per current regulations when subjected to flood in Zone A, an additional model is prepared. There are no prior analytical models to examine the performance of LFWS against combined action of flood and wind. The analytical model assumed is a typical residential wooden building located in US in Flood and hurricane prone area. Models are categorized in to two groups to study the difference in performance of buildings in 100-year and 500-year flood plain. Group I comprise of a two-story LFWS residential building located in 500-year flood plain, assumed to be located in Houston, Texas for wind loading purpose. Group II include two buildings, Single and Two-Story (LFWS) residential building located in 100-year flood plain in Houston, Texas.

4.2 Design Basis Report – LFWS Slab-On-Grade type on 500-year flood plain (Group I)

4.2.1 Foundation

A building located in 500-year plain is assumed to be Slab-on-grade. Based on prevalent situation, most of the buildings, either it be created before NFIP regulations or after NFIP regulation are not elevated on 500-year flood plain. The buildings located in 500 -year plan is not required to be elevated because the ground level in 500-year flood plain is above the 100-year Base Flood Elevation (BFE). Complying the requirement as provided in Technical guide by Home of Texas (1995), the adopted size of grade beam below both internal and external walls are 12 inches by 24 inches, 12 inches and 24 inches being the width and depth of beam respectively (Figure 4.1 and Figure 4.2). Similarly, the adopted depth of grade slab is 4 inches (Figure 4.1 and Figure 4.2). The minimum thickness of concrete floor slabs directly on the ground shall not be less than 3.5 inches (IBC, 2015). Haunch between the slab and beam is generally equal to the depth of the slab, thus haunch with depth and width equal to 4 inches is adopted. All the calculation of dead load is based on the assumption that foundation is made up of normal weight concrete, which has a density of 150 pound/cubic ft (lb/ft^3) (IBC, 2015).

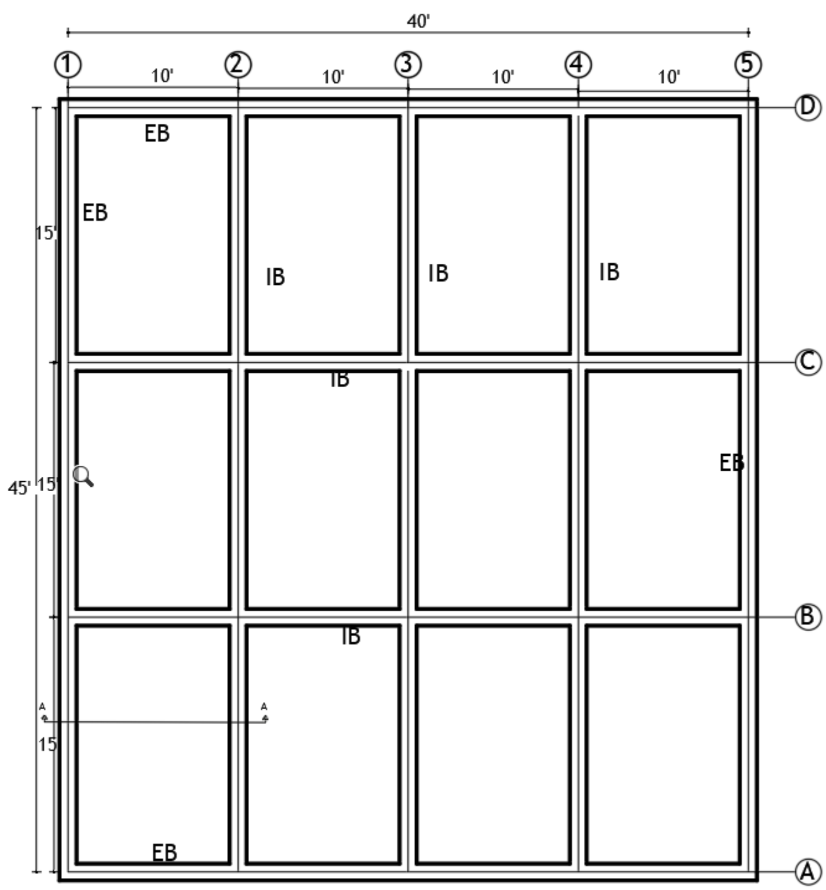


Figure 4.1 Foundation layout detail

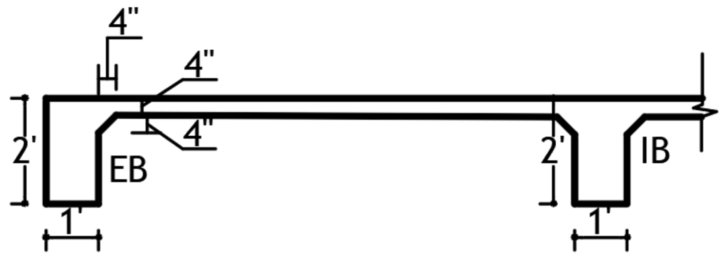


Figure 4.2 Section A-A of Foundation layout detail

4.2.2 Light-Frame Wooden Structure (LFWS)

Typically, studs are spaced 16 to 24 inches on center but building code requires maximum spacing be based on maximum nominal design wind speed, V_{asd} (IBC, 2015). The height of each floor is assumed to be 10 ft. As per IBC (2015), the maximum floor-to-floor height shall not exceed 11 ft and 7 inches, and the height of exterior bearing wall and interior braced wall shall not exceed a stud height of 10 ft. For a building located in Houston (Exposure category B, Risk category II), when 6d common nail is used with1.5 inches penetration, as per IBC (2015):

V_{ult}	= 150 mph	
$V_{asd} = V_{ult}\,(0.6)^{0.5}$	= 116.2 mph	Equation 16-33 (IBC, 2015)
Maximum wall stud spacing	= 16 inches	Table 2304.6.1 (IBC, 2015)
Minimum nominal panel thickness	= 7/16 inches	Table 2304.6.1 (IBC, 2015)
Minimum wood structural panel	= 24/16	Table 2304.6.1 (IBC, 2015)

Thus, for external walls, maximum wall stud spacing in analytical model is 16 inches and minimum nominal panel thickness is 7/16 inches. However, for internal walls, the stud spacing, and nominal panel thickness is based on the lateral and vertical loads.

Maximum span for Structural I Sheathing to be used in roof is 24 inches (Table 2304.8(5) IBC, 2015). However, thickness of sheathing depends on maximum live load and total load conditions. For a roof of residential building,

Live load (L)	= 20 psf	(ASCE 7-10)
Dead load (except self-weight)	= 10 psf	(ASCE 7-10)
Thickness of sheathing	= 7/16 inch	Table 2304.8(5), (IBC, 2015)

4.2.3 Frame geometry and material properties:

The assumed LFWS model is a two-story building with total height of 28.33 ft, each story being 10 ft and the slope of roof is assumed to be 5:12. The RISA model of the geometry for LFWS Slab-on-grade type building is shown in Figure 4.3. The windows and doors are assumed to be 5 ft x 4.5 ft and 4 ft x 7.5 ft, respectively. Risa floor 9.0 is used to model all the lateral and vertical load bearing members. All the gravity loads are applied to the model in Risa Floor 9.0 and members only bearing gravity loads are analyzed and designed. The model is then exported to Risa 3D 13.0 for applying lateral loads: flood and wind loads. After application of lateral loads members are analyzed for all load cases and load combinations as obtained from IBC 2015 and designed accordingly.

All internal and external walls are designed for both lateral and vertical loads. 2 x 6 stud with minimum spacing 16 inches and maximum spacing 24 inches along with 2 numbers of 2 x 6 top plate, 2 x 6 sill plate and 6 x 8 header, all made of Douglas Fir material is modeled as Wood wall. Schedule for Wood wall fasteners is obtained from IBC2012 Panel database and wood structural panels are double sided with minimum panel thickness of 0.375 inch and maximum panel thickness of 0.75 inch. Two number of chords each 2 x 6 is considered and hold downs are considered from SIMPSON catalog. Similarly, maximum and minimum nail spacing is 6 inches and 2 inches respectively.

Floor system of first floor (Figure A.1) consist of 1-inch wood deck, which is supported by APA I joist designed to carry only gravity loads. APA I joists are supported by Glulam Southern Pine (24F-1.8 DF Balanced) beam, which is designed to carry both gravity and lateral loads. Similarly, roof sheathing is supported by Southern Pine Glulam (24F-1.8E DF balanced) rafters which are supported on walls, so the wall at center of building is extended to all the way top of the building.

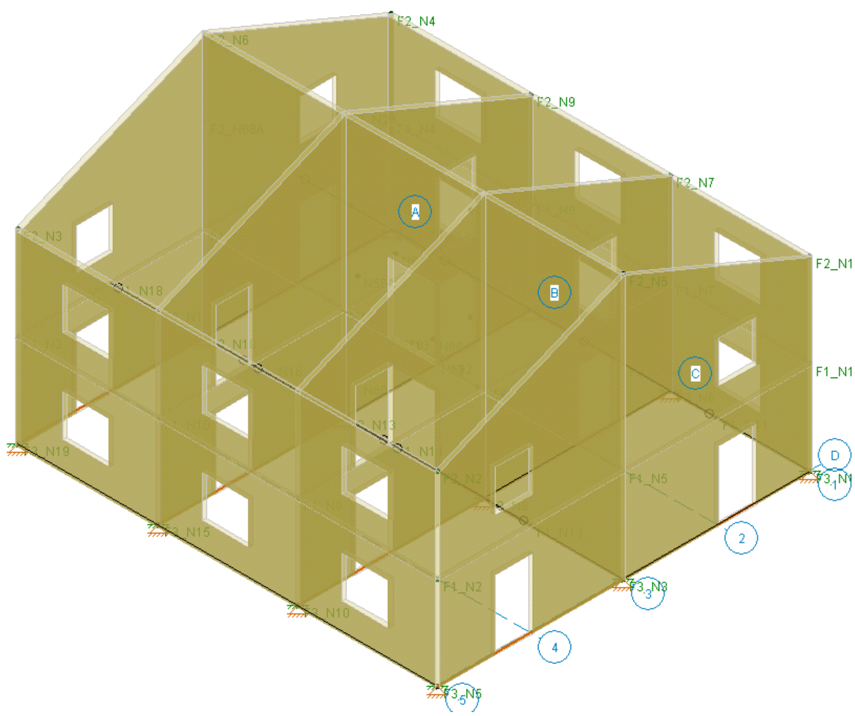


Figure 4.3 Three-dimensional view of two story building (obtained from Risa 3D)

4.2.4 Load definition and iterations

Both gravity and lateral loads are defined according to the provisions of IBC (2015) and are applied accordingly in the building models. The gravity loads being considered in building models are included in Appendix A.5. Similarly, the lateral flood load and wind loads being considered in models are computed in Appendix A.3 and Appendix A.4, respectively, based on the assumption made in Appendix A.2. To understand the behavior of LFWS Slab-on-grade type building in 500-year flood plain, different models are created with different flood loading parameters, keeping wind and gravity loads constant. 12 different models with different stillwater depth, ranging from 0 ft to “11 ft” are prepared to study the impact of variation in stillwater depth, assuming flood velocity, debris weight and water properties to be constant. The scouring depth is assumed to be 2 ft so the stillwater elevation is measured from the bottom of the foundation, which is 2 ft below the ground surface (Figure 4.2). Again, to understand the behavior of building when subjected to different stillwater elevation in internal and external face of building, each of the 12 models prepared earlier are modeled assuming different level difference in internal and external face, ranging from 0 to 4 ft, when possible to attain the difference. For instance, design stillwater depth of 4 ft cannot achieve more than 2 ft level difference because stillwater depth is measured 2 ft below the floor level. stillwater elevation is assumed to be higher in external face than in internal side. It is assumed that the building and the foundation slab is reasonably symmetrical and uniform. Therefore, it is assumed the center of gravity for the dead loads is at center grid.

4.3 Design Basis Report- LFWS elevated type on 100-year flood plain (Group 2)

4.3.1 Foundation

A building located in 100-year flood plain is assumed to be elevated on timber pile foundation. It is not recommended to build a building in A zone with wall or solid type of foundation because such type of foundation will be subjected to higher hydrodynamic force, breaking wave load and wave slam forces as compared to open foundations like moment frame, piles or pier type of foundation. NFIP allows buildings to be elevated and supported by fill in A-zones, however, the fill must be protected against erosion. It is possible to protect the foundation against erosion, but it is rather economic to construct timber pile foundation.

FEMA (2010) requires round pile to have minimum 8 inches diameter and square pile to have a minimum cross section of 8 inches by 8 inches. Also, more than 50% of pile cross-section cannot be reduced for placing girder. The most commonly available timber pile sizes are 12-inch tip diameter with length in between 60 to 65 ft (Table 1). In both 1 story and 2 story models 12-inch diameter piles are assumed. Piles are pin- connected at the top on first floor level, which is 5 ft above the ground level, by glulam girders at-least 6-

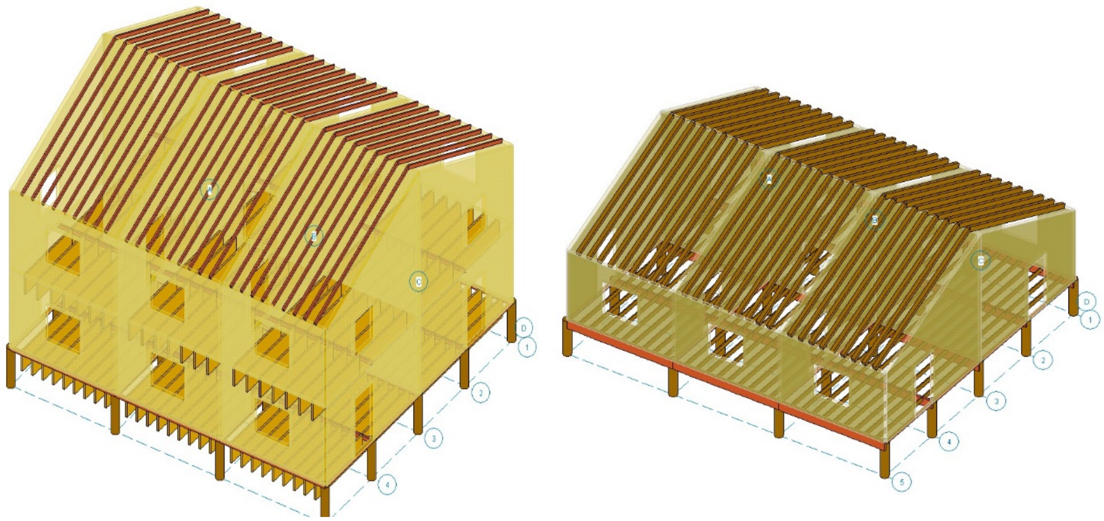


Figure 4.4 Three-dimensional view of two story and one-story building (obtained from Risa Floor)

inch wide and no grade beams are modeled (Figure 4.4).

4.3.2 Superstructure

The RISA model geometry for one and two-story building is shown in Figure 4.4. The modeling of super structure for both 1-story and 2-story is done as in Group I (Chapter 4.2.2 and Chapter 4.2.3). Unlike, use of anchor rods for Slab-on-Grade type of foundation, the sill plates are connected with the glulam beam by means of nails, screws or bolts. Full diameter 8d to 20d short nails are commonly specified for specific hurricane/seismic connection hardware.

4.3.3 Load definition and iterations

Both gravity and lateral loads are defined according to the provisions of IBC (2015) and are applied accordingly in the analytical model. The gravity loads being considered in analytical model are included in Appendix B.5. Similarly, the lateral flood load and wind loads being considered in models are computed in Appendix B.3 and Appendix B.4, respectively, based on the assumption made in Appendix B.2. To understand the behavior of LFWS elevated type building in 100-year flood plain, different models are created with different flood loading parameters, keeping wind and gravity loads constant. Five different models with different stillwater depth equal to 3 ft, 5 ft, 7 ft, 9 ft, and 11 ft are prepared to study the impact of variation in stillwater depth, assuming flood velocity, debris weight, water properties and level difference in internal and external face (equal to 3 ft, when possible) to be constant.

Chapter 5 Analysis and Results

5.1 Introduction

Several building models as defined in Chapter 4 are modeled in Risa Floor 9.0 for gravity loads obtained according to IBC (2015). After optimizing the required sizes of various components by designing and detailing for gravity load combinations, they were checked for serviceability requirement. Finalized models with optimized member dimensions is exported to RISA 3D 13.0 for lateral load analysis and design. Flood and wind loads are calculated based on the characteristics developed in Chapter 3. The lateral flood load and wind loads being considered in models are computed in Appendix A.3 and B.3 and Appendix A.4 and B.4, respectively, based on the assumption made in Appendix A.2 and B.2. The results of analytical models as mentioned above is presented in this chapter.

Building was analyzed to understand the performance when lateral wind and flood load were acting simultaneously. The performance of building was determined for different stillwater flood depth assuming different level difference in interior and exterior face of building wall. Both flood loads and wind loads are assumed to be acting in one direction at one time. Generally, the direction of hydrodynamic load can be estimated in coastal flooding because the storm surge flows from sea towards land. To better understand the behavior of building it is assumed the flood flow is in both direction, but one at a time.

Based on analysis, magnitude of effective dead load and width of building parallel to the lateral loads direction is found playing significant role to stabilize the overturning moments. If the Moment per linear ft is less than zero the building is safe against overturning. To improve the performance of Slab-on-grade type building against overturning failure, it is required to increase the dead load of structure or the orientation and dimension of building should be modified such that maximum counteracting moment arm can be achieved, when possible but when not possible, structural performance can be increased by allowing the interior of building to flooding such that there is no difference in flood water elevation in interior and exterior side. Similarly, for elevated type building, additional counteracting moment is also provided by the friction resistance of the timber piles against soil. Thus, the timber pile must be designed for overturning moments. Similarly, if the vertical uplift force is greater than zero for the most critical load combination, the resistance of foundation against soil can be considered. The friction resistance of timber pile acting downward helps building against uplifting, thus timber piles need to be designed from uplifting perspective too.

Similarly, if the lateral sliding force per linear ft is less than zero the building is safe against sliding. Apart from lateral force obtained from the critical combination, the difference in active and passive soil pressure also affects the lateral stability. Assuming the ground level after scouring is equal to the bottom elevation of foundation, the building is unsafe once the resulting lateral sliding force exceeds zero. To improve the performance of Slab-on-grade type against sliding failure, it is required to increase the dead load of structure and the depth of foundation on rear side can be increased so that the resisting passive soil pressure increases. Maintaining the flood water elevation on interior and exterior side also prevents building against sliding failure. Unlike slab-on-grade type building, elevated structures with piles properly embedded into soil do not slide but should be checked against the sway of floor and should not surpass the serviceability requirement. The diameter of timber piles must be designed for the base shear at the point of fixity and ground surface.

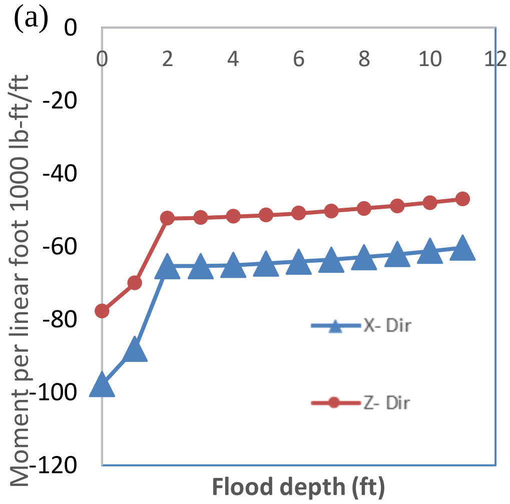
5.2 Performance of building along direction parallel and perpendicular to ridge

Both Slab-on-grade type and elevated buildings (1-story and 2-story) were analyzed against all global failure modes: overturning, sliding and uplift. The impact of lateral loads was found different when acting from different directions. In all cases, the dimension of building parallel to ridge is 45 ft and 40 ft when perpendicular to ridge.

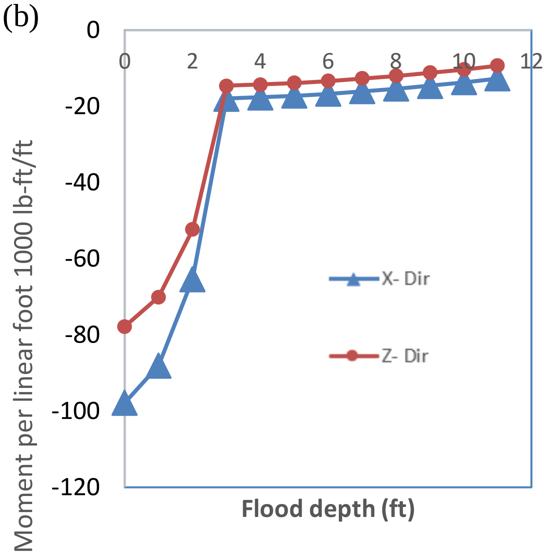
As shown in Figure 5.1, 5.2 and 5.3, Slab-on-Grade type building is subjected to lesser overturning moments and uplift forces per unit length when the lateral loads are acting parallel to the direction of ridge and stillwater depth is lesser than 3 ft and lesser sliding forces per unit length when stillwater depth is lesser than 2.5 ft for sliding, for all possible level differences in interior and exterior face of building. But when the building is subjected to lateral loads acting perpendicular to the direction of ridge with stillwater depth greater than 3 ft and difference in flood elevation in interior and exterior face is equal to or more than 2 ft (1 ft for sliding), it resulted in lesser overturning moments and uplift force per unit length compared to when lateral loads were acting parallel. Similarly, when the building was subjected to lateral loads acting perpendicular to the direction of ridge with stillwater depth greater than 3 ft and difference in flood elevation in interior and exterior face is equal to or more than 1 ft, it resulted in lesser sliding forces per unit length compared to when lateral loads were acting parallel.

However, if the level difference in flood water elevation in internal and external face is less than 2 ft (1 ft for sliding), the building is subjected to lesser forces per unit length for stillwater depth up to 11 ft when the lateral loads are acting perpendicular to smaller building width or parallel to the ridge.

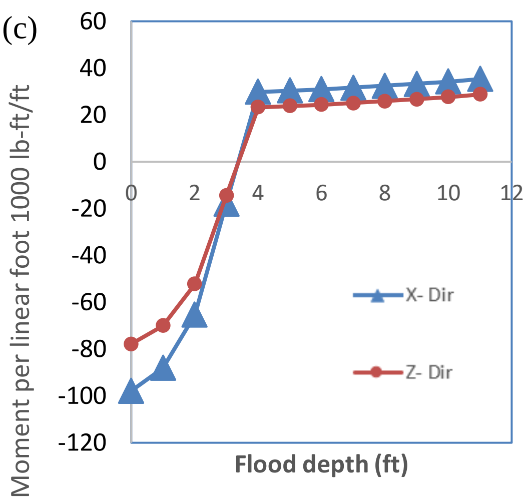
(a)



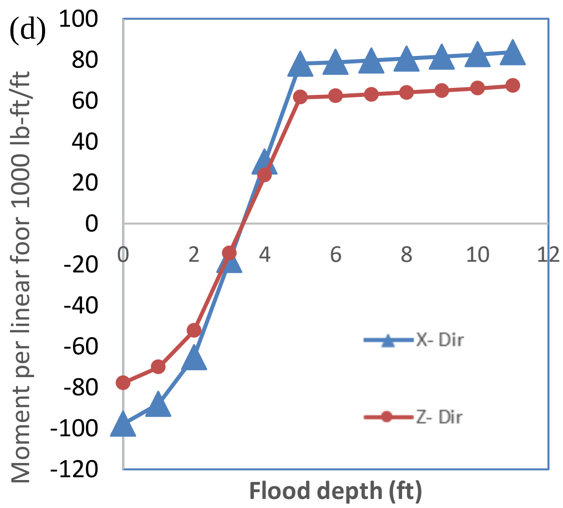
(b)



(c)



(d)



(e)

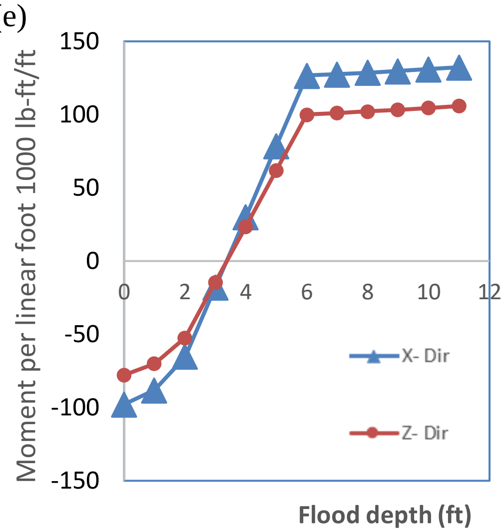
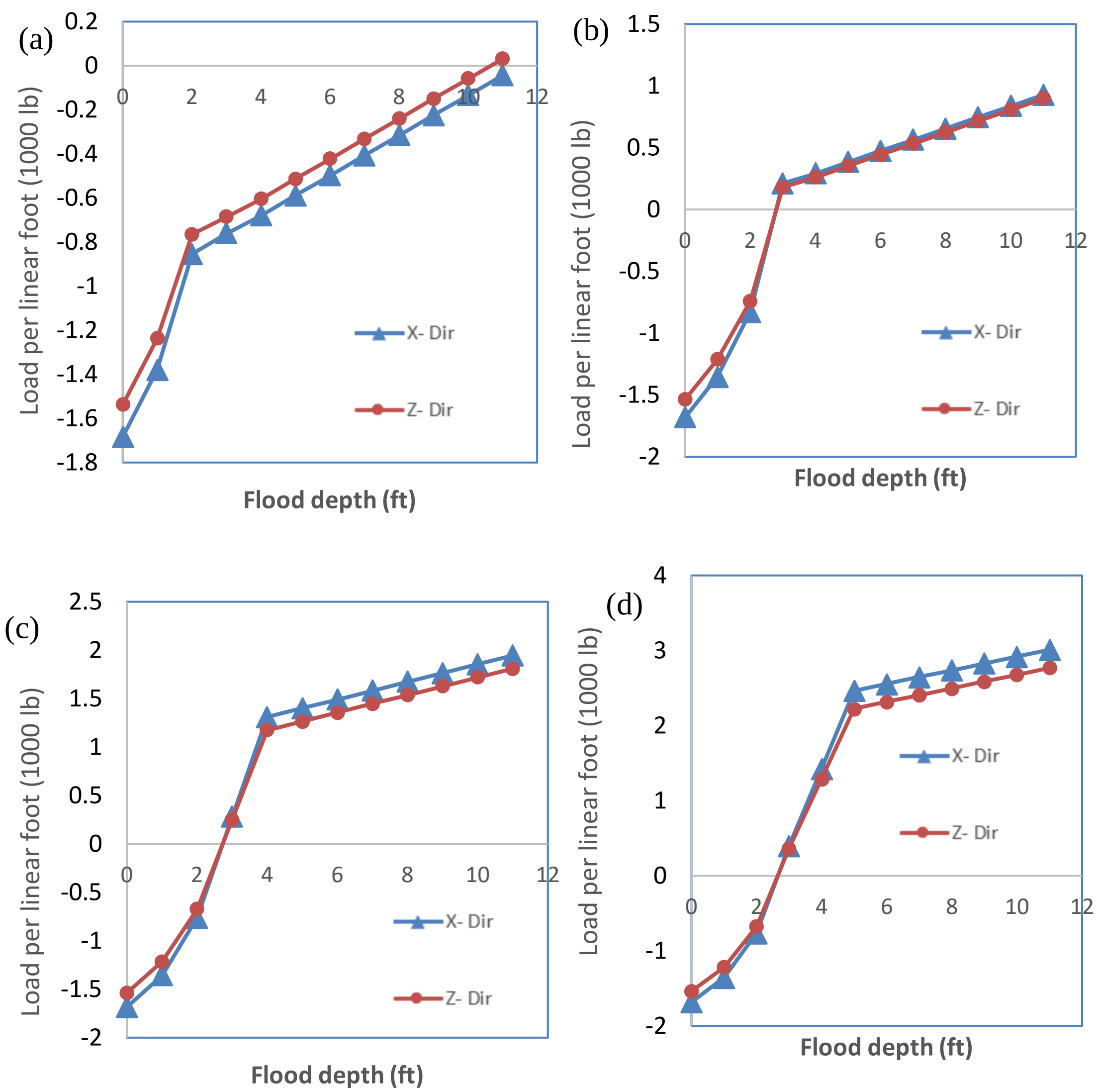


Figure 5.1 Overturning moment per unit length on Slab-On-Grade type building on 500-year flood plain due to combined action of flood and wind for different flood depth; X- dir is parallel to ridge and Z- dir is perpendicular to ridge; Flood elevation difference in interior and exterior face 0 ft (a), 1 ft (b), 2 ft (c), 3 ft (d), 4 ft (e)



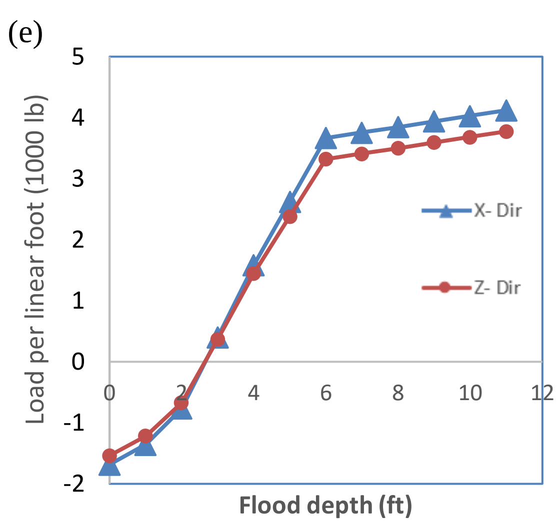
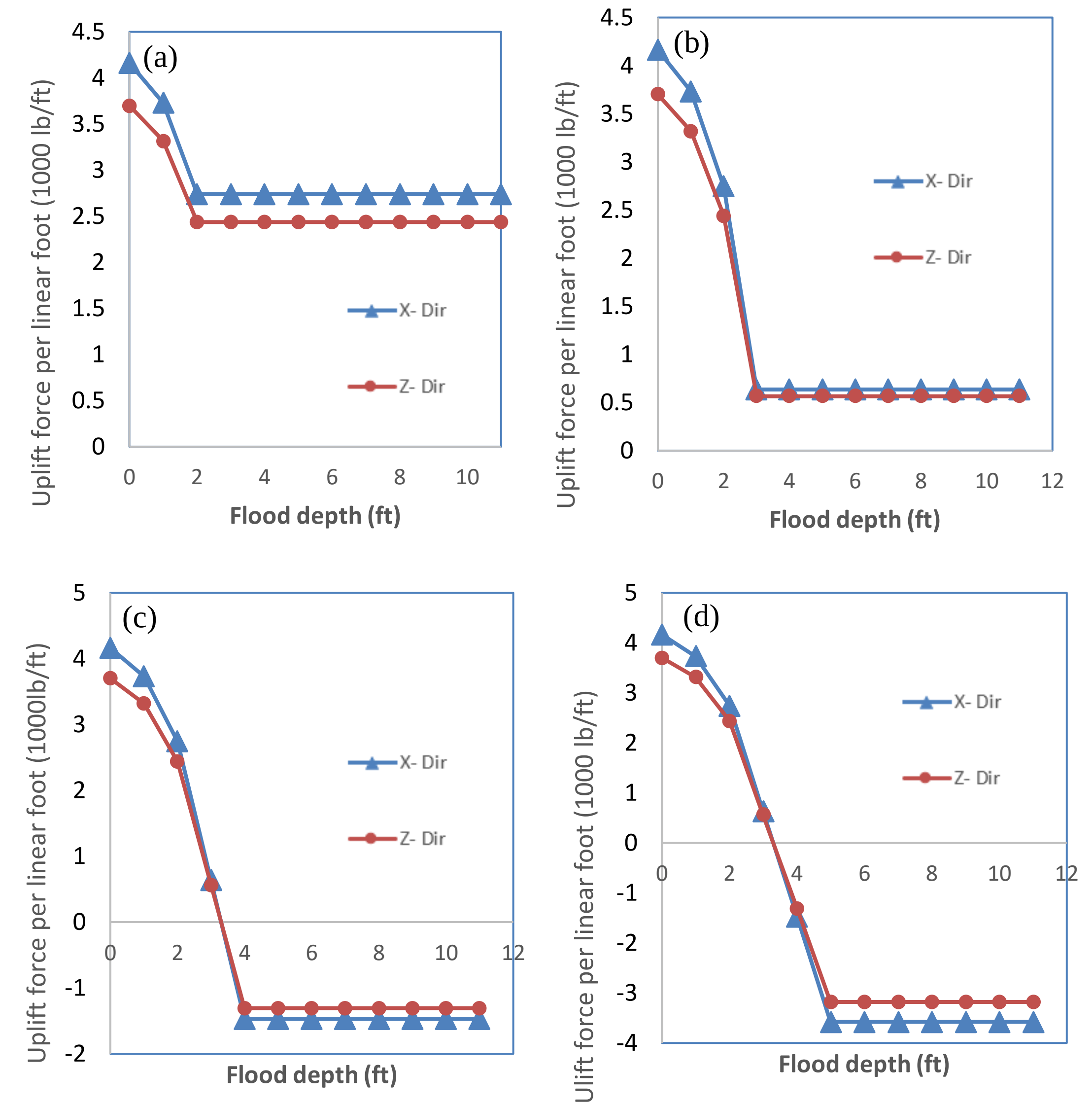


Figure 5.2 Lateral sliding force per unit length on Slab-On-Grade type building on 500-year flood plain due to combined action of flood and wind for different flood depth; X- dir is parallel to ridge and Z- dir is perpendicular to ridge; Flood elevation difference in interior and exterior face 0 ft (a), 1 ft (b), 2 ft (c), 3 ft (d), 4 ft (e)



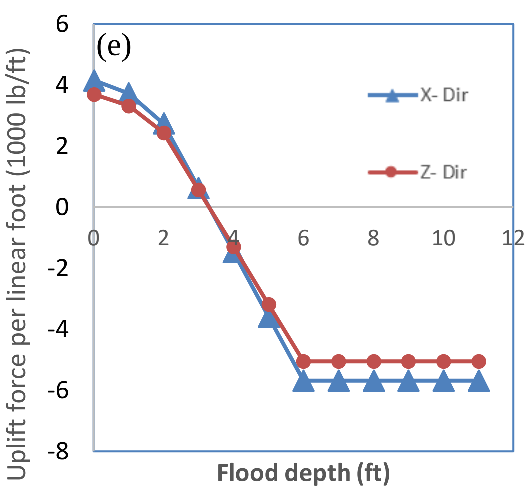


Figure 5.3 Vertical uplift force per unit length on Slab-On-Grade type building on 500-year flood plain due to combined action of flood and wind for different flood depth; X- dir is parallel to ridge and Z- dir is perpendicular to ridge; Flood elevation difference in interior and exterior face 0 ft (a), 1 ft (b), 2 ft (c), 3 ft (d), 4 ft (e)

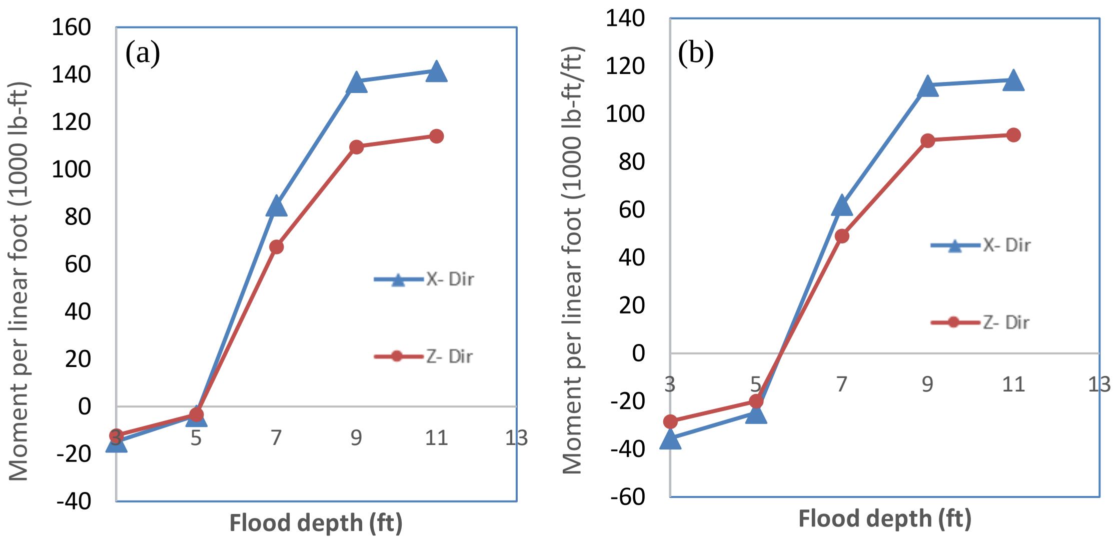


Figure 5.4 Overturning moment per unit length on elevated type building (a) one-story (b) two story on 100-year flood plain due to combined action of flood and wind for different flood depth; X- dir is parallel to ridge and Z- dir is perpendicular to ridge; Flood elevation difference in interior and exterior face is 3 ft

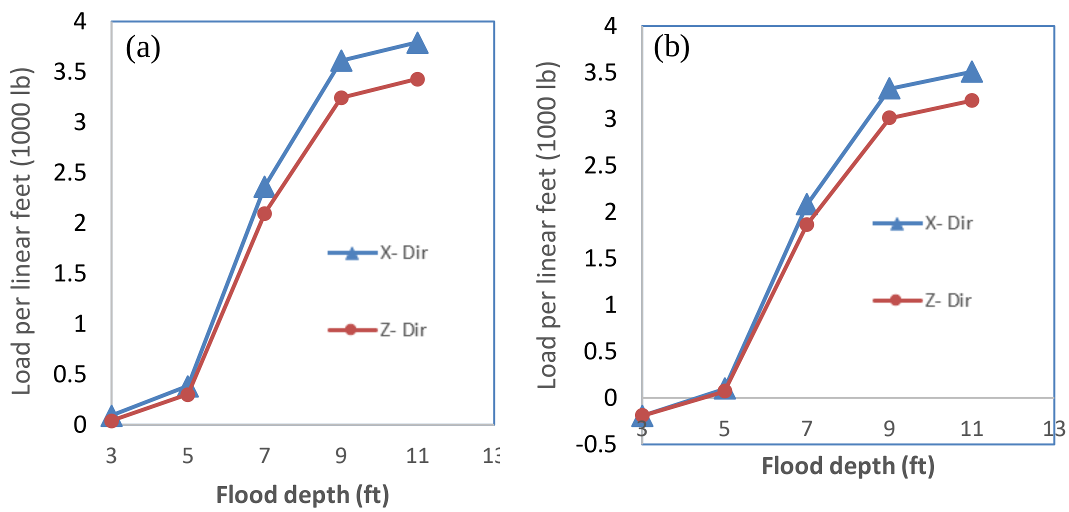


Figure 5.5 Lateral sliding force per unit length on elevated type building (a) one-story (b) two story on 100-year flood plain due to combined action of flood and wind for different flood depth; X- dir is parallel to ridge and Z- dir is perpendicular to ridge; Flood elevation difference in interior and exterior face is 3 ft

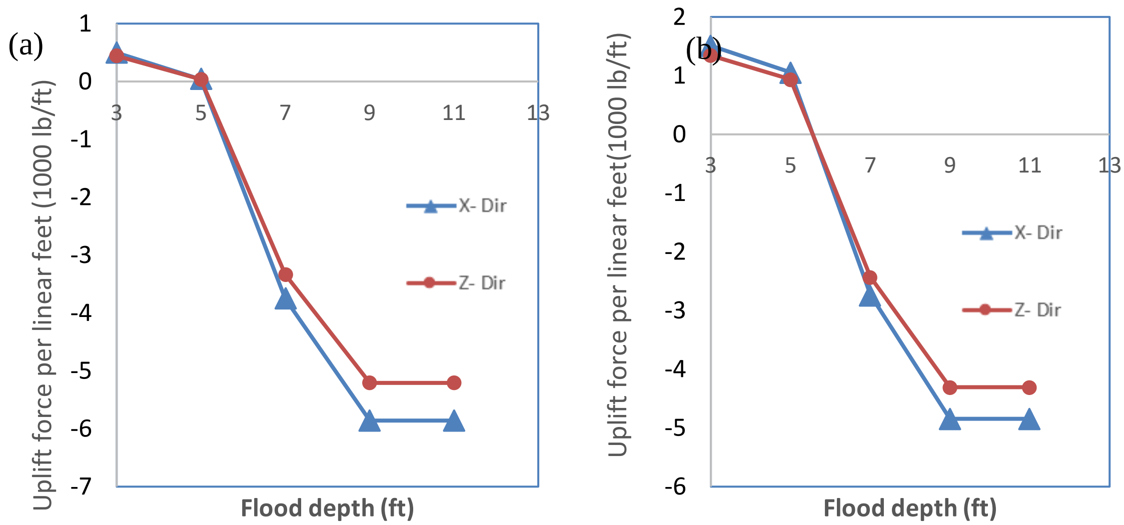


Figure 5.4 Vertical uplift force per unit length on elevated type building (a) one-story (b) two story on 100-year flood plain due to combined action of flood and wind for different flood depth; X- dir is parallel to ridge and Z- dir is perpendicular to ridge; Flood elevation difference in interior and exterior face is 3 ft

Results determined for elevated building types (Figure 5.4, 5.5 and 5.6) are similar to results obtained for the Slab-On-Grade type. For flood flow depth lesser than 5 ft, building is subjected to lesser lateral forces and moments per unit length, when lateral loads are acting perpendicular to smaller building width or parallel to the ridge. However, for flood flow depth more than 5 ft, building is subjected to lesser forces and moments per unit length when lateral loads are acting parallel to smaller building dimension.

5.3 Comparison of stability of Slab-on-grade vs elevated structure

Based on results, as shown in Figure 5.1 through Figure 5.6, if the Slab-on-grade type and elevated type building are subjected to same intensity of flood applied in same direction, it is observed elevated building is subjected to lesser overturning moment, lateral sliding force and vertical uplift forces per unit length until the flood depth reaches 7 ft for two stories building. However, in case flood depth is more than 7 ft for two stories, elevated building is found subjected to more overturning moment, lateral sliding force and vertical uplift forces per unit length compared to the Slab-on-grade type. Adding one story above single-story building reduces the overturning moment per unit length by 27% when the flood depth is 7 ft, by 19% when the flood depth is 9 ft and 20% when the flood depth is 11 ft because of counteracting stabilizing moment due to additional self-weight. However, it should be noted that addition of story also increases the overturning moment due to wind load.

5.4 Comparison of effect of lateral loads on buildings with and without effect of buoyancy

To understand the necessity of including the impact of buoyancy in flood load combination, all the building cases developed in Chapter 4 are analyzed without considering the impact of buoyancy force. Overturning moments, lateral sliding forces and vertical uplift forces per unit length along direction perpendicular to ridge on both Slab-on-grade and elevated type building is obtained for most critical load combination as defined in Chapter 2.6. As seen in Figure 5.7 (a), when Slab-on-grade type building is subjected to flood with stillwater depth ranging from 0 to 11 ft and for level difference in exterior and interior face lesser than 1 ft moment per linear ft is negative, which implies the counteracting moment due to effective self-weight is greater than overturning moment. Once the elevation difference raises to 2 ft, building is found unsafe for combined action of flooding with 4 ft stillwater elevation and wind conditions as described earlier. For buildings in Non-SFHA areas, there is no requirement of flood loads in the combination, so the impact of buoyancy is not considered while assessing the global structural stability.

As shown in Figure 5.7 (b), building is found safer against overturning due to combined action of high wind loads and lateral component of flood load excluding the buoyancy effect, for as deep as 11 ft stillwater depth flood. Because in this case effective self-weight of building is not reduced due to upward acting buoyancy force. The results are found similar even for elevated one-story and two-story building as shown in Figure 5.10.

Figure 5.8 is obtained by plotting lateral sliding force per ft due to wind and different stillwater depth flood acting in the direction perpendicular to ridge. Assuming the coefficient of friction of soil to be 0.45 and no lateral resistance in the form of passive soil pressure due to scouring and erosion during flood, the frictional resisting force due to effective self-weight of building must resist the lateral force. If the load per linear ft is less than 0, it implies that the effective self-weight of building can resist the lateral loads. As shown in Figure 5.8(a), building is able to withstand lateral loads only if there is no level difference in exterior and interior face. Difference in level, approximately equal to 1 ft when stillwater depth is as less as 3 ft can result in sliding failure. An elevated 2 story building is found performing well among other types, wherein, building is safer against sliding if floodwater depth is lesser than 5 ft.

Similarly, the performance of both types of buildings against uplift failure is, as shown in figure 5.9 and 5.12, is similar to overturning case. All types of buildings are found safe against uplift if buoyancy force is not considered in the design. But when uplift force is considered, if the difference in flood elevation is more than 1 ft in exterior and interior face, slab-on-grade type building was found unsafe for all depth of flow greater than 3 ft.

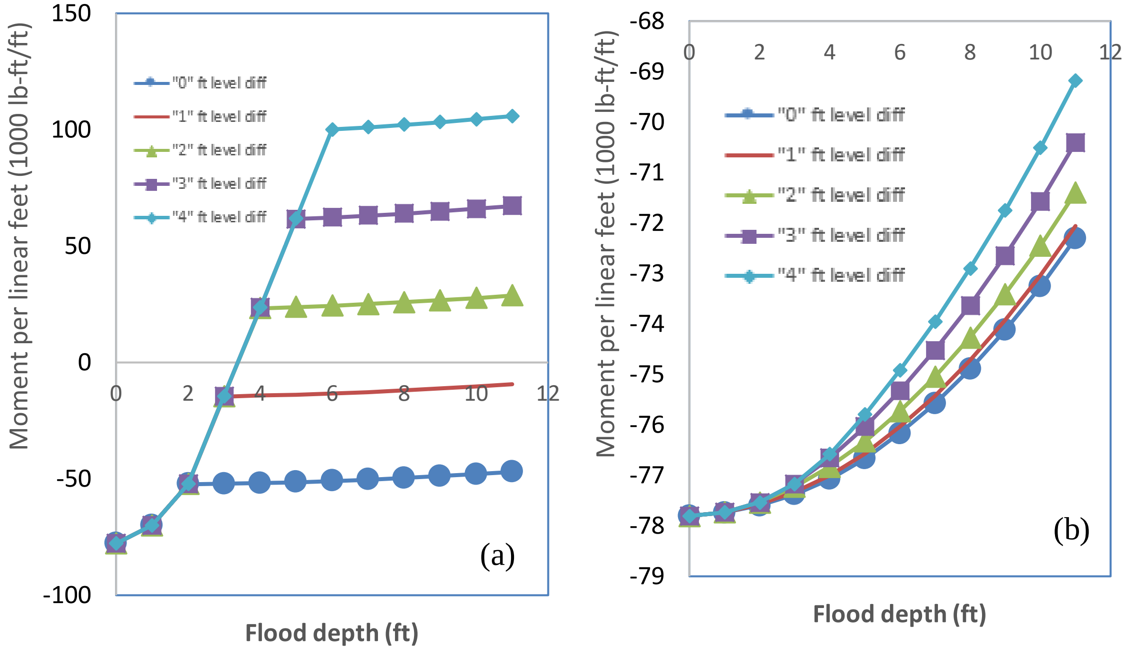


Figure 5.7 Overturning moment per unit length on Slab-On-Grade type building on 500-year flood plain due to com bined action of flood and wind for different flood depth for different stillwater elevation difference in interior and exterior face (a) considering buoyancy (b) without considering buoyancy

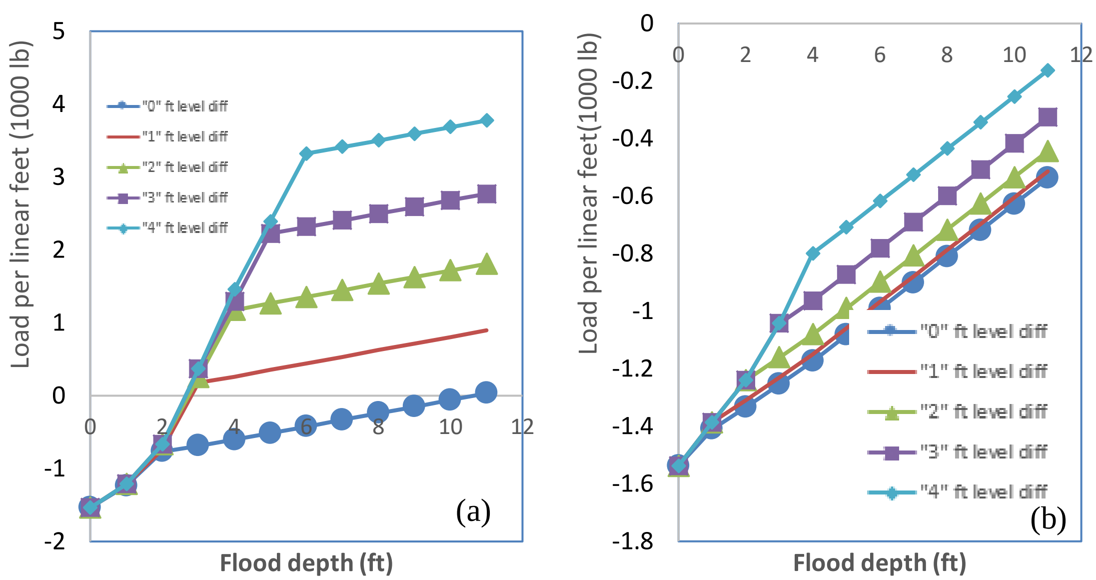


Figure 5.8 Lateral sliding force per unit length on Slab-On-Grade type building on 500-year flood plain due to combined action of flood and wind for different flood depth for different stillwater elevation difference in interior and exterior face (a)considering buoyancy (b) without considering buoyancy

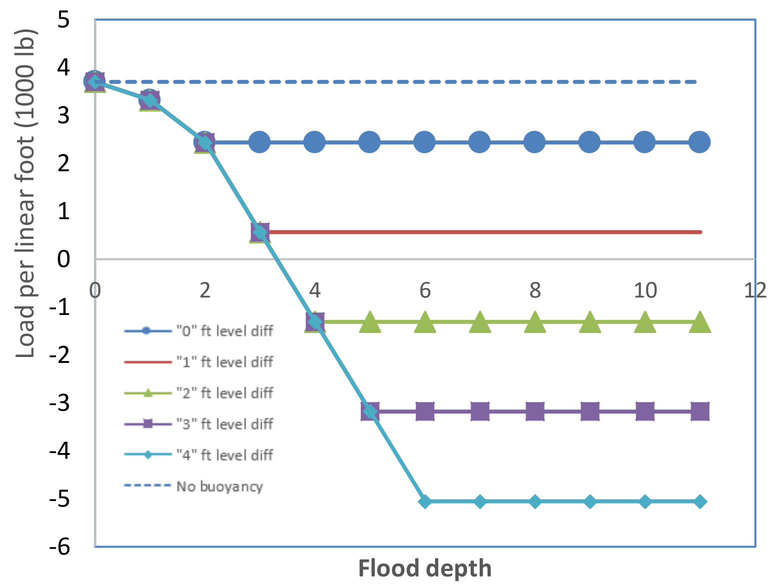


Figure 5.9 Vertical uplift force per unit length on Slab-On-Grade type building on 500-year flood plain due to combined action of flood and wind for different flood depth for different stillwater elevation difference in interior and exterior face

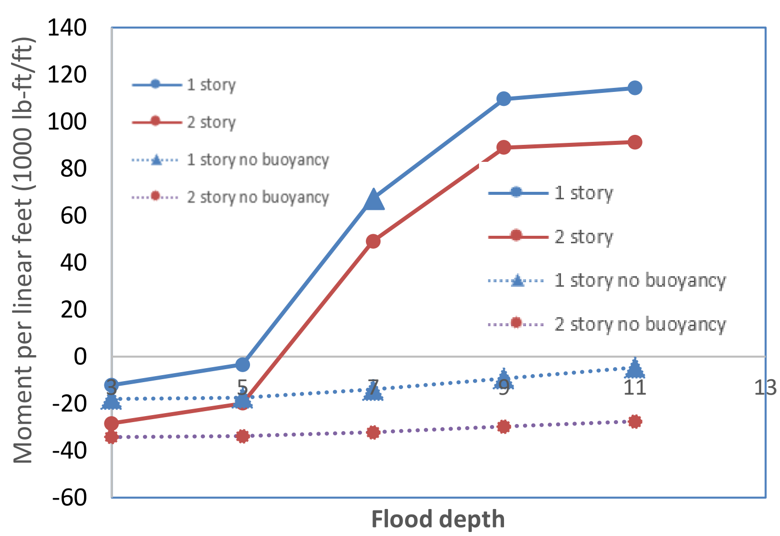


Figure 5.10 Overturning moment per unit length on elevated type one and two story building on 100-year flood plain due to combined action of flood and wind for different flood depth with and without buoyancy force

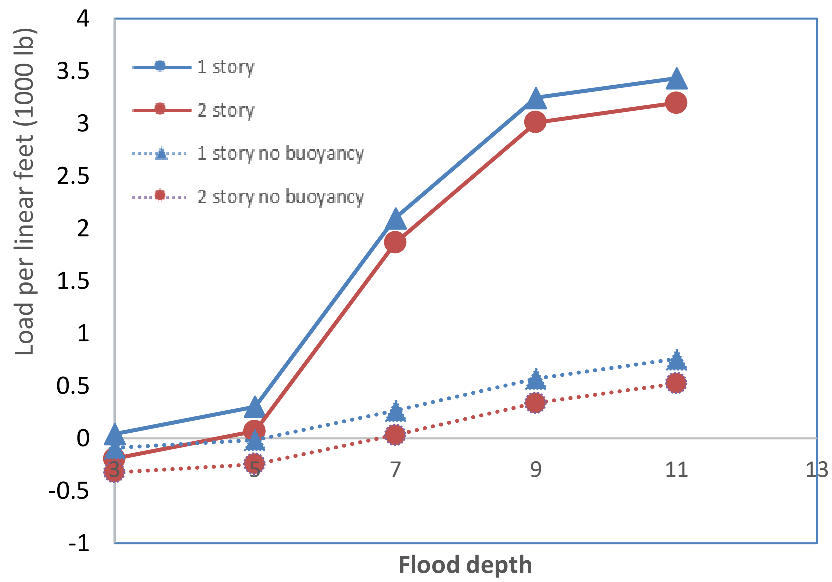


Figure 5.11 Lateral sliding force per unit length on elevated type one and two story building on 100-year flood plain due to combined action of flood and wind for different flood depth with and without buoyancy force

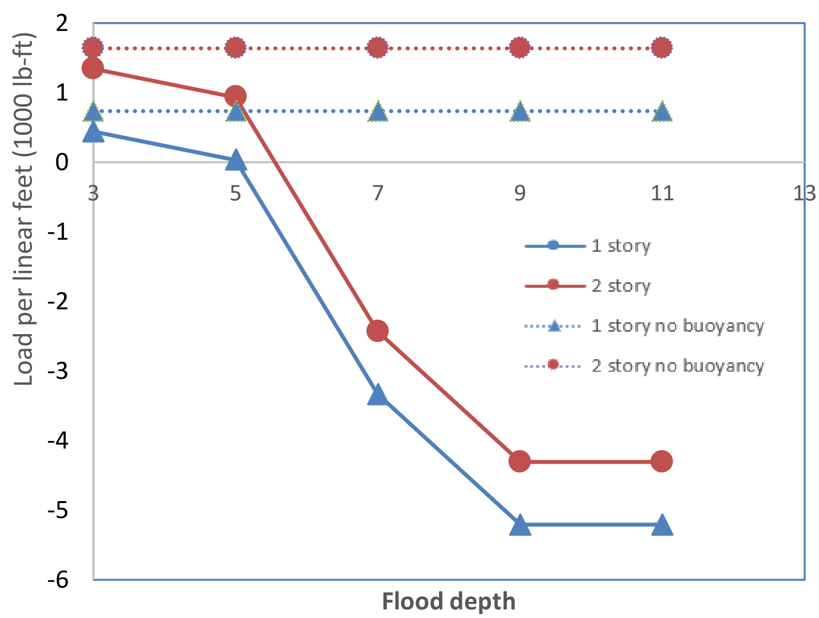


Figure 5.12 Vertical uplift force per unit length on elevated type one and two story building on 100-year flood plain due to combined action of flood and wind for different flood depth with and without buoyancy force

5.5 Analysis and design of Floor to foundation shear connection

Shear can be transferred from super structure to the foundation of a building by connecting sill plate to concrete foundation using series of anchor bolts. The bolts are added in addition to the tie-down bolts used for the chord forces. 2 X 6 sill plate made of Douglas Fir Larch (Specific gravity, G) and anchor bolt $\frac{1}{2}$ inch diameter is considered for design. Requirement of anchor bolts for lateral shear connection for Slab-on-grade type building subjected to various intensity of flooding and wind is studied.

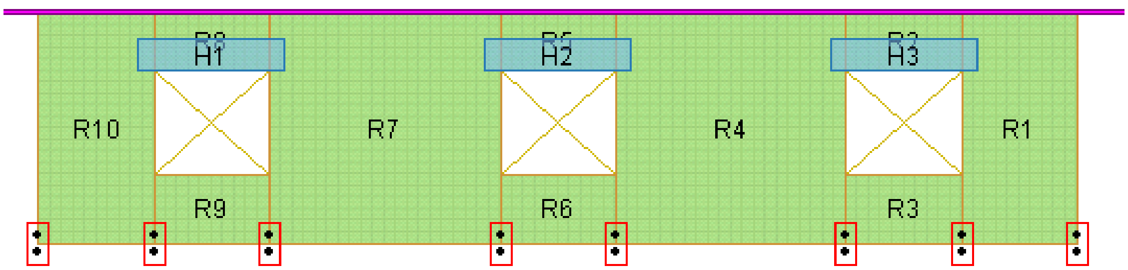


Figure 5.5 Elevation of external wall along grid 5 parallel to the ridge; R1=R10= 5 ft and R4=R7= 10 ft

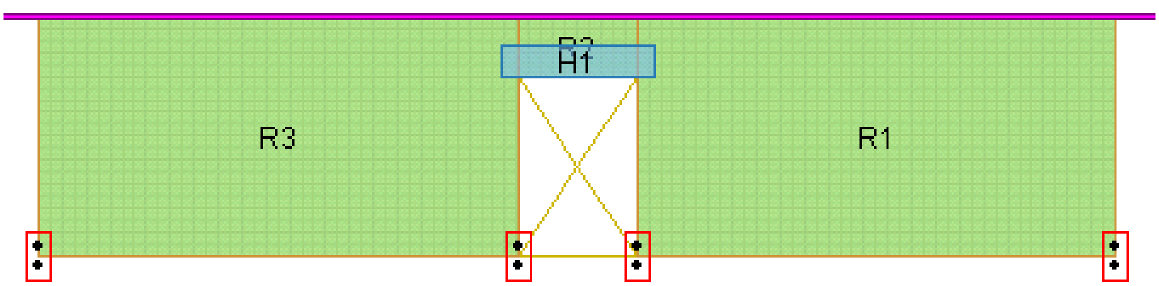


Figure 5.6 Elevation of internal wall along grid 3 parallel to the ridge; R1=R3=20 ft

One exterior and one interior wall, parallel to direction of ridge is considered for study. Wall along grid 5 is the exterior wall and is divided into 10 regions, R1 through R10. Complete regions (i.e. R1, R4, R7 and R10) are designed for resisting lateral loads. Similarly, interior wall along grid 3 is divided into 3 regions, R1, R2 and R3, wherein R1 and R3 are designed to resist lateral loads. The selection of wall is based on the most critical condition, determined on analyzing the structure on RISA 3D for lateral loads.

According to Breyer (2014), The anchor bolt requirement can be determined by first assuming a size of anchor bolt and determining the adjusted value per bolt and determining the adjusted design value per bolt. The adjusted design value per anchor bolt Z' (parallel-to-grain design value is determined from Table 12E of NDS (2015) and the total lateral force parallel to the shearwall segment is the unit shear times the length of wall segment (v x b). The required number of bolt and average spacing is

$$Number\ of\ bolt, N = \frac{v\ x\ b}{Z'} \tag{52}$$

$$Spacing\ of\ bolt, S = \frac{b}{N} \tag{53}$$

The number of bolts required also must satisfy the anchorage requirement for lateral loads perpendicular to wall. The number of bolts for perpendicular loads can be determined as

$$Number\ of\ bolt, N = \frac{R\ x\ b}{Z'} \tag{54}$$

Where R is the reaction per ft on the base of floor due to lateral loads.

The code minimum anchor bolt requirement for wood frame walls is given in IBC Sec 2308.6 as 1/2 -in diameter anchor bolts at 6 ft on center. Additionally, a properly sized nut and washer or steel plate washer must be used on each anchor bolt. A minimum of two bolts or anchor straps are required per wall plate and one bolt or strap is required within 12 inches of, and at least 4 inches from, the end of each plate piece. However, according to Bureau of Recovery and Mitigation (2017), in areas where wind speed is greater than 120 mph, slab on grade construction requires straps embedded in the concrete and nailed to the bottom plate or wall studs, or 5/8-inch diameter anchor bolts spaced at 18-inches or less and require 3-inch by 3-inch by 1/8-inch thick washers between nuts and the bottom plate if anchor bolts are used. The ends of the walls should have large anchors called hold downs to resist the uplift due to lateral loads on tension end.

A slab-on-grade type building when subjected to different flooding event with stillwater elevation ranging from 0 to 5 ft and maximum achievable flood elevation difference of 3 ft in internal and external face (external elevation is assumed greater). For consistency, all walls are designed using Shear panel label S1_7/16_8d@d and double-sided panel for all interior and exterior beam. For different conditions, shear per unit length is determined from RISA3D. Assuming, 5/8" of anchor bolts are used as means of shear connection in between wall plate and foundation, spacing is determined for in-plane shear, which is checked for spacing requirement of out of plane shear. Also, internal walls are not subjected to the hydrodynamic and hydrostatic laterals loads and wind pressure. Adjusted lateral design value (Z') is obtained assuming the moisture content is less than 19% during fabrication and more than 19% while in service, the temperature of the building site is more than 100^0 F and other relevant factors as per building and structural considerations.

Figure 5.15 compares the required spacing of anchor bolts for flood conditions against the minimum anchor bolt diameter and spacing requirement established by International Code Council (2015) and Bureau of Recovery and Mitigation (2017). In figure 5.15 (b) and (d), the minimum requirement is according to IBC (2015), which requires minimum ½" anchor bolt at spacing not more than 72 inches. Similarly, in figure 5.15 (a) and (c), the minimum requirement is according to Bureau of Recovery and Mitigation

(2017), which requires 5/8- inch diameter anchor bolts spaced at 18 inches or less in areas where the 3-second gust design wind speed is greater than 120 miles per hour (mph). In all cases, either it be internal (Figure 5.15 d) or external wall Figure 5.15 b), the required spacing of anchor bolt determined for combined lateral wind and flood load is way lesser than the maximum set by IBC (2015). Non-engineered building that adopted the minimum requirement set by IBC without engineering calculations are subject to local shear failure in between wall and foundation.

For a building constructed following minimum recommendation established by Bureau of Recovery and Mitigation (2017), shear connection in between internal wall (Figure 5.15 c) and foundation is safe for flood depth lesser than 4 ft with level difference not more than 3 ft in exterior and interior face. Similarly, shear connection in between external wall and foundation (Figure 5.15 a) is safe for flood depth lesser than 2 ft with level difference not more than 1 ft in exterior and interior face of building.

Comparing results as obtained in Figure 5.15 vs Figure 5.1, 5.2 and 5.3, if a Slab-on-grade type building is constructed making use of only minimum requirement set by IBC (i.e. with ½ inch anchor bolts at 72 inches on center), building will fail locally in wall to foundation shear connection before any form of global failure. Similarly, if a Slab-on-grade type building constructed complying just the minimum recommendation established by Bureau of Recovery and Mitigation (2017) (i.e. 5/8-inch anchor bolt spaced at 18 inches on center) will also fail locally in wall to foundation shear connection before it fails globally.

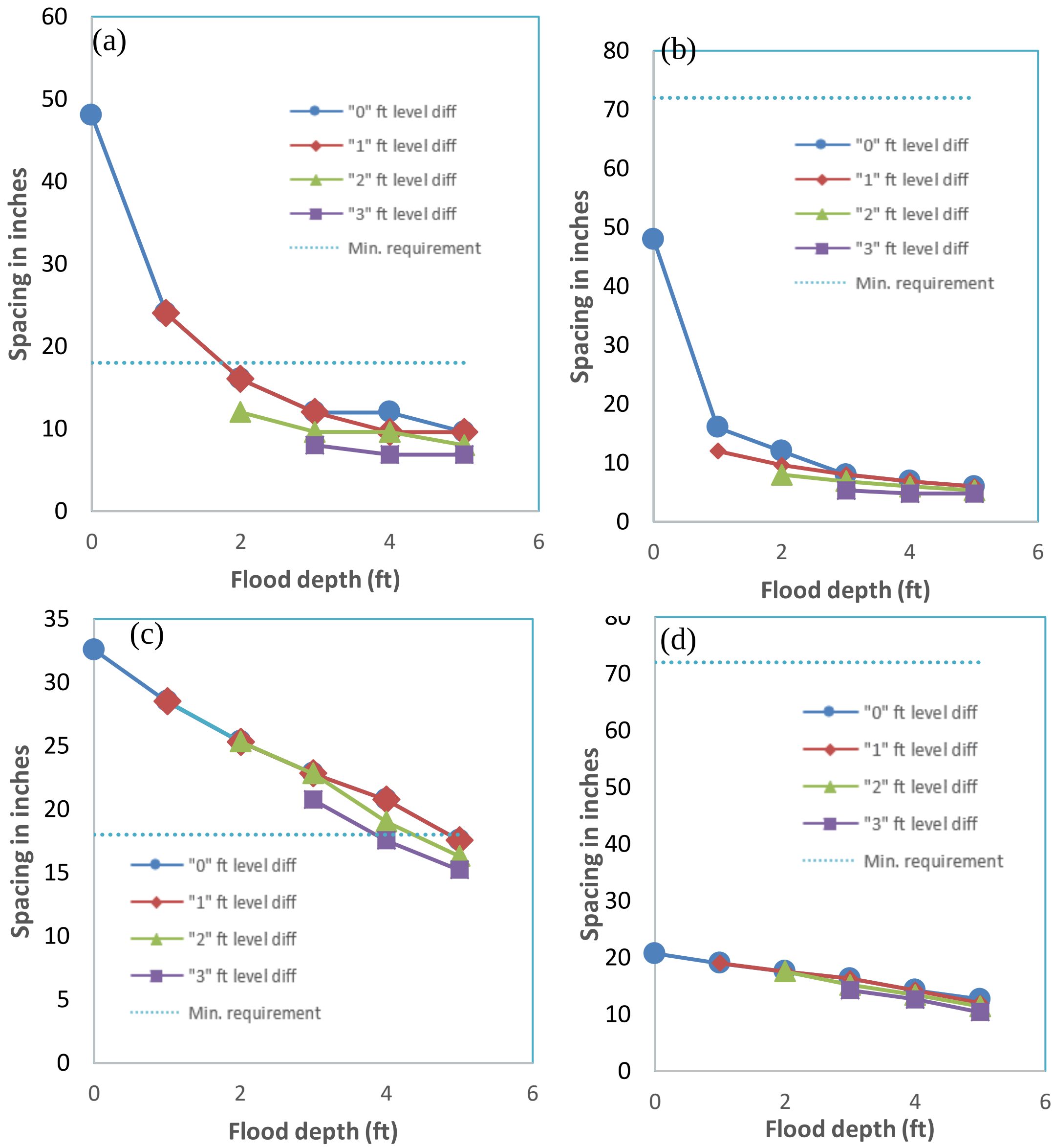


Figure 5.7 Comparison of spacing of anchor bolts with minimum requirement as defined by IBC (2015) and Bureau of Recovery and Mitigation (2017) when subjected to combined lateral flood and wind load (a) and (b) R1 of external wall along Grid 5 (Figure 5.13); c) and d) R1 of internal wall along Grid 3 (Figure 5.14); Flood depth considered from top of the slab

Chapter 6 Conclusion and future work

6.1 Research Conclusions

This thesis documents a comprehensive investigation and analysis of low rise residential Light Weight Frame Structure (LFWS) building, both elevated and non-elevated type, built on Slab-on-Grade and timber pile foundation, respectively. It also describes various characteristics of flood and high wind hazards and the prevalent load calculation methodologies by incorporating flood loading parameter in regular design combinations. The performance of several types of low rise residential LFWS subjected to different intensities of flood and high wind has been studied. Based on comprehensive investigation undertaken to understand the performance of several types of buildings for different flooding cases acting simultaneously with High wind hazards as described in Chapter 5 following conclusions are made:

a) Slab-on-grade type buildings constructed complying only the minimum requirement set by IBC (2015) (i.e. with ½ inch anchor bolts at 72 inches on center), is subject to local wall to foundation shear connection failure before any other form of global failure happens when subjected to combined flood and wind hazard.

b) Slab-on-grade type buildings constructed complying just the minimum recommendation established by Bureau of Recovery and Mitigation (2017) (i.e. with 5/8-inch anchor bolts at 18 inches on center), is subject to local wall to foundation shear connection failure before any other form of global failure happens when subjected to combined flood and wind hazard.

c) Elevating the building helps reducing non-structural damage for sure. It also improves the global stability against the combined lateral action of flood and wind, as compared to Slab-on-grade type building when floodwater depth is lesser than elevated height, so it is suggested to elevate the building above possible floodwater depth.

d) The slab-on-grade building is found safe against global failure mode for flooding of up to 11 ft depth flowing at 10 fps when floodwater is allowed to be intruded inside building such that there is no internal and external water level difference.

e) Two-story elevated building is found safer against overturning, sliding and uplifting global failure modes compared to One-story elevated building with same floor plan and purpose.

f) Results and design methodology presented in this thesis research can be used by practicing engineers to better understand the combined action of flood and wind on a LFWS and other structures and apply it for conventional and retrofit design. It can also be used in academics for further research on flood and wind hazards.

6.3 Limitations of this Study

Several limitations and assumptions were made for this study which are listed below:

- Only a simple, rectangular, gable roof building was used for investigations without overhangs. Additional uplift wind forces applied in overhang portion must be calculated for roofs with overhangs, this will impact adversely in the calculation of overturning moment about the assumed pivot.
- No reduction in wind load has been done when the building is flooded as high as 11 ft. The submerged portion of building are not subjected to wind pressures whilst inside water.

- Both flood loads and wind loads are assumed to be acting in one direction, the performance of building when flood and wind loads are acting at right angle can be different.
- The velocity of flood is assumed to be 10 ft per second (fps) for entire research formulation in all building types and all depth of flood. Velocities can be higher for smaller depth flow and lesser for high depth flow.

6.4 Recommendations for Future Work

- The most optimum aspect ratio (B/H) of building parallel and perpendicular to the direction of lateral wind and flood load can be achieved by analyzing wide range of buildings with different dimensions.
- The experimental study of time variant impact of buoyancy in a building for different soil types can be done for proper establishment of conclusion made analytically.
- For buildings with crawlspace, wall to floor shear connection can be analyzed against local failure.
- Performance of buildings for different velocities can be determined analytically and verified experimentally.

Appendix A

Description and load calculations- Group I

A.1 Structural framing plans

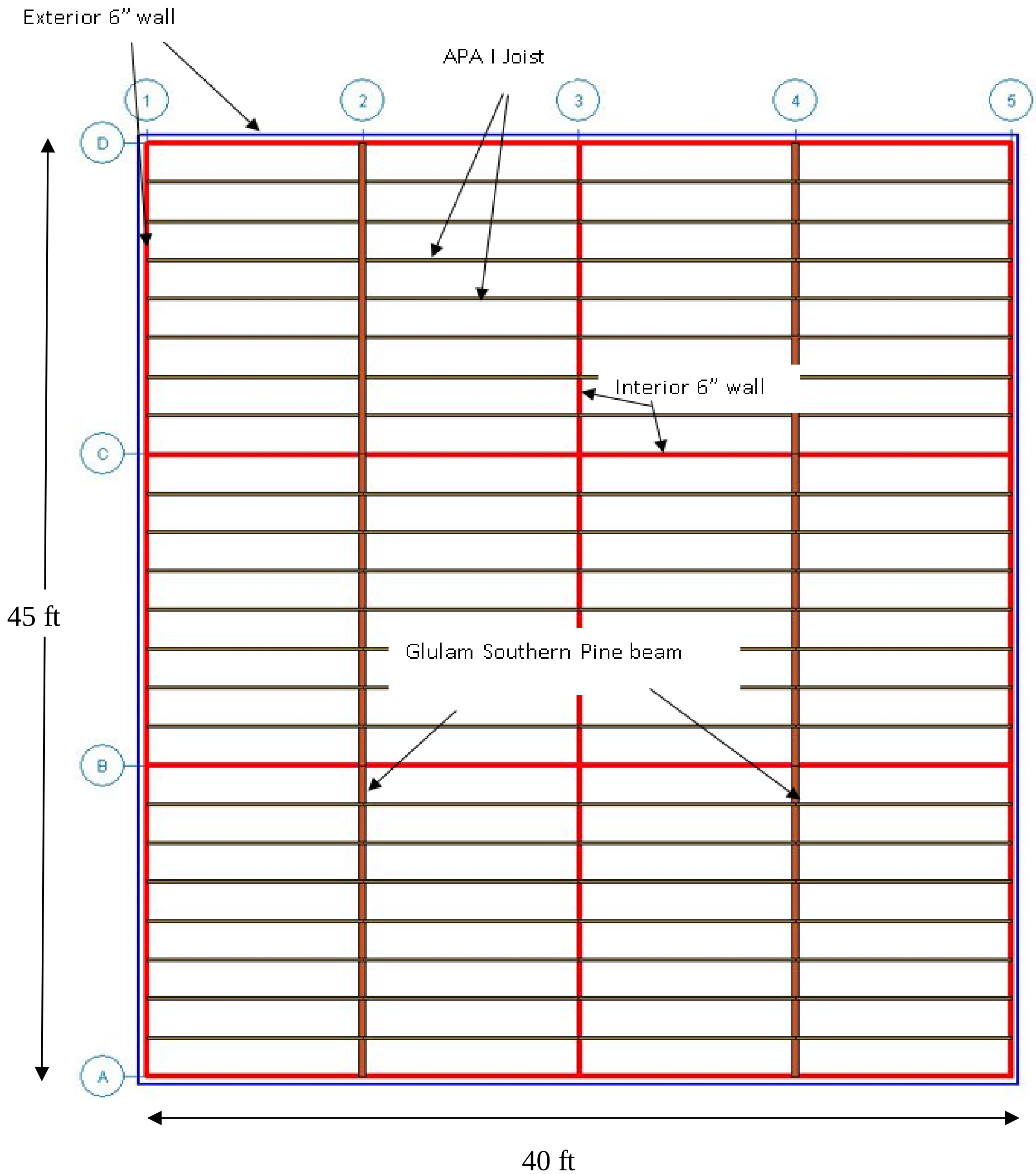


Figure A.0.1 Structural framing plan of First floor

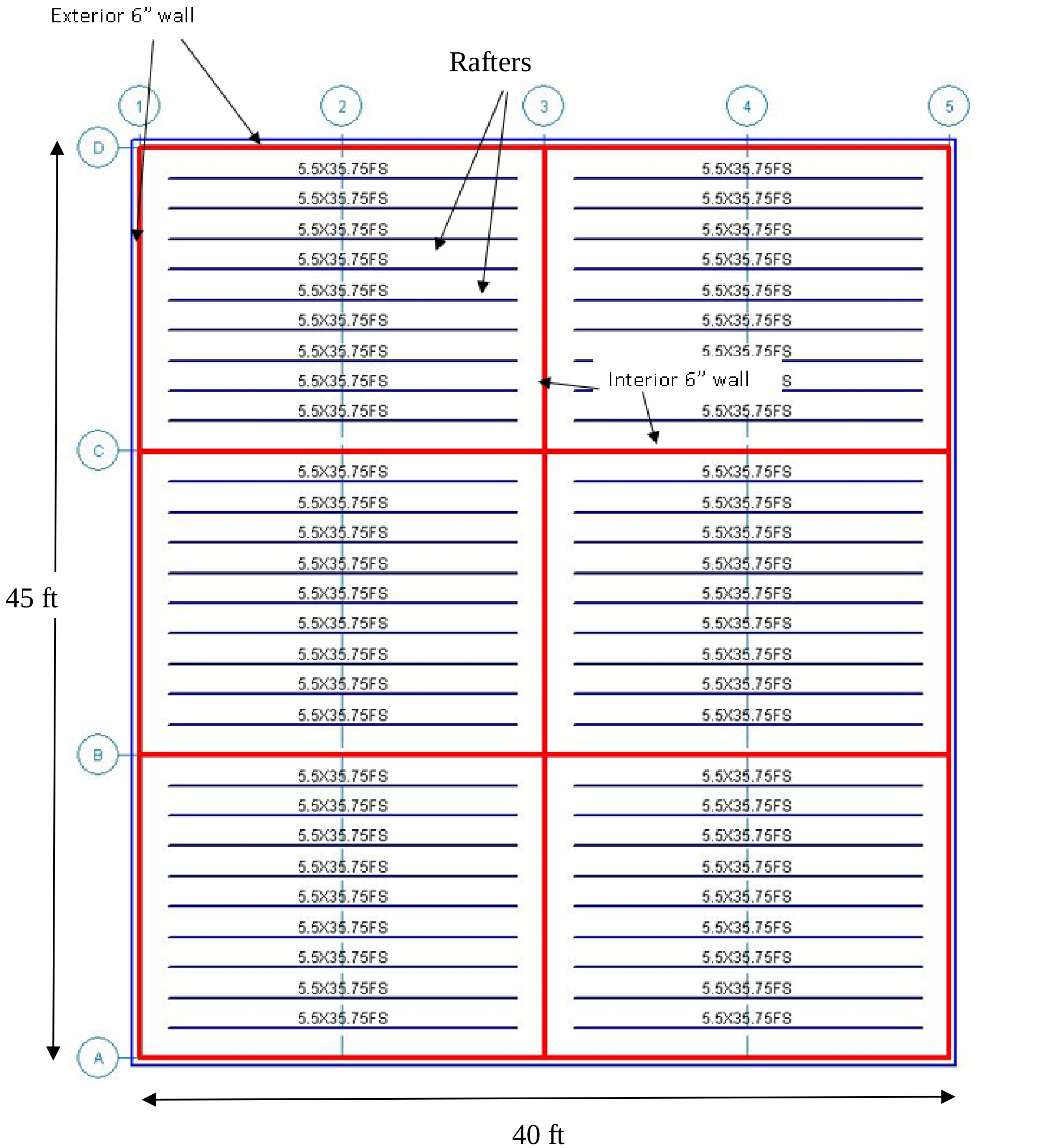


Figure A.0.2 Structural framing plan of roof

A.2 Assumptions

A.2.1 When no flood loads are applied- Case 1

The load calculation for this modeling condition depends on various assumptions listed below.
a) The building is located on flood Zone B in non-Coastal Area and the load and it's combination as per Allowable Stress Design is obtained as per IBC 2015/ ASCE 7/10

b) Dead, Live, Roof live, Wind and Flood loads are obtained from IBC, 2015

1. D (IBC 16.8)
2. D + L (IBC 16.9)
3. D + Lr (IBC 16.10)
4. D + 0.75L +0.75 Lr (IBC 16.11)
5. D + 0.6W (IBC 16.12)
6. D + 0.75L + 0.75 (0.6W) + 0.75Lr (IBC16.13)
7. 0.6D + 0.6W (IBC16.15)

c) Assumptions for Wind loading:

1. Risk Category: II
2. Basic Wind Speed (V): 150 mph (Assuming, Houston, Texas) (Table 26.5-1A ASCE 7/10)
3. Importance factor, I=1 (Category II building: 1 (ASCE 7/10))
4. Velocity pressure exposure coefficient evaluated at height z (K_z) (Table 27.3-1 ASCE 7/10)
5. Topographical Factor (K_{zt})= 1 (assuming a flat surface) (Table 26.8-1 ASCE 7/10)
6. Wind Directionality Factor (K_d)= 0.85 (Table 26.6-1 ASCE 7/10)
7. Exposure category= B (26.7.3 ASCE 7/10)

A.2.2 When flood loads are applied assuming the building is located in Flood Zone A:

Case 2 (stillwater Flood depth ranging from 1 ft to 11 ft)

The load calculation for this modeling condition depends on various assumptions listed below.
a) The building is located on flood Zone A in non-Coastal Area and the load and it's combination as per Allowable Stress Design is obtained as per IBC 2015/ ASCE 7/10

b) Dead, Live, Roof live, Wind and Flood loads are obtained from IBC, 2015

1. D (IBC 16.8)
2. D + L (IBC 16.9)
3. D + Lr (IBC 16.10)
4. D + 0.75L +0.75 Lr (IBC 16.11)
5. D + 0.6W + 1.5Fa (IBC 16.12)

6. D + 0.75L + 0.75 (0.6W) + 0.75Lr + 1.5Fa (IBC16.13)

7. 0.6D + 0.6W + 1.5Fa (IBC16.15)

c) Assumptions for Wind loading:

1. Risk Category: II

2. Basic Wind Speed (V): 150 mph (Assuming, Houston, Texas) (Table 26.5-1A ASCE 7/10)

3. Importance factor, I=1 (Category II building: 1 (ASCE 7/10))

4. Velocity pressure exposure coefficient evaluated at height z (K_z) (Table 27.3-1 ASCE 7/10)

5. Topographical Factor (K_{zt})= 1 (assuming a flat surface) (Table 26.8-1 ASCE 7/10)

6. Wind Directionality Factor (K_d)= 0.85 (Table 26.6-1 ASCE 7/10)

7. Exposure category= B (26.7.3 ASCE 7/10)

A.3 Flood load calculation

A.3.1 When stillwater Flood depth is 1 ft

			Level difference
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)			0
Source of Water	=	river	
Depth of flood, d_s (ft)	=	1	
Difference in water level (ft)		=	0
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)		=	0
F_{sta} (lb/ft)		=	0
b) Hydrostatic buoyancy (F_{buoy}):			
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)		=	0
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)		=	0
Uplift force on slab {F_{buoy} = *(Vol)* γ_w} (lb)		=	0
Add. force due to submerged foundation ,F_{buoy} (lb)		=	23025.6
Total ,F_{buoy} (lb)			23025.6
c) Hydrodynamic Load (F_{dyn})			
Velocity of flow (fps)	=	10	
Width in X Direction (ft)	=	40	
Drag coefficient (C_d) X direction	=	1.5	
Width in Z Direction	=	45	
Drag coefficient (C_d) Z direction	=	1.75	
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}			
Hydrodynamic Pressure (P_d) -X (psf)		=	145.5
Hydrodynamic Pressure (P_d) -Z (psf)		=	169.75
d) Wave loads (*f_{brkp}*):			
Dynamic pressure coefficient, C_p	=	2.8	
f_{brkp} *(lb per ft acting at stillwater elevation)*		=	310.752
e) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}			
Weight of the object (lb), W	=	1000	
Depth Coefficient, C_D	=	0	
Blockage Coefficient, C_B	=	1	
Building Str. Coefficient, C_{Str}	=	0.2	
F_i (lb)		=	0

A.3.2 When stillwater Flood depth is 2 ft

			Level difference
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)			0
Source of Water	=	river	
Depth of flood, d_s (ft)	=	2	
Difference in water level (ft)		=	0
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)		=	0
F_{sta} (lb/ft)		=	0
b) Hydrostatic buoyancy (F_{buoy}):			
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)		=	0
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)		=	0
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)		=	0
Add. force due to submerged foundation ,F_{buoy} (lb)		=	75816
Total ,F_{buoy} (lb)			75816
c) Hydrodynamic Load (F_{dyn})			
Velocity of flow (fps)	=	10	
Width in X Direction (ft)	=	40	
Drag coefficient (C_d) X direction	=	1.3	
Width in Z Direction	=	45	
Drag coefficient (C_d) Z direction	=	1.4	
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}			
Hydrodynamic Pressure (P_d) -X (psf)		=	126.1
Hydrodynamic Pressure (P_d) -Z (psf)		=	135.8
d) Wave loads (*f_{brkp}*):			
Dynamic pressure coefficient, C_p	=	2.8	
f_{brkp} *(lb per ft acting at stillwater elevation)*		=	1243.008
e) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$			
Weight of the object (lb), W	=	1000	
Depth Coefficient, C_D	=	0.25	
Blockage Coefficient, C_B	=	1	
Building Str. Coefficient, C_{Str}	=	0.2	
F_i (lb)		=	500

A.3.3 When stillwater Flood depth is 3 ft

		Level difference	
a) Hydrostatic Load (F_{sta}= $0.5P_hd_s$)		0	1
Source of Water	= river		
Depth of flood, d_s (ft)	= 3		
Difference in water level (ft)	=	0	1
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=	0	62.4
F_{sta} (lb/ft)	=	0	31.2
b) Hydrostatic buoyancy (F_{buoy}):			
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=	0	1
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=	0	62.4
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=	0	112320
Add. force due to submerged foundation ,F_{buoy} (lb)	=	75816	75816
Total ,F_{buoy} (lb)		75816	188136
c) Hydrodynamic Load (F_{dyn})			
Velocity of flow (fps)	= 10		
Width in X Direction (ft)	= 40		
Drag coefficient (C_d) X direction	= 1.3		
Width in Z Direction	= 45		
Drag coefficient (C_d) Z direction	= 1.3		
Hydrodynamic Pressure (P_d) {$= C_d\rho.V^2/2$}			
Hydrodynamic Pressure (P_d) -X (psf)	=	126.1	126.1
Hydrodynamic Pressure (P_d) -Z (psf)	=	126.1	126.1
d) Wave loads (f_{brkp}):			
Dynamic pressure coefficient, C_p	= 2.8		
f_{brkp} *(lb per ft acting at stillwater elevation)*	=	2796.768	2796.768
e) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}			
Weight of the object (lb), W	= 1000		
Depth Coefficient, C_D	= 0.5		
Blockage Coefficient, C_B	= 1		
Building Str. Coefficient, C_{Str}	= 0.2		
F_i (lb)	=	1000	1000

A.3.4 When stillwater Flood depth is 4 ft

		Level difference		
		0	1	2
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)				
Source of Water	= river			
Depth of flood, d_s (ft)	= 4			
Difference in water level (ft)	=	0	1	2
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=	0	62.4	124.8
F_{sta} (lb/ft)	=	0	31.2	124.8
b) Hydrostatic buoyancy (F_{buoy}):				
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=	0	1	2
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=	0	62.4	124.8
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=	0	112320	224640
Add. force due to submerged foundation ,F_{buoy} (lb)	=	75816	75816	75816
Total ,F_{buoy} (lb)		75816	188136	300456
c) Hydrodynamic Load (F_{dyn})				
Velocity of flow (fps)	= 10			
Width in X Direction (ft)	= 40			
Drag coefficient (C_d) X direction	= 1.25			
Width in Z Direction	= 45			
Drag coefficient (C_d) Z direction	= 1.25			
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}				
Hydrodynamic Pressure (P_d) -X (psf)	=	121.3	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=	121.3	121.25	121.25
d) Wave loads (f_{brkp}):				
Dynamic pressure coefficient, C_p	= 2.8			
f_{brkp} *(lb per ft acting at stillwater elevation)*	=	4972	4972	4972
e) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$				
Weight of the object (lb), W	= 1000			
Depth Coefficient, C_D	= 0.75			
Blockage Coefficient, C_B	= 1			
Building Str. Coefficient, C_{Str}	= 0.2			
F_i (lb)	=	1500	1500	1500

A.3.5 When stillwater Flood depth is 5 ft

			Level difference			
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)			0	1	2	3
Source of Water	=	river				
Depth of flood, d_s (ft)	=	5				
Difference in water level (ft)	=		0	1	2	3
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=		0	62.4	124.8	187.2
F_{sta} (lb/ft)	=		0	31.2	124.8	280.8
b) Hydrostatic buoyancy (F_{buoy}):						
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=		0	1	2	3
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=		0	62.4	124.8	187.2
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=		0	112320	224640	336960
Add. force due to submerged foundation ,F_{buoy} (lb)	=		75816	75816	75816	75816
Total ,F_{buoy} (lb)			75816	188136	300456	412776
c) Hydrodynamic Load (F_{dyn})						
Velocity of flow (fps)	=	10				
Width in X Direction (ft)	=	40				
Drag coefficient (C_d) X direction	=	1.25				
Width in Z Direction	=	45				
Drag coefficient (C_d) Z direction	=	1.25				
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}						
Hydrodynamic Pressure (P_d) -X (psf)	=		121.3	121.25	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=		121.3	121.25	121.25	121.25
d) Wave loads (*f_{brkp}*):						
Dynamic pressure coefficient, C_p	=	2.8				
f_{brkp} *(lb per ft acting at stillwater elevation)*	=		7769	7768.8	7768.8	7768.8
e) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}						
Weight of the object (lb), W	=	1000				
Depth Coefficient, C_D	=	1				
Blockage Coefficient, C_B	=	1				
Building Str. Coefficient, C_{Str}	=	0.2				
F_i (lb)	=		2000	2000	2000	2000

A.3.6 When stillwater Flood depth is 6 ft

			Level difference				
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)			0	1	2	3	4
Source of Water	=	river					
Depth of flood, d_s (ft)	=	6					
Difference in water level (ft)	=		0	1	2	3	4
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=		0	62.4	124.8	187.2	249.6
F_{sta} (lb/ft)	=		0	31.2	124.8	280.8	499.2
b) Hydrostatic buoyancy (F_{buoy}):							
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=		0	1	2	3	4
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=		0	62.4	124.8	187.2	249.6
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=		0	112320	224640	336960	449280
Add. force due to submerged foundation ,F_{buoy} (lb)	=		75816	75816	75816	75816	75816
Total ,F_{buoy} (lb)			75816	188136	300456	412776	525096
c) Hydrodynamic Load (F_{dyn})							
Velocity of flow (fps)	=	10					
Width in X Direction (ft)	=	40					
Drag coefficient (C_d) X direction	=	1.25					
Width in Z Direction	=	45					
Drag coefficient (C_d) Z direction	=	1.25					
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}							
Hydrodynamic Pressure (P_d) -X (psf)	=		121.3	121.25	121.25	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=		121.3	121.25	121.25	121.25	121.25
d) Wave loads (f_{brkp}):							
Dynamic pressure coefficient, C_p	=	2.8					
f_{brkp} *(lb per ft acting at stillwater elevation)*	=		11187	11187	11187	11187.1	11187
e) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$							
Weight of the object (lb), W	=	1000					
Depth Coefficient, C_D	=	1					
Blockage Coefficient, C_B	=	1					
Building Str. Coefficient, C_{Str}	=	0.2					
F_i (lb)	=		2000	2000	2000	2000	2000

A.3.7 When stillwater Flood depth is 7 ft

			Level difference				
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)			0	1	2	3	4
Source of Water	=	river					
Depth of flood, d_s (ft)	=	7					
Difference in water level (ft)	=		0	1	2	3	4
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=		0	62.4	124.8	187.2	249.6
F_{sta} (lb/ft)	=		0	31.2	124.8	280.8	499.2
b) Hydrostatic buoyancy (F_{buoy}):							
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=		0	1	2	3	4
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=		0	62.4	124.8	187.2	249.6
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=		0	112320	224640	336960	449280
Add. force due to submerged foundation ,F_{buoy} (lb)	=		75816	75816	75816	75816	75816
Total ,F_{buoy} (lb)			75816	188136	300456	412776	525096
c) Hydrodynamic Load (F_{dyn})							
Velocity of flow (fps)	=	10					
Width in X Direction (ft)	=	40					
Drag coefficient (C_d) X direction	=	1.25					
Width in Z Direction	=	45					
Drag coefficient (C_d) Z direction	=	1.25					
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}							
Hydrodynamic Pressure (P_d) -X (psf)	=		121.3	121.25	121.25	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=		121.3	121.25	121.25	121.25	121.25
d) Wave loads (f_{brkp}):							
Dynamic pressure coefficient, C_p	=	2.8					
f_{brkp} *(lb per ft acting at stillwater elevation)*	=		15227	15227	15227	15226.8	15227
e) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}							
Weight of the object (lb), W	=	1000					
Depth Coefficient, C_D	=	1					
Blockage Coefficient, C_B	=	1					
Building Str. Coefficient, C_{Str}	=	0.2					
F_i (lb)	=		2000	2000	2000	2000	2000

A.3.8 When stillwater Flood depth is 8 ft

			Level difference				
a) Hydrostatic Load (F_{sta} = $0.5P_h d_s$)			0	1	2	3	4
Source of Water	=	river					
Depth of flood, d_s (ft)	=	8					
Difference in water level (ft)	=		0	1	2	3	4
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=		0	62.4	124.8	187.2	249.6
F_{sta} (lb/ft)	=		0	31.2	124.8	280.8	499.2
b) Hydrostatic buoyancy (F_{buoy}):							
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=		0	1	2	3	4
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=		0	62.4	124.8	187.2	249.6
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=		0	112320	224640	336960	449280
Add. force due to submerged foundation ,F_{buoy} (lb)	=		75816	75816	75816	75816	75816
Total ,F_{buoy} (lb)			75816	188136	300456	412776	525096
c) Hydrodynamic Load (F_{dyn})							
Velocity of flow (fps)	=	10					
Width in X Direction (ft)	=	40					
Drag coefficient (C_d) X direction	=	1.25					
Width in Z Direction	=	45					
Drag coefficient (C_d) Z direction	=	1.25					
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}							
Hydrodynamic Pressure (P_d) -X (psf)	=		121.3	121.25	121.25	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=		121.3	121.25	121.25	121.25	121.25
d) Wave loads (f_{brkp}):							
Dynamic pressure coefficient, C_p	=	2.8					
f_{brkp} *(lb per ft acting at stillwater elevation)*	=		19888	19888	19888	19888.1	19888
e) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}							
Weight of the object (lb), W	=	1000					
Depth Coefficient, C_D	=	1					
Blockage Coefficient, C_B	=	1					
Building Str. Coefficient, C_{Str}	=	0.2					
F_i (lb)	=		2000	2000	2000	2000	2000

A.3.9 When stillwater Flood depth is 9 ft

			Level difference				
			0	1	2	3	4
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)							
Source of Water	=	river					
Depth of flood, d_s (ft)	=	9					
Difference in water level (ft)	=		0	1	2	3	4
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=		0	62.4	124.8	187.2	249.6
F_{sta} (lb/ft)	=		0	31.2	124.8	280.8	499.2
b) Hydrostatic buoyancy (F_{buoy}):							
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=		0	1	2	3	4
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=		0	62.4	124.8	187.2	249.6
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=		0	112320	224640	336960	449280
Add. force due to submerged foundation ,F_{buoy} (lb)	=		75816	75816	75816	75816	75816
Total ,F_{buoy} (lb)			75816	188136	300456	412776	525096
c) Hydrodynamic Load (F_{dyn})							
Velocity of flow (fps)	=	10					
Width in X Direction (ft)	=	40					
Drag coefficient (C_d) X direction	=	1.25					
Width in Z Direction	=	45					
Drag coefficient (C_d) Z direction	=	1.25					
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}							
Hydrodynamic Pressure (P_d) -X (psf)	=		121.3	121.25	121.25	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=		121.3	121.25	121.25	121.25	121.25
d) Wave loads (f_{brkp}):							
Dynamic pressure coefficient, C_p	=	2.8					
f_{brkp} *(lb per ft acting at stillwater elevation)*	=		25171	25171	25171	25170.9	25171
e) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}							
Weight of the object (lb), W	=	1000					
Depth Coefficient, C_D	=	1					
Blockage Coefficient, C_B	=	1					
Building Str. Coefficient, C_{Str}	=	0.2					
F_i (lb)	=		2000	2000	2000	2000	2000

A.3.10 When stillwater Flood depth is 10 ft

			Level difference				
a) Hydrostatic Load (F_{sta}= $0.5P_h d_s$)			0	1	2	3	4
Source of Water	=	river					
Depth of flood, d_s (ft)	=	10					
Difference in water level (ft)	=		0	1	2	3	4
Hydrostatic Pressure, $P_h = \gamma_w d_s$ (psf)	=		0	62.4	124.8	187.2	249.6
F_{sta} (lb/ft)	=		0	31.2	124.8	280.8	499.2
b) Hydrostatic buoyancy (F_{buoy}):							
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=		0	1	2	3	4
Uplift pressure on slab {$P_{buoy} = h\gamma_w$} (psf)	=		0	62.4	124.8	187.2	249.6
Uplift force on slab {$F_{buoy} = (Vol)\,\gamma_w$} (lb)	=		0	112320	224640	336960	449280
Add. force due to submerged foundation ,F_{buoy} (lb)	=		75816	75816	75816	75816	75816
Total ,F_{buoy} (lb)			75816	188136	300456	412776	525096
c) Hydrodynamic Load (F_{dyn})							
Velocity of flow (fps)	=	10					
Width in X Direction (ft)	=	40					
Drag coefficient (C_d) X direction	=	1.25					
Width in Z Direction	=	45					
Drag coefficient (C_d) Z direction	=	1.25					
Hydrodynamic Pressure (P_d) {$= C_d \rho . V^2/2$}							
Hydrodynamic Pressure (P_d) -X (psf)	=		121.3	121.25	121.25	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=		121.3	121.25	121.25	121.25	121.25
d) Wave loads (f_{brkp}):							
Dynamic pressure coefficient, C_p	=	2.8					
f_{brkp} *(lb per ft acting at stillwater elevation)*	=		31075	31075	31075	31075.2	31075
e) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$							
Weight of the object (lb), W	=	1000					
Depth Coefficient, C_D	=	1					
Blockage Coefficient, C_B	=	1					
Building Str. Coefficient, C_{Str}	=	0.2					
F_i (lb)	=		2000	2000	2000	2000	2000

A.3.11 When stillwater Flood depth is 11 ft

			Level difference				
a) Hydrostatic Load (F_{sta}= $0.5P_hd_s$)			0	1	2	3	4
Source of Water	=	river					
Depth of flood, d_s (ft)	=	11					
Difference in water level (ft)	=		0	1	2	3	4
Hydrostatic Pressure, P_h= $\gamma_w d_s$ (psf)	=		0	62.4	124.8	187.2	249.6
F_{sta} (lb/ft)	=		0	31.2	124.8	280.8	499.2
b) Hydrostatic buoyancy (F_{buoy}):							
Difference in flood depth in interior and exterior side above slab invert level ,h (ft)	=		0	1	2	3	4
Uplift pressure on slab {P_{buoy} = $h\gamma_w$} (psf)	=		0	62.4	124.8	187.2	249.6
Uplift force on slab {F_{buoy} = *(Vol)* γ_w} (lb)	=		0	112320	224640	336960	449280
Add. force due to submerged foundation ,F_{buoy} (lb)	=		75816	75816	75816	75816	75816
Total ,F_{buoy} (lb)			75816	188136	300456	412776	525096
c) Hydrodynamic Load (F_{dyn})							
Velocity of flow (fps)	=	10					
Width in X Direction (ft)	=	40					
Drag coefficient (C_d) X direction	=	1.25					
Width in Z Direction	=	45					
Drag coefficient (C_d) Z direction	=	1.25					
Hydrodynamic Pressure (P_d) {= $C_d\rho.V^2/2$}							
Hydrodynamic Pressure (P_d) -X (psf)	=		121.3	121.25	121.25	121.25	121.25
Hydrodynamic Pressure (P_d) -Z (psf)	=		121.3	121.25	121.25	121.25	121.25
d) Wave loads (f_{brkp}):							
Dynamic pressure coefficient, C_p	=	2.8					
f_{brkp} *(lb per ft acting at stillwater elevation)*	=		37601	37601	37601	37601	37601
e) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}							
Weight of the object (lb), W	=	1000					
Depth Coefficient, C_D	=	1					
Blockage Coefficient, C_B	=	1					
Building Str. Coefficient, C_{Str}	=	0.2					
F_i (lb)	=		2000	2000	2000	2000	2000

A.4 Wind load calculation

Following assumptions are made to calculate wind load on the building based on ASCE (ASCE/SEI 7-10, 2010).

Table A.1 Assumptions made

Details	Assumptions
Wind speed, V(mph)	150
Exposure Category	B
Topographic Factor K_1	0
Topographic Factor K_2	0
Topographic Factor K_3	0
Directionality Factor K_d	0.85

Table A.2 Detail results

Details	Results
Exposure Constant, α	7
Exposure Constant, Z_g	1200
Gust effect factor, G	0.85
Topographical factor, K_{zt}	1
Mean roof height, h(ft)	24.167
Velocity pressure exposure coefficients (Table 27.3-1 ASCE 7/10), K_h	0.659
External pressure coefficient -Windward (Table 27.4-1 ASCE 7/10) C_p	0.8
Velocity pressure calculated using (Eq 27.3-1 ASCE 7/10) at mean roof height, q_h (psf)	32.246

Table A.3 Geometry results

Floor level	Height (ft)	K_z	Width (X), ft	Width (Z), ft	Leeward C_p (X)	Leeward C_p (Z)
Roof	28.333	0.689	166.67 ft^2 (area)	0 ft^2 (area)	0.475	0.5
3rd floor	20	0.624	45.667	40.667	0.475	0.5
2nd floor	10	0.575	45.667	40.667	0.475	0.5

Table A.4 Floor force results

Floor level	Velocity pressure, q_z (psf)	Windward pressure, qGC_p (psf)	Leeward pressure X, q_iGC_{pi} (psf)	Leeward pressure Z, q_iGC_{pi} (psf)	Force X (lb)	Force Z (lb)
Roof	33.745	22.947	13.031	13.705	5996.231	0
3rd floor	30.549	20.773	13.031	13.705	12869.661	7872.419
2nd floor	28.138	19.134	13.031	13.705	13080.268	14996.288

A.5 Gravity load calculation

Table A.5 Gravity load calculation

Floor	Post Dead load (psf)	Live load (psf)
Roof	10	20 (Roof LL)
2nd floor	10	40 (Reducible, LL)
1st floor	10	40 (Reducible, LL)

Computation of dead load of Slab-On-Grade:

Slab portion: (41X46) X $\frac{4}{12}$ X 150 = 94,300 lb. (For normal weight concrete)

Beam portion: (2-$\frac{4}{12}$) X (46 X 5 + 35 X 4) X 1 X150 = 92,500 lb.

Total foundation weight: 186,800 lb.

A.6 Overturning Moment Calculation

A.6.1 Overturning Moment calculation for stillwater depth ranging from 0 to 11 ft for 0 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Moment per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75Fa - 0.6D < 0 to avoid overturning			
	0.6D + Fa uplift	0.6W (Lateral)	0.75Fa	Total
1 (x)	93943.69875	5822.9382	63.65625	-88057.1043
1 (z)	74227.12	4098.8168	63.65625	-70064.647
2 (x)	71672.74875	5822.9382	407.4	-65442.4106
2 (z)	56630.32	4098.8168	203.7	-52327.8032
3 (x)	71672.74875	5822.9382	425.5875	-65424.2231
3 (z)	56630.32	4098.8168	425.5875	-52105.9157
4 (x)	71672.74875	5822.9382	727.5	-65122.3106
4 (z)	56630.32	4098.8168	727.5	-51804.0032
5 (x)	71672.74875	5822.9382	1136.71875	-64713.0918
5 (z)	56630.32	4098.8168	1136.71875	-51394.7845
6 (x)	71672.74875	5822.9382	1636.875	-64212.9356
6 (z)	56630.32	4098.8168	1636.875	-50894.6282
7 (x)	71672.74875	5822.9382	2227.96875	-63621.8418
7 (z)	56630.32	4098.8168	2227.96875	-50303.5345
8 (x)	71672.74875	5822.9382	2910	-62939.8106
8 (z)	56630.32	4098.8168	2910	-49621.5032
9 (x)	71672.74875	5822.9382	3682.96875	-62166.8418
9 (z)	56630.32	4098.8168	3682.96875	-48848.5345
10 (x)	71672.74875	5822.9382	4546.875	-61302.9356
10 (z)	56630.32	4098.8168	4546.875	-47984.6282
11 (x)	71672.74875	5822.9382	5501.71875	-60348.0918
11 (z)	56630.32	4098.8168	5501.71875	-47029.7845
0 (x)	103657.6238	5822.9382		-97834.6856
0 (z)	81902.32	4098.8168		-77803.5032

A.6.2 Overturning Moment calculation for stillwater depth ranging from 0 to 11 ft for 1 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Moment per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75Fa - 0.6D < 0 to avoid overturning			
	0.6D + Fa uplift	0.6W (Lateral)	0.75Fa	Total
1 (x)	93943.69875	5822.9382	71.45625	-88049.3043
1 (z)	74227.12	4098.8168	71.45625	-70056.847
2 (x)	71672.74875	5822.9382	469.8	-65380.0106
2 (z)	56630.32	4098.8168	234.9	-52296.6032
3 (x)	24287.74875	5822.9382	480.1875	-17984.6231
3 (z)	19190.32	4098.8168	480.1875	-14611.3157
4 (x)	24287.74875	5822.9382	805.5	-17659.3106
4 (z)	19190.32	4098.8168	805.5	-14286.0032
5 (x)	24287.74875	5822.9382	1238.11875	-17226.6918
5 (z)	19190.32	4098.8168	1238.11875	-13853.3845
6 (x)	24287.74875	5822.9382	1761.675	-16703.1356
6 (z)	19190.32	4098.8168	1761.675	-13329.8282
7 (x)	24287.74875	5822.9382	2376.16875	-16088.6418
7 (z)	19190.32	4098.8168	2376.16875	-12715.3345
8 (x)	24287.74875	5822.9382	3081.6	-15383.2106
8 (z)	19190.32	4098.8168	3081.6	-12009.9032
9 (x)	24287.74875	5822.9382	3877.96875	-14586.8418
9 (z)	19190.32	4098.8168	3877.96875	-11213.5345
10 (x)	24287.74875	5822.9382	4765.275	-13699.5356
10 (z)	19190.32	4098.8168	4765.275	-10326.2282
11 (x)	24287.74875	5822.9382	5743.51875	-12721.2918
11 (z)	19190.32	4098.8168	5743.51875	-9347.98445
0 (x)	103657.6238	5822.9382		-97834.6856
0 (z)	81902.32	4098.8168		-77803.5032

A.6.3 Overturning Moment calculation for stillwater depth ranging from 0 to 11 ft for 2 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Moment per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75Fa - 0.6D < 0 to avoid overturning			
	0.6D + Fa uplift	0.6W (Lateral)	0.75Fa	Total
1 (x)	93943.69875	5822.9382	71.45625	-88049.3043
1 (z)	74227.12	4098.8168	71.45625	-70056.847
2 (x)	71672.74875	5822.9382	532.2	-65317.6106
2 (z)	56630.32	4098.8168	266.1	-52265.4032
3 (x)	24287.74875	5822.9382	581.5875	-17883.2231
3 (z)	19190.32	4098.8168	581.5875	-14509.9157
4 (x)	-23097.25125	5822.9382	977.1	29897.2895
4 (z)	-18249.68	4098.8168	977.1	23325.5968
5 (x)	-23097.25125	5822.9382	1479.91875	30400.1082
5 (z)	-18249.68	4098.8168	1479.91875	23828.4156
6 (x)	-23097.25125	5822.9382	2073.675	30993.8645
6 (z)	-18249.68	4098.8168	2073.675	24422.1718
7 (x)	-23097.25125	5822.9382	2758.36875	31678.5582
7 (z)	-18249.68	4098.8168	2758.36875	25106.8656
8 (x)	-23097.25125	5822.9382	3534	32454.1895
8 (z)	-18249.68	4098.8168	3534	25882.4968
9 (x)	-23097.25125	5822.9382	4400.56875	33320.7582
9 (z)	-18249.68	4098.8168	4400.56875	26749.0656
10 (x)	-23097.25125	5822.9382	5358.075	34278.2645
10 (z)	-18249.68	4098.8168	5358.075	27706.5718
11 (x)	-23097.25125	5822.9382	6406.51875	35326.7082
11 (z)	-18249.68	4098.8168	6406.51875	28755.0156
0 (x)	103657.6238	5822.9382		-97834.6856
0 (z)	81902.32	4098.8168		-77803.5032

A.6.4 Overturning Moment calculation for stillwater depth ranging from 0 to 11 ft for 3 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Moment per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75Fa - 0.6D < 0 to avoid overturning			
	0.6D + Fa uplift	0.6W (Lateral)	0.75Fa	Total
1 (x)	93943.69875	5822.9382	71.45625	-88049.3043
1 (z)	74227.12	4098.8168	71.45625	-70056.847
2 (x)	71672.74875	5822.9382	532.2	-65317.6106
2 (z)	56630.32	4098.8168	266.1	-52265.4032
3 (x)	24287.74875	5822.9382	636.1875	-17828.6231
3 (z)	19190.32	4098.8168	636.1875	-14455.3157
4 (x)	-23097.25125	5822.9382	1148.7	30068.8895
4 (z)	-18249.68	4098.8168	1148.7	23497.1968
5 (x)	-70482.25125	5822.9382	1768.51875	78073.7082
5 (z)	-55689.68	4098.8168	1768.51875	61557.0156
6 (x)	-70482.25125	5822.9382	2479.275	78784.4645
6 (z)	-55689.68	4098.8168	2479.275	62267.7718
7 (x)	-70482.25125	5822.9382	3280.96875	79586.1582
7 (z)	-55689.68	4098.8168	3280.96875	63069.4656
8 (x)	-70482.25125	5822.9382	4173.6	80478.7895
8 (z)	-55689.68	4098.8168	4173.6	63962.0968
9 (x)	-70482.25125	5822.9382	5157.16875	81462.3582
9 (z)	-55689.68	4098.8168	5157.16875	64945.6656
10 (x)	-70482.25125	5822.9382	6231.675	82536.8645
10 (z)	-55689.68	4098.8168	6231.675	66020.1718
11 (x)	-70482.25125	5822.9382	7397.11875	83702.3082
11 (z)	-55689.68	4098.8168	7397.11875	67185.6156
0 (x)	103657.6238	5822.9382		-97834.6856
0 (z)	81902.32	4098.8168		-77803.5032

A.6.5 Overturning Moment calculation for stillwater depth ranging from 0 to 11 ft for 4 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Moment per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75Fa - 0.6D < 0 to avoid overturning			
	0.6D + Fa uplift	0.6W (Lateral)	0.75Fa	Total
1 (x)	93943.69875	5822.9382	71.45625	-88049.304
1 (z)	74227.12	4098.8168	71.45625	-70056.847
2 (x)	71672.74875	5822.9382	532.2	-65317.611
2 (z)	56630.32	4098.8168	266.1	-52265.403
3 (x)	24287.74875	5822.9382	636.1875	-17828.623
3 (z)	19190.32	4098.8168	636.1875	-14455.316
4 (x)	-23097.25125	5822.9382	1226.7	30146.8895
4 (z)	-18249.68	4098.8168	1226.7	23575.1968
5 (x)	-70482.25125	5822.9382	2010.31875	78315.5082
5 (z)	-55689.68	4098.8168	2010.31875	61798.8156
6 (x)	-117867.2513	5822.9382	2884.875	126575.064
6 (z)	-93129.68	4098.8168	2884.875	100113.372
7 (x)	-117867.2513	5822.9382	3850.36875	127540.558
7 (z)	-93129.68	4098.8168	3850.36875	101078.866
8 (x)	-117867.2513	5822.9382	4906.8	128596.989
8 (z)	-93129.68	4098.8168	4906.8	102135.297
9 (x)	-117867.2513	5822.9382	6054.16875	129744.358
9 (z)	-93129.68	4098.8168	6054.16875	103282.666
10 (x)	-117867.2513	5822.9382	7292.475	130982.664
10 (z)	-93129.68	4098.8168	7292.475	104520.972
11 (x)	-117867.2513	5822.9382	8621.71875	132311.908
11 (z)	-93129.68	4098.8168	8621.71875	105850.216
0 (x)	103657.6238	5822.9382		-97834.686
0 (z)	81902.32	4098.8168		-77803.503

A.7 Sliding Forces Calculation

A.7.1 Sliding Forces calculation for stillwater depth ranging from 0 to 11 ft for 0 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in respective Direction			
	(-)0.45(0.6D+0.75Fy) + 0.6W + 0.75Fa <0 (safe) , where 0.45 is the coeffiient of friction			
	(-)0.45(0.6D+Fy)	0.6W	0.75Fa	Total
1 (x)	-1878.873975	389.24892	109.125	-1380.500055
1 (z)	-1670.1102	304.9160933	127.3125	-1237.881607
2 (x)	-1433.454975	389.24892	189.15	-855.056055
2 (z)	-1274.1822	304.9160933	203.7	-765.5661067
3 (x)	-1433.454975	389.24892	283.725	-760.481055
3 (z)	-1274.1822	304.9160933	283.725	-685.5411067
4 (x)	-1433.454975	389.24892	363.75	-680.456055
4 (z)	-1274.1822	304.9160933	363.75	-605.5161067
5 (x)	-1433.454975	389.24892	454.6875	-589.518555
5 (z)	-1274.1822	304.9160933	454.6875	-514.5786067
6 (x)	-1433.454975	389.24892	545.625	-498.581055
6 (z)	-1274.1822	304.9160933	545.625	-423.6411067
7 (x)	-1433.454975	389.24892	636.5625	-407.643555
7 (z)	-1274.1822	304.9160933	636.5625	-332.7036067
8 (x)	-1433.454975	389.24892	727.5	-316.706055
8 (z)	-1274.1822	304.9160933	727.5	-241.7661067
9 (x)	-1433.454975	389.24892	818.4375	-225.768555
9 (z)	-1274.1822	304.9160933	818.4375	-150.8286067
10 (x)	-1433.454975	389.24892	909.375	-134.831055
10 (z)	-1274.1822	304.9160933	909.375	-59.89110667
11 (x)	-1433.454975	389.24892	1000.3125	-43.893555
11 (z)	-1274.1822	304.9160933	1000.3125	31.04639333
0 (x)	-2073.152475	389.24892		-1683.903555
0 (z)	-1842.8022	304.9160933		-1537.886107

A.7.2 Sliding Forces calculation for stillwater depth ranging from 0 to 11 ft for 1 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in respective Direction			
	(-)0.45(0.6D+0.75Fy) + 0.6W + 0.75Fa <0 (safe) , where 0.45 is the coeffiient of friction			
	(-)0.45(0.6D+Fy)	0.6W	0.75Fa	Total
1 (x)	-1878.873975	389.24892	132.525	-1357.100055
1 (z)	-1670.1102	304.9160933	150.7125	-1214.481607
2 (x)	-1433.454975	389.24892	212.55	-831.656055
2 (z)	-1274.1822	304.9160933	227.1	-742.1661067
3 (x)	-485.754975	389.24892	307.125	210.618945
3 (z)	-431.7822	304.9160933	307.125	180.2588933
4 (x)	-485.754975	389.24892	387.15	290.643945
4 (z)	-431.7822	304.9160933	387.15	260.2838933
5 (x)	-485.754975	389.24892	478.0875	381.581445
5 (z)	-431.7822	304.9160933	478.0875	351.2213933
6 (x)	-485.754975	389.24892	569.025	472.518945
6 (z)	-431.7822	304.9160933	569.025	442.1588933
7 (x)	-485.754975	389.24892	659.9625	563.456445
7 (z)	-431.7822	304.9160933	659.9625	533.0963933
8 (x)	-485.754975	389.24892	750.9	654.393945
8 (z)	-431.7822	304.9160933	750.9	624.0338933
9 (x)	-485.754975	389.24892	841.8375	745.331445
9 (z)	-431.7822	304.9160933	841.8375	714.9713933
10 (x)	-485.754975	389.24892	932.775	836.268945
10 (z)	-431.7822	304.9160933	932.775	805.9088933
11 (x)	-485.754975	389.24892	1023.7125	927.206445
11 (z)	-431.7822	304.9160933	1023.7125	896.8463933
0 (x)	-2073.152475	389.24892		-1683.903555
0 (z)	-1842.8022	304.9160933		-1537.886107

A.7.3 Sliding Forces calculation for stillwater depth ranging from 0 to 11 ft for 2 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in respective Direction			
	(-)0.45(0.6D+0.75Fy) + 0.6W + 0.75Fa <0 (safe) , where 0.45 is the coeffiient of friction			
	(-)0.45(0.6D+Fy)	0.6W	0.75Fa	Total
1 (x)	-1878.873975	389.24892	132.525	-1357.100055
1 (z)	-1670.1102	304.9160933	150.7125	-1214.481607
2 (x)	-1433.454975	389.24892	282.75	-761.456055
2 (z)	-1274.1822	304.9160933	297.3	-671.9661067
3 (x)	-485.754975	389.24892	377.325	280.818945
3 (z)	-431.7822	304.9160933	377.325	250.4588933
4 (x)	461.945025	389.24892	457.35	1308.543945
4 (z)	410.6178	304.9160933	457.35	1172.883893
5 (x)	461.945025	389.24892	548.2875	1399.481445
5 (z)	410.6178	304.9160933	548.2875	1263.821393
6 (x)	461.945025	389.24892	639.225	1490.418945
6 (z)	410.6178	304.9160933	639.225	1354.758893
7 (x)	461.945025	389.24892	730.1625	1581.356445
7 (z)	410.6178	304.9160933	730.1625	1445.696393
8 (x)	461.945025	389.24892	821.1	1672.293945
8 (z)	410.6178	304.9160933	821.1	1536.633893
9 (x)	461.945025	389.24892	912.0375	1763.231445
9 (z)	410.6178	304.9160933	912.0375	1627.571393
10 (x)	461.945025	389.24892	1002.975	1854.168945
10 (z)	410.6178	304.9160933	1002.975	1718.508893
11 (x)	461.945025	389.24892	1093.9125	1945.106445
11 (z)	410.6178	304.9160933	1093.9125	1809.446393
0 (x)	-2073.152475	389.24892		-1683.903555
0 (z)	-1842.8022	304.9160933		-1537.886107

A.7.4 Sliding Forces calculation for stillwater depth ranging from 0 to 11 ft for 3 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in respective Direction			
	(-)0.45(0.6D+0.75Fy) + 0.6W + 0.75Fa <0 (safe) , where 0.45 is the coeffiient of friction			
	(-)0.45(0.6D+0.75Fy)	0.6W	0.75Fa	Total
1 (x)	-1878.873975	389.24892	132.525	-1357.100055
1 (z)	-1670.1102	304.9160933	150.7125	-1214.481607
2 (x)	-1433.454975	389.24892	282.75	-761.456055
2 (z)	-1274.1822	304.9160933	297.3	-671.9661067
3 (x)	-485.754975	389.24892	494.325	397.818945
3 (z)	-431.7822	304.9160933	494.325	367.4588933
4 (x)	461.945025	389.24892	574.35	1425.543945
4 (z)	410.6178	304.9160933	574.35	1289.883893
5 (x)	1409.645025	389.24892	665.2875	2464.181445
5 (z)	1253.0178	304.9160933	665.2875	2223.221393
6 (x)	1409.645025	389.24892	756.225	2555.118945
6 (z)	1253.0178	304.9160933	756.225	2314.158893
7 (x)	1409.645025	389.24892	847.1625	2646.056445
7 (z)	1253.0178	304.9160933	847.1625	2405.096393
8 (x)	1409.645025	389.24892	938.1	2736.993945
8 (z)	1253.0178	304.9160933	938.1	2496.033893
9 (x)	1409.645025	389.24892	1029.0375	2827.931445
9 (z)	1253.0178	304.9160933	1029.0375	2586.971393
10 (x)	1409.645025	389.24892	1119.975	2918.868945
10 (z)	1253.0178	304.9160933	1119.975	2677.908893
11 (x)	1409.645025	389.24892	1210.9125	3009.806445
11 (z)	1253.0178	304.9160933	1210.9125	2768.846393
0 (x)	-2073.152475	389.24892		-1683.903555
0 (z)	-1842.8022	304.9160933		-1537.886107

A.7.5 Sliding Forces calculation for stillwater depth ranging from 0 to 11 ft for 4 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in respective Direction			
	(-)0.45(0.6D+0.75Fy) + 0.6W + 0.75Fa <0 (safe) , where 0.45 is the coeffiient of friction			
	(-)0.45(0.6D+Fy)	0.6W	0.75Fa	Total
1 (x)	-1878.873975	389.24892	132.525	-1357.100055
1 (z)	-1670.1102	304.9160933	150.7125	-1214.481607
2 (x)	-1433.454975	389.24892	282.75	-761.456055
2 (z)	-1274.1822	304.9160933	297.3	-671.9661067
3 (x)	-485.754975	389.24892	494.325	397.818945
3 (z)	-431.7822	304.9160933	494.325	367.4588933
4 (x)	461.945025	389.24892	738.15	1589.343945
4 (z)	410.6178	304.9160933	738.15	1453.683893
5 (x)	1409.645025	389.24892	829.0875	2627.981445
5 (z)	1253.0178	304.9160933	829.0875	2387.021393
6 (x)	2357.345025	389.24892	920.025	3666.618945
6 (z)	2095.4178	304.9160933	920.025	3320.358893
7 (x)	2357.345025	389.24892	1010.9625	3757.556445
7 (z)	2095.4178	304.9160933	1010.9625	3411.296393
8 (x)	2357.345025	389.24892	1101.9	3848.493945
8 (z)	2095.4178	304.9160933	1101.9	3502.233893
9 (x)	2357.345025	389.24892	1192.8375	3939.431445
9 (z)	2095.4178	304.9160933	1192.8375	3593.171393
10 (x)	2357.345025	389.24892	1283.775	4030.368945
10 (z)	2095.4178	304.9160933	1283.775	3684.108893
11 (x)	2357.345025	389.24892	1374.7125	4121.306445
11 (z)	2095.4178	304.9160933	1374.7125	3775.046393
0 (x)	-2073.152475	389.24892		-1683.903555
0 (z)	-1842.8022	304.9160933		-1537.886107

A.8 Uplift Forces calculation

A.8.1 Uplift forces calculation for stillwater depth ranging from 0 to 11 ft for 0 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75 Fa +0.6D > 0 to avoid uplift			
	0.6D (+ve)	0.6W (uplift)	0.75Fa (uplift)	Total
1 (x)	4607.0055	-444.015	-431.73	3731.2605
1 (z)	4095.116	-394.68	-383.76	3316.676
2 (x)	4607.0055	-444.015	-1421.55	2741.4405
2 (z)	4095.116	-394.68	-1263.6	2436.836
3 (x)	4607.0055	-444.015	-1421.55	2741.4405
3 (z)	4095.116	-394.68	-1263.6	2436.836
4 (x)	4607.0055	-444.015	-1421.55	2741.4405
4 (z)	4095.116	-394.68	-1263.6	2436.836
5 (x)	4607.0055	-444.015	-1421.55	2741.4405
5 (z)	4095.116	-394.68	-1263.6	2436.836
6 (x)	4607.0055	-444.015	-1421.55	2741.4405
6 (z)	4095.116	-394.68	-1263.6	2436.836
7 (x)	4607.0055	-444.015	-1421.55	2741.4405
7 (z)	4095.116	-394.68	-1263.6	2436.836
8 (x)	4607.0055	-444.015	-1421.55	2741.4405
8 (z)	4095.116	-394.68	-1263.6	2436.836
9 (x)	4607.0055	-444.015	-1421.55	2741.4405
9 (z)	4095.116	-394.68	-1263.6	2436.836
10 (x)	4607.0055	-444.015	-1421.55	2741.4405
10 (z)	4095.116	-394.68	-1263.6	2436.836
11 (x)	4607.0055	-444.015	-1421.55	2741.4405
11 (z)	4095.116	-394.68	-1263.6	2436.836
0 (x)	4607.0055	-444.015		4162.9905
0 (z)	4095.116	-394.68		3700.436

A.8.2 Uplift forces calculation for stillwater depth ranging from 0 to 11 ft for 1 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75 Fa +0.6D > 0 to avoid uplift			
	0.6D (+ve)	0.6W (uplift)	0.75Fa (uplift)	Total
1 (x)	4607.0055	-444.015	-431.73	3731.2605
1 (z)	4095.116	-394.68	-383.76	3316.676
2 (x)	4607.0055	-444.015	-1421.55	2741.4405
2 (z)	4095.116	-394.68	-1263.6	2436.836
3 (x)	4607.0055	-444.015	-3527.55	635.4405
3 (z)	4095.116	-394.68	-3135.6	564.836
4 (x)	4607.0055	-444.015	-3527.55	635.4405
4 (z)	4095.116	-394.68	-3135.6	564.836
5 (x)	4607.0055	-444.015	-3527.55	635.4405
5 (z)	4095.116	-394.68	-3135.6	564.836
6 (x)	4607.0055	-444.015	-3527.55	635.4405
6 (z)	4095.116	-394.68	-3135.6	564.836
7 (x)	4607.0055	-444.015	-3527.55	635.4405
7 (z)	4095.116	-394.68	-3135.6	564.836
8 (x)	4607.0055	-444.015	-3527.55	635.4405
8 (z)	4095.116	-394.68	-3135.6	564.836
9 (x)	4607.0055	-444.015	-3527.55	635.4405
9 (z)	4095.116	-394.68	-3135.6	564.836
10 (x)	4607.0055	-444.015	-3527.55	635.4405
10 (z)	4095.116	-394.68	-3135.6	564.836
11 (x)	4607.0055	-444.015	-3527.55	635.4405
11 (z)	4095.116	-394.68	-3135.6	564.836
0 (x)	4607.0055	-444.015		4162.9905
0 (z)	4095.116	-394.68		3700.436

A.8.3 Uplift forces calculation for stillwater depth ranging from 0 to 11 ft for 2 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75 Fa +0.6D > 0 to avoid uplift			
	0.6D (+ve)	0.6W (uplift)	0.75Fa (uplift)	Total
1 (x)	4607.0055	-444.015	-431.73	3731.2605
1 (z)	4095.116	-394.68	-383.76	3316.676
2 (x)	4607.0055	-444.015	-1421.55	2741.4405
2 (z)	4095.116	-394.68	-1263.6	2436.836
3 (x)	4607.0055	-444.015	-3527.55	635.4405
3 (z)	4095.116	-394.68	-3135.6	564.836
4 (x)	4607.0055	-444.015	-5633.55	-1470.5595
4 (z)	4095.116	-394.68	-5007.6	-1307.164
5 (x)	4607.0055	-444.015	-5633.55	-1470.5595
5 (z)	4095.116	-394.68	-5007.6	-1307.164
6 (x)	4607.0055	-444.015	-5633.55	-1470.5595
6 (z)	4095.116	-394.68	-5007.6	-1307.164
7 (x)	4607.0055	-444.015	-5633.55	-1470.5595
7 (z)	4095.116	-394.68	-5007.6	-1307.164
8 (x)	4607.0055	-444.015	-5633.55	-1470.5595
8 (z)	4095.116	-394.68	-5007.6	-1307.164
9 (x)	4607.0055	-444.015	-5633.55	-1470.5595
9 (z)	4095.116	-394.68	-5007.6	-1307.164
10 (x)	4607.0055	-444.015	-5633.55	-1470.5595
10 (z)	4095.116	-394.68	-5007.6	-1307.164
11 (x)	4607.0055	-444.015	-5633.55	-1470.5595
11 (z)	4095.116	-394.68	-5007.6	-1307.164
0 (x)	4607.0055	-444.015		4162.9905
0 (z)	4095.116	-394.68		3700.436

A.8.4 Uplift forces calculation for stillwater depth ranging from 0 to 11 ft for 3 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75 Fa +0.6D > 0 to avoid uplift			
	0.6D (+ve)	0.6W (uplift)	0.75Fa (uplift)	Total
1 (x)	4607.0055	-444.015	-431.73	3731.2605
1 (z)	4095.116	-394.68	-383.76	3316.676
2 (x)	4607.0055	-444.015	-1421.55	2741.4405
2 (z)	4095.116	-394.68	-1263.6	2436.836
3 (x)	4607.0055	-444.015	-3527.55	635.4405
3 (z)	4095.116	-394.68	-3135.6	564.836
4 (x)	4607.0055	-444.015	-5633.55	-1470.5595
4 (z)	4095.116	-394.68	-5007.6	-1307.164
5 (x)	4607.0055	-444.015	-7739.55	-3576.5595
5 (z)	4095.116	-394.68	-6879.6	-3179.164
6 (x)	4607.0055	-444.015	-7739.55	-3576.5595
6 (z)	4095.116	-394.68	-6879.6	-3179.164
7 (x)	4607.0055	-444.015	-7739.55	-3576.5595
7 (z)	4095.116	-394.68	-6879.6	-3179.164
8 (x)	4607.0055	-444.015	-7739.55	-3576.5595
8 (z)	4095.116	-394.68	-6879.6	-3179.164
9 (x)	4607.0055	-444.015	-7739.55	-3576.5595
9 (z)	4095.116	-394.68	-6879.6	-3179.164
10 (x)	4607.0055	-444.015	-7739.55	-3576.5595
10 (z)	4095.116	-394.68	-6879.6	-3179.164
11 (x)	4607.0055	-444.015	-7739.55	-3576.5595
11 (z)	4095.116	-394.68	-6879.6	-3179.164
0 (x)	4607.0055	-444.015		4162.9905
0 (z)	4095.116	-394.68		3700.436

A.8.5 Uplift forces calculation for stillwater depth ranging from 0 to 11 ft for 4 ft level difference in interior and exterior face

Stillwater depth, ft (Direction)	Load per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75 Fa +0.6D > 0 to avoid uplift			
	0.6D (+ve)	0.6W (uplift)	0.75Fa (uplift)	Total
1 (x)	4607.0055	-444.015	-431.73	3731.2605
1 (z)	4095.116	-394.68	-383.76	3316.676
2 (x)	4607.0055	-444.015	-1421.55	2741.4405
2 (z)	4095.116	-394.68	-1263.6	2436.836
3 (x)	4607.0055	-444.015	-3527.55	635.4405
3 (z)	4095.116	-394.68	-3135.6	564.836
4 (x)	4607.0055	-444.015	-5633.55	-1470.5595
4 (z)	4095.116	-394.68	-5007.6	-1307.164
5 (x)	4607.0055	-444.015	-7739.55	-3576.5595
5 (z)	4095.116	-394.68	-6879.6	-3179.164
6 (x)	4607.0055	-444.015	-9845.55	-5682.5595
6 (z)	4095.116	-394.68	-8751.6	-5051.164
7 (x)	4607.0055	-444.015	-9845.55	-5682.5595
7 (z)	4095.116	-394.68	-8751.6	-5051.164
8 (x)	4607.0055	-444.015	-9845.55	-5682.5595
8 (z)	4095.116	-394.68	-8751.6	-5051.164
9 (x)	4607.0055	-444.015	-9845.55	-5682.5595
9 (z)	4095.116	-394.68	-8751.6	-5051.164
10 (x)	4607.0055	-444.015	-9845.55	-5682.5595
10 (z)	4095.116	-394.68	-8751.6	-5051.164
11 (x)	4607.0055	-444.015	-9845.55	-5682.5595
11 (z)	4095.116	-394.68	-8751.6	-5051.164
0 (x)	4607.0055	-444.015		4162.9905
0 (z)	4095.116	-394.68		3700.436

A.9 Calculation for Anchor bolts for different flood conditions:

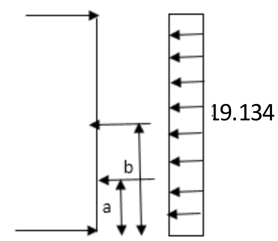
A.9.1 Spacing and number of 5/8” anchor bolts when stillwater elevation is 0 ft and stillwater elevation difference in interior and exterior face of building is 0 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	107.08	136.382	149.033	88.842
Number of bolt, N	=	1.096	2.79179	3.05076	0.909315

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	0
Dynamic flood load, $F_{a\ (dynamic)}$	=	0
h from base where static load is applied, a (ft)	=	0
h from base where dynamic load is applied, b (ft)	=	0
Reaction at Base, R, (lbs)	=	95.67



Number of bolt, N	=	0.9792	1.9584	1.9584	0.979201
Final number or bolt required, N	=	**2**	**3**	**4**	**2**
Spacing of bolt, S	=	**48**	**54**	**36**	**48**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	195.26	199.996
Number of bolt, N	=	8	9
Spacing of bolt, S	=	**32.571**	**28.5**

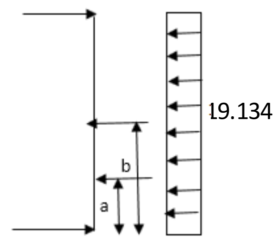
(assuming end bolts are 6-inches from end)

A.9.2 Spacing and number of 5/8” anchor bolts when stillwater elevation is 1 ft and stillwater elevation difference in interior and exterior face of building is 0 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	115.41	144.015	154.883	93.142
Number of bolt, N	=	1.1812	2.94804	3.17052	0.953327

B) Check for lateral force perpendicular to wall		
Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	0
Dynamic flood load, $F_{a\ (dynamic)}$	=	169.75
h from base where static load is applied, a (ft)	=	1
h from base where dynamic load is applied, b (ft)	=	0.5
Reaction at Base, R, (lbs)	=	256.93



Number of bolt, N	=	2.6298	5.25951	5.25951	2.629755
Final number or bolt required, N	=	**3**	**6**	**6**	**3**
Spacing of bolt, S	=	**24**	**21.6**	**21.6**	**24**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	211.7	208.051
Number of bolt, N	=	9	9
Spacing of bolt, S	=	**28.5**	**28.5**

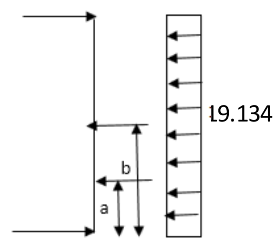
(assuming end bolts are 6-inches from end)

A.9.3 Spacing and number of 5/8” anchor bolts when stillwater elevation is 1 ft and stillwater elevation difference in interior and exterior face of building is 1 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	117.24	145.684	156.138	94.064
Number of bolt, N	=	1.2	2.98221	3.19621	0.962764

B) Check for lateral force perpendicular to wall		
Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	31.2
Dynamic flood load, $F_{a\ (dynamic)}$	=	169.75
h from base where static load is applied, a (ft)	=	0.3333
h from base where dynamic load is applied, b (ft)	=	0.5
Reaction at Base, R, (lbs)	=	287.09



Number of bolt, N	=	2.9384	5.8769	5.8769	2.938448
Final number or bolt required, N	=	**3**	**6**	**6**	**3**
Spacing of bolt, S	=	**24**	**21.6**	**21.6**	**24**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	215.21	209.775
Number of bolt, N	=	9	9
Spacing of bolt, S	=	**28.5**	**28.5**

(assuming end bolts are 6-inches from end)

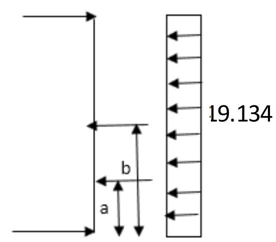
A.9.4 Spacing and number of 5/8" anchor bolts when stillwater elevation is 2 ft and stillwater elevation difference in interior and exterior face of building is 0 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

		R1	R4	R7	R10
a) For Lateral force parallel to wall					
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	122.12	150.005	159.348	96.418
Number of bolt, N	=	1.2499	3.07066	3.26192	0.986857

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	0
Dynamic flood load, $F_{a\ (dynamic)}$	=	271.6
h from base where static load is applied, a (ft)	=	2
h from base where dynamic load is applied, b (ft)	=	1
Reaction at Base, R, (lbs)	=	340.11



		R1	R4	R7	R10
Number of bolt, N	=	3.4811	6.96219	6.96219	3.481093
Final number or bolt required, N	=	**4**	**7**	**7**	**4**
Spacing of bolt, S	=	**16**	**18**	**18**	**16**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

		R1	R3
a) For Lateral force parallel to wall			
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	224.16	214.046
Number of bolt, N	=	10	9
Spacing of bolt, S	=	**25.333**	**28.5**

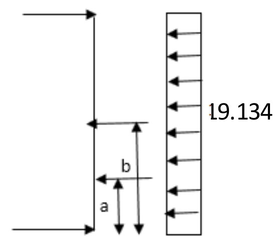
(assuming end bolts are 6-inches from end)

A.9.5 Spacing and number of 5/8” anchor bolts when stillwater elevation is 2 ft and stillwater elevation difference in interior and exterior face of building is 1 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	123.98	151.692	160.612	97.347
Number of bolt, N	=	1.269	3.10519	3.28779	0.996366

B) Check for lateral force perpendicular to wall		
Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	31.2
Dynamic flood load, $F_{a\ (dynamic)}$	=	271.6
h from base where static load is applied, a (ft)	=	1.3333
h from base where dynamic load is applied, b (ft)	=	1
Reaction at Base, R, (lbs)	=	367.15



Number of bolt, N	=	3.7579	7.5157	7.5157	3.757852
Final number or bolt required, N	=	**4**	**8**	**8**	**4**
Spacing of bolt, S	=	**16**	**15.4286**	**15.4286**	**16**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	227.76	215.788
Number of bolt, N	=	10	9
Spacing of bolt, S	=	**25.333**	**28.5**

(assuming end bolts are 6-inches from end)

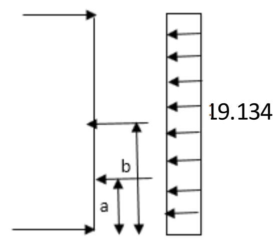
A.9.6 Spacing and number of 5/8” anchor bolts when stillwater elevation is 2 ft and stillwater elevation difference in interior and exterior face of building is 2 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

		R1	R4	R7	R10
a) For Lateral force parallel to wall					
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	129.48	156.701	164.375	100.113
Number of bolt, N	=	1.3253	3.20773	3.36482	1.024676

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	124.8
Dynamic flood load, $F_{a\ (dynamic)}$	=	271.6
h from base where static load is applied, a (ft)	=	0.6667
h from base where dynamic load is applied, b (ft)	=	1
Reaction at Base, R, (lbs)	=	456.59



Number of bolt, N	=	4.6733	9.34658	9.34658	4.673288
Final number or bolt required, N	=	**5**	**10**	**10**	**5**
Spacing of bolt, S	=	**12**	**12**	**12**	**12**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

		R1	R3
a) For Lateral force parallel to wall			
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	238.32	220.967
Number of bolt, N	=	10	10
Spacing of bolt, S	=	**25.333**	**25.3333**

(assuming end bolts are 6-inches from end)

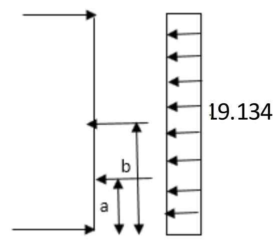
A.9.7 Spacing and number of 5/8” anchor bolts when stillwater elevation is 3 ft and stillwater elevation difference in interior and exterior face of building is 0 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

		R1	R4	R7	R10
a) For Lateral force parallel to wall					
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	137.53	163.856	170.098	104.355
Number of bolt, N	=	1.4077	3.3542	3.48197	1.068094

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	0
Dynamic flood load, $F_{a\ (dynamic)}$	=	378.3
h from base where static load is applied, a (ft)	=	3
h from base where dynamic load is applied, b (ft)	=	1.5
Reaction at Base, R, (lbs)	=	417.23



		R1	R4	R7	R10
Number of bolt, N	=	4.2704	8.54076	8.54076	4.27038
Final number or bolt required, N	=	**5**	**9**	**9**	**5**
Spacing of bolt, S	=	**12**	**13.5**	**13.5**	**12**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

		R1	R3
a) For Lateral force parallel to wall			
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	246.97	227.05
Number of bolt, N	=	11	10
Spacing of bolt, S	=	**22.8**	**25.3333**

(assuming end bolts are 6-inches from end)

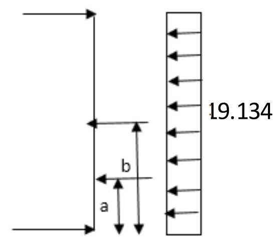
A.9.8 Spacing and number of 5/8” anchor bolts when stillwater elevation is 3 ft and stillwater elevation difference in interior and exterior face of building is 1 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

		R1	R4	R7	R10
a) For Lateral force parallel to wall					
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	141.03	166.948	172.468	106.103
Number of bolt, N	=	1.4435	3.41749	3.53049	1.085985

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	31.2
Dynamic flood load, $F_{a\ (dynamic)}$	=	378.3
h from base where static load is applied, a (ft)	=	2.3333
h from base where dynamic load is applied, b (ft)	=	1.5
Reaction at Base, R, (lbs)	=	441.15



		R1	R4	R7	R10
Number of bolt, N	=	4.5152	9.03041	9.03041	4.515206
Final number or bolt required, N	=	**5**	**10**	**10**	**5**
Spacing of bolt, S	=	**12**	**12**	**12**	**12**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

		R1	R3
a) For Lateral force parallel to wall			
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	251.88	229.835
Number of bolt, N	=	11	10
Spacing of bolt, S	=	**22.8**	**25.3333**

(assuming end bolts are 6-inches from end)

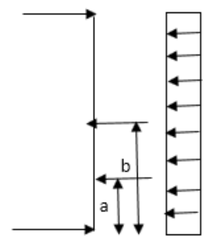
A.9.9 Spacing and number of 5/8” anchor bolts when stillwater elevation is 3 ft and stillwater elevation difference in interior and exterior face of building is 2 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	146.87	172.279	176.495	109.066
Number of bolt, N	=	1.5032	3.52662	3.61292	1.116312

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	124.8
Dynamic flood load, $F_{a\ (dynamic)}$	=	378.3
h from base where static load is applied, a (ft)	=	1.6667
h from base where dynamic load is applied, b (ft)	=	1.5
Reaction at Base, R, (lbs)	=	521.23



Number of bolt, N	=	5.3348	10.6697	10.6697	5.33484
Final number or bolt required, N	=	**6**	**11**	**11**	**6**
Spacing of bolt, S	=	**9.6**	**10.8**	**10.8**	**9.6**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	262.89	235.268
Number of bolt, N	=	11	10
Spacing of bolt, S	=	**22.8**	**25.3333**

(assuming end bolts are 6-inches from end)

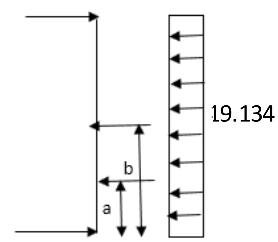
A.9.10 Spacing and number of 5/8” anchor bolts when stillwater elevation is 3 ft and stillwater elevation difference in interior and exterior face of building is 3 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	156.16	180.854	182.997	113.853
Number of bolt, N	=	1.5983	3.70215	3.74602	1.165308

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	280.8
Dynamic flood load, $F_{a\ (dynamic)}$	=	378.3
h from base where static load is applied, a (ft)	=	1
h from base where dynamic load is applied, b (ft)	=	1.5
Reaction at Base, R, (lbs)	=	669.95



Number of bolt, N	=	6.857	13.714	13.714	6.857019
Final number or bolt required, N	=	**7**	**14**	**14**	**7**
Spacing of bolt, S	=	**8**	**8.30769**	**8.30769**	**8**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	280.73	244.165
Number of bolt, N	=	12	10
Spacing of bolt, S	=	**20.727**	**25.3333**

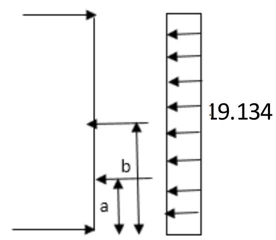
(assuming end bolts are 6-inches from end)

A.9.11 Spacing and number of 5/8” anchor bolts when stillwater elevation is 4 ft and stillwater elevation difference in interior and exterior face of building is 0 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	153.68	179.361	182.602	113.619
Number of bolt, N	=	1.573	3.67159	3.73793	1.162913

B) Check for lateral force perpendicular to wall		
Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	0
Dynamic flood load, $F_{a\ (dynamic)}$	=	485
h from base where static load is applied, a (ft)	=	4
h from base where dynamic load is applied, b (ft)	=	2
Reaction at Base, R, (lbs)	=	483.67



Number of bolt, N	=	4.9505	9.90092	9.90092	4.950458
Final number or bolt required, N	=	**5**	**10**	**10**	**5**
Spacing of bolt, S	=	**12**	**12**	**12**	**12**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	279.09	244.846
Number of bolt, N	=	12	11
Spacing of bolt, S	=	**20.727**	**22.8**

(assuming end bolts are 6-inches from end)

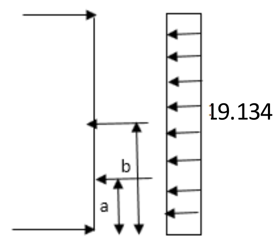
A.9.12 Spacing and number of 5/8" anchor bolts when stillwater elevation is "" ft and stillwater elevation difference in interior and exterior face of building is 1 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	158.12	183.552	185.959	116.103
Number of bolt, N	=	1.6184	3.75738	3.80665	1.188337

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	31.2
Dynamic flood load, $F_{a\ (dynamic)}$	=	485
h from base where static load is applied, a (ft)	=	3.3333
h from base where dynamic load is applied, b (ft)	=	2
Reaction at Base, R, (lbs)	=	504.47



Number of bolt, N	=	5.1633	10.3267	10.3267	5.16335
Final number or bolt required, N	=	**6**	**11**	**11**	**6**
Spacing of bolt, S	=	**9.6**	**10.8**	**10.8**	**9.6**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	287.74	249.614
Number of bolt, N	=	12	11
Spacing of bolt, S	=	**20.727**	**22.8**

(assuming end bolts are 6-inches from end)

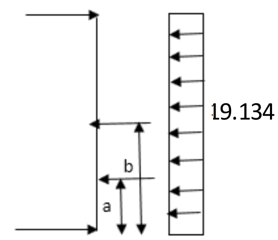
A.9.13 Spacing and number of 5/8” anchor bolts when stillwater elevation is 4 ft and stillwater elevation difference in interior and exterior face of building is 2 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	167.53	192.393	192.989	121.315
Number of bolt, N	=	1.7147	3.93836	3.95056	1.241683

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	124.8
Dynamic flood load, $F_{a\ (dynamic)}$	=	485
h from base where static load is applied, a (ft)	=	2.6667
h from base where dynamic load is applied, b (ft)	=	2
Reaction at Base, R, (lbs)	=	575.19



Number of bolt, N	=	5.8872	11.7744	11.7744	5.887183
Final number or bolt required, N	=	**6**	**12**	**12**	**6**
Spacing of bolt, S	=	**9.6**	**9.81818**	**9.81818**	**9.6**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	303.89	258.782
Number of bolt, N	=	13	11
Spacing of bolt, S	=	**19**	**22.8**

(assuming end bolts are 6-inches from end)

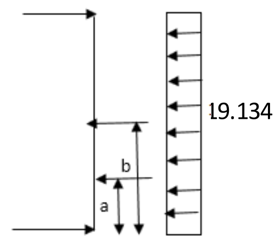
A.9.14 Spacing and number of 5/8” anchor bolts when stillwater elevation is 4 ft and stillwater elevation difference in interior and exterior face of building is 3 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	178.95	202.275	200.239	126.621
Number of bolt, N	=	1.8316	4.14065	4.09897	1.295991

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	280.8
Dynamic flood load, $F_{a\ (dynamic)}$	=	485
h from base where static load is applied, a (ft)	=	2
h from base where dynamic load is applied, b (ft)	=	2
Reaction at Base, R, (lbs)	=	708.31



		R1	R4	R7	R10
Number of bolt, N	=	7.2497	14.4994	14.4994	7.249692
Final number or bolt required, N	=	**8**	**15**	**15**	**8**
Spacing of bolt, S	=	**6.8571**	**7.71429**	**7.71429**	**6.857143**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	322.17	267.727
Number of bolt, N	=	14	11
Spacing of bolt, S	=	**17.538**	**22.8**

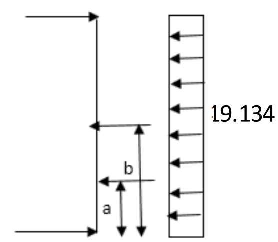
(assuming end bolts are 6-inches from end)

A.9.15 Spacing and number of 5/8” anchor bolts when stillwater elevation is 5 ft and stillwater elevation difference in interior and exterior face of building is 0 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	173.6	199.881	199.688	126.332
Number of bolt, N	=	1.7768	4.09164	4.08769	1.293033

B) Check for lateral force perpendicular to wall		
Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	0
Dynamic flood load, $F_{a\ (dynamic)}$	=	606.25
h from base where static load is applied, a (ft)	=	5
h from base where dynamic load is applied, b (ft)	=	2.5
Reaction at Base, R, (lbs)	=	550.36



Number of bolt, N	=	5.633	11.266	11.266	5.633017
Final number or bolt required, N	=	**6**	**12**	**12**	**6**
Spacing of bolt, S	=	**9.6**	**9.81818**	**9.81818**	**9.6**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	317.78	267.777
Number of bolt, N	=	14	11
Spacing of bolt, S	=	**17.538**	**22.8**

(assuming end bolts are 6-inches from end)

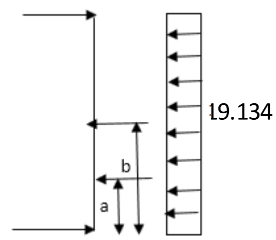
A.9.16 Spacing and number of 5/8” anchor bolts when stillwater elevation is 5 ft and stillwater elevation difference in interior and exterior face of building is 1 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	178.66	205.068	203.992	129.532
Number of bolt, N	=	1.8286	4.19782	4.1758	1.325785

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	31.2
Dynamic flood load, $F_{a\ (dynamic)}$	=	606.25
h from base where static load is applied, a (ft)	=	4.3333
h from base where dynamic load is applied, b (ft)	=	2.5
Reaction at Base, R, (lbs)	=	568.04



Number of bolt, N	=	5.814	11.628	11.628	5.813976
Final number or bolt required, N	=	**6**	**12**	**12**	**6**
Spacing of bolt, S	=	**9.6**	**9.81818**	**9.81818**	**9.6**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	327.63	273.584
Number of bolt, N	=	14	12
Spacing of bolt, S	=	**17.538**	**20.7273**

(assuming end bolts are 6-inches from end)

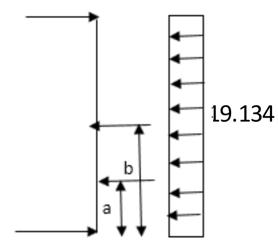
A.9.17 Spacing and number of 5/8” anchor bolts when stillwater elevation is 5 ft and stillwater elevation difference in interior and exterior face of building is 2 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	192	217.743	214.145	137.048
Number of bolt, N	=	1.9651	4.45728	4.38363	1.402713

B) Check for lateral force perpendicular to wall

Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	124.8
Dynamic flood load, $F_{a\ (dynamic)}$	=	606.25
h from base where static load is applied, a (ft)	=	3.6667
h from base where dynamic load is applied, b (ft)	=	2.5
Reaction at Base, R, (lbs)	=	629.4



Number of bolt, N	=	6.442	12.884	12.884	6.442007
Final number or bolt required, N	=	**7**	**13**	**13**	**7**
Spacing of bolt, S	=	**8**	**9**	**9**	**8**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	353.53	287.888
Number of bolt, N	=	15	12
Spacing of bolt, S	=	**16.286**	**20.7273**

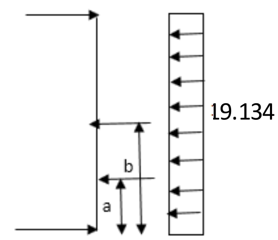
(assuming end bolts are 6-inches from end)

A.9.18 Spacing and number of 5/8” anchor bolts when stillwater elevation is 5 ft and stillwater elevation difference in interior and exterior face of building is 3 ft

Determination of 5/8" anchor bolt spacing for wall in grid 5

a) For Lateral force parallel to wall		R1	R4	R7	R10
Bolt lateral design value, Z' (lbs)	=	488.51			
Length of region of wall, b (ft)	=	5	10	10	5
Unit shear force acting on wall, v (lb/ft)	=	192	217.743	214.145	137.048
Number of bolt, N	=	1.9651	4.45728	4.38363	1.402713

B) Check for lateral force perpendicular to wall		
Wind lateral load perpendicular to wall, w_w (psf)	=	19.134
Static flood load, $F_{a\ (static)}$	=	124.8
Dynamic flood load, $F_{a\ (dynamic)}$	=	606.25
h from base where static load is applied, a (ft)	=	3.6667
h from base where dynamic load is applied, b (ft)	=	2.5
Reaction at Base, R, (lbs)	=	629.4



Number of bolt, N	=	6.442	12.884	12.884	6.442007
Final number or bolt required, N	=	**7**	**13**	**13**	**7**
Spacing of bolt, S	=	**8**	**9**	**9**	**8**

(assuming end bolts are 6-inches from end)

Determination of 5/8" anchor bolt spacing for wall in grid 3

a) For Lateral force parallel to wall		R1	R3
Bolt lateral design value, Z' (lbs)	=	488.51	
Length of region of wall, b (ft)	=	20	20
Unit shear force acting on wall, v (lb/ft)	=	353.53	287.888
Number of bolt, N	=	15	12
Spacing of bolt, S	=	**16.286**	**20.7273**

(assuming end bolts are 6-inches from end)

Appendix B

Description and load calculations- Group II (1 and 2 story)

B.1 Structural Framing plans

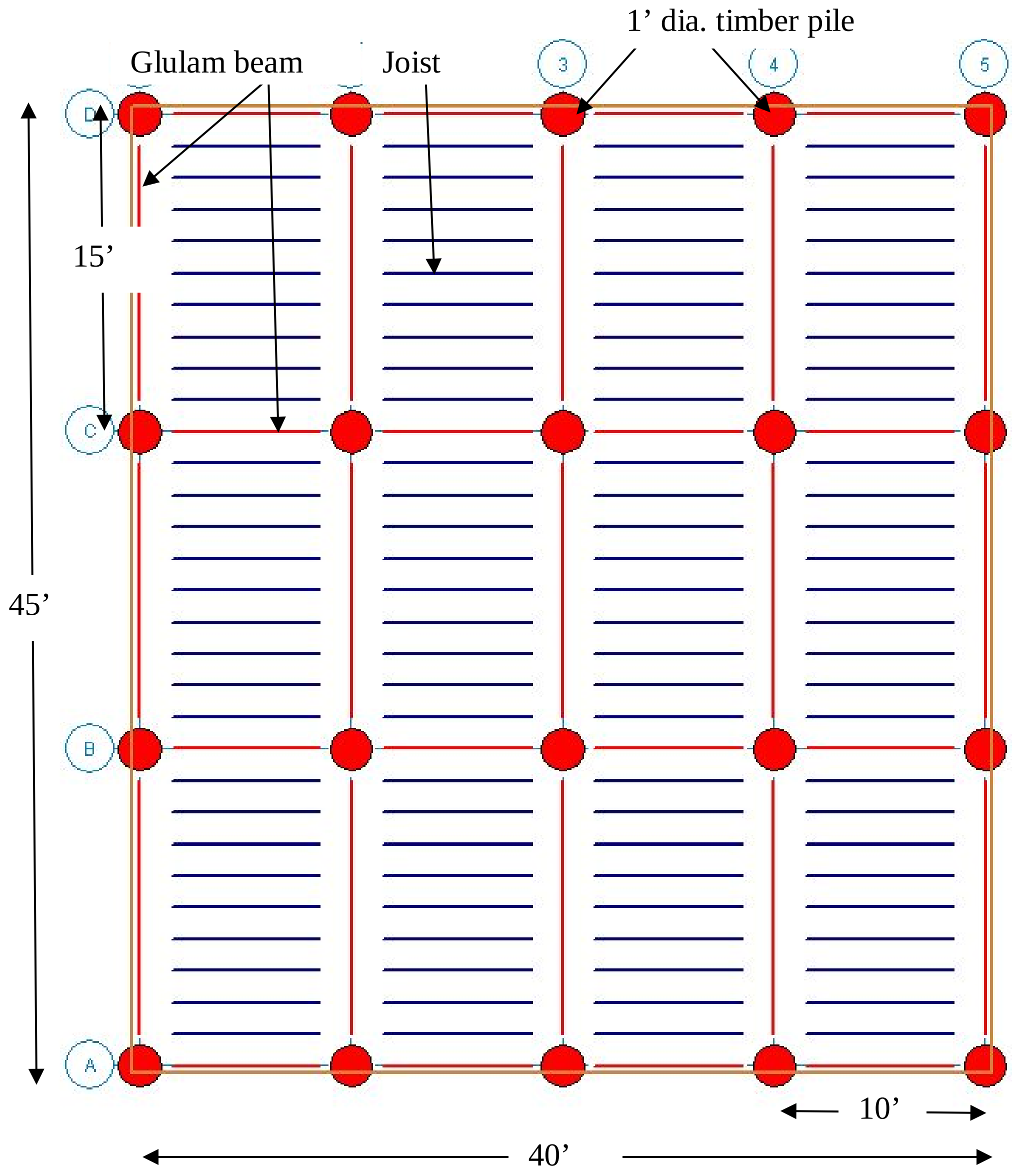


Figure B.1 Foundation plan (1 and 2 story building- elevated)

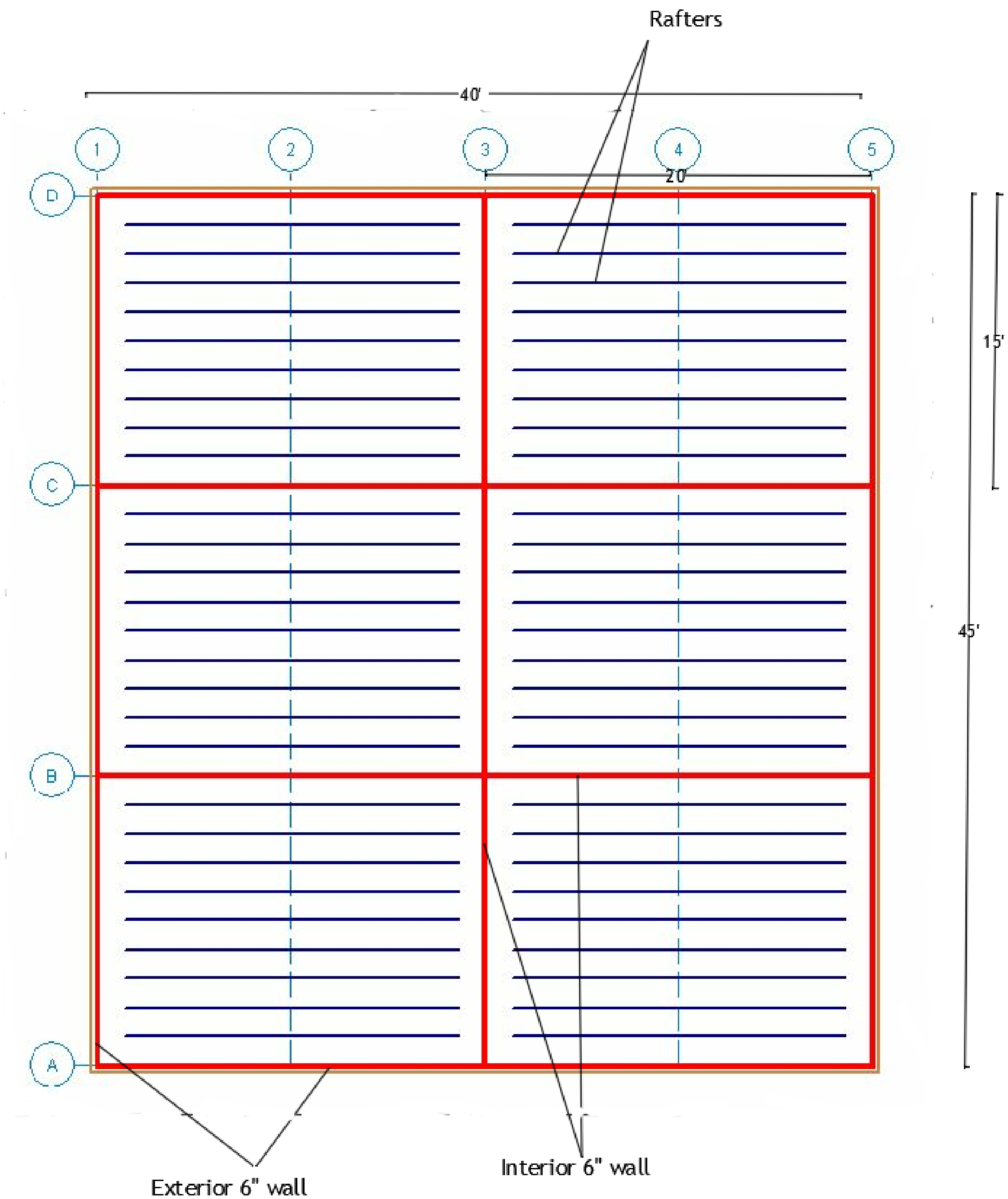


Figure B.2 Roof Flooring plan (1 story and 2 story-elevated)

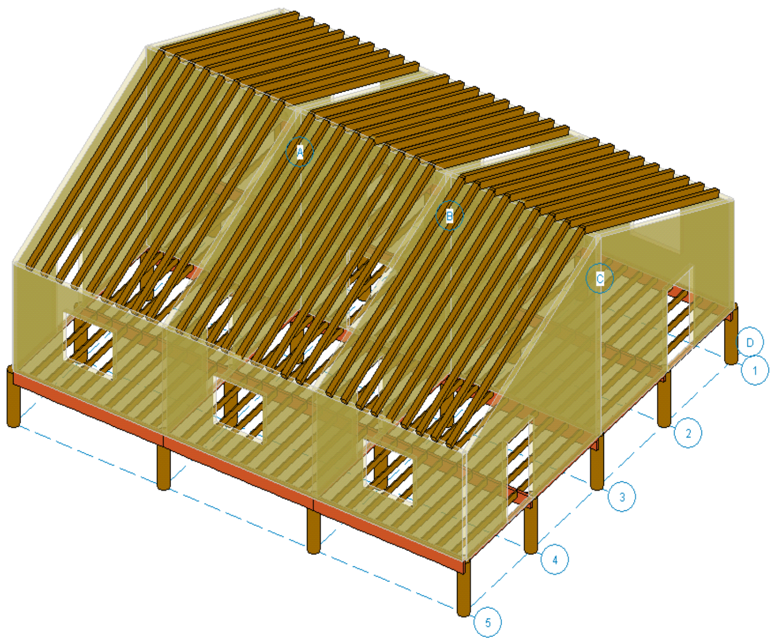


Figure B-3 Three-dimensional view of one story-elevated (obtained from Risa Floor 9.0)

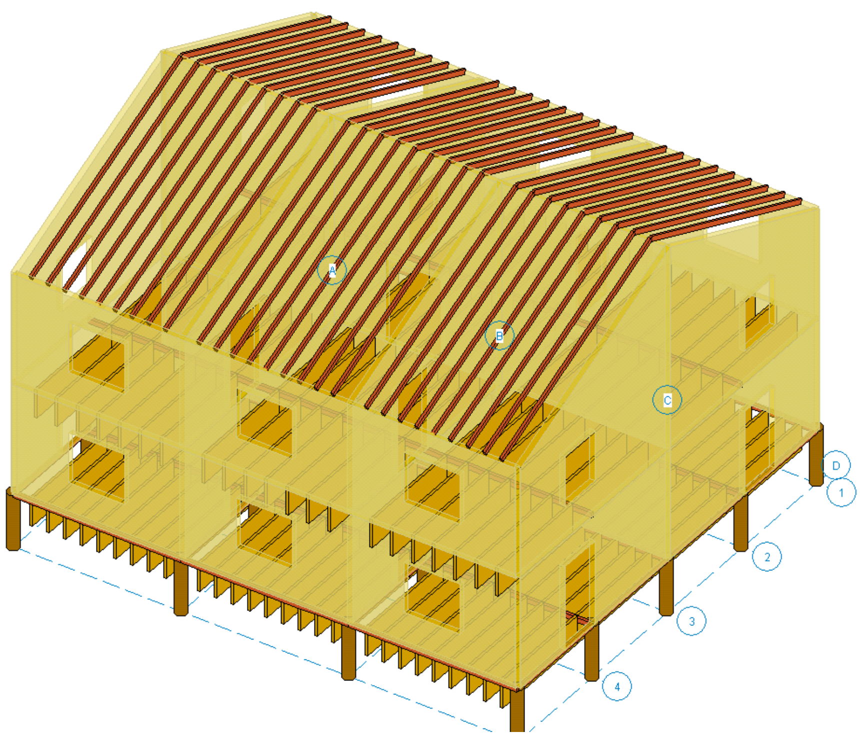


Figure B-4 Three-dimensional view of two story-elevated (obtained from Risa Floor 9.0)

B.2 Assumptions

When flood loads are applied on the elevated building located in non- coastal

Flood Zone A: (stillwater Flood depth ranging from 1 ft to 11 ft)

The load calculation for this modeling condition depends on various assumptions listed below.
a) The building is located on flood Zone A in non-Coastal Area and the load and it's combination as per Allowable Stress Design is obtained as per IBC 2015/ ASCE 7/10

b) Dead, Live, Roof live, Wind and Flood loads are obtained from IBC, 2015

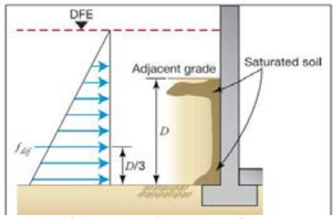
1. D (IBC 16.8)
2. D + L (IBC 16.9)
3. D + Lr (IBC 16.10)
4. D + 0.75L +0.75 Lr (IBC 16.11)
5. D + 0.6W + 1.5Fa (IBC 16.12)
6. D + 0.75L + 0.75 (0.6W) + 0.75Lr + 1.5Fa (IBC16.13)
7. 0.6D + 0.6W + 1.5Fa (IBC16.15)

c) Assumptions for Wind loading:

1. Risk Category: II
2. Basic Wind Speed (V): 150 mph (Assuming, Houston, Texas) (Table 26.5-1A ASCE 7/10)
3. Importance factor, I=1 (Category II building: 1 (ASCE 7/10))
4. Velocity pressure exposure coefficient evaluated at height z (K_z) (Table 27.3-1 ASCE 7/10)
5. Topographical Factor (K_{zt})= 1 (assuming a flat surface) (Table 26.8-1 ASCE 7/10)
6. Wind Directionality Factor (K_d)= 0.85 (Table 26.6-1 ASCE 7/10)
7. Exposure category= B (26.7.3 ASCE 7/10)

B.3 Flood load calculation:

B.3.1 For 1-story building, when stillwater Flood depth is 3 feet



a) Hydrostatic Load (F_h= $0.5P_hH$)

Source of Water = fresh water

Depth of flood, H = 3 ft

Difference in water level in interior and exterior side, H = 0 ft

Hydrostatic Pressure, ($P_h = \gamma H$) = 0 psf

F_h = 0 lb/ft

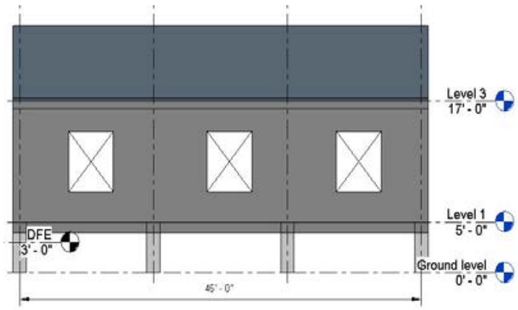


Fig. North Elevation of building

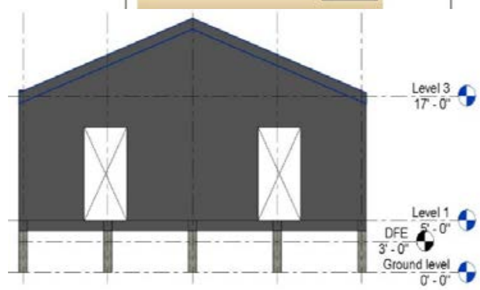
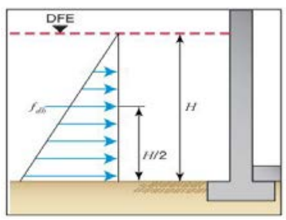


Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps) = 10

Width in X Direction = 40 ft

Drag coefficient (C_d) X direction = 1.3

Width in Z Direction = 45 ft

Drag coefficient (C_d) Z direction = 1.3

Hydrodynamic Pressure (P_d) = $C_d\rho.V^2/2$

Hydrodynamic Pressure (P_d) -X = 126.1 psf

Hydrodynamic Pressure (P_d) -Z = 129.35 psf

For Piles, =

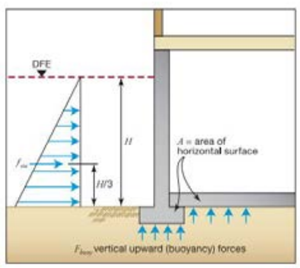
Width of piles in X Direction = 1 ft

Width of piles in Z Direction = 1 ft

Hydrodynamic Pressure (P_d) -X = 126.1 lb per pier

Hydrodynamic Pressure (P_d) -Z = 129.35 lb per pier

c) Hydrostatic buoyancy (F_{buoy}):



Depth of Pile below Ground Level = 15 ft

Uplift force due to buoyancy = 882.5143 lb

{Fbuoy = (*Vol*)γ_w} (per pile)

Difference in flood depth in interior and ex side above slab invert level (h) = 0 ft

Uplift force on slab {$P_{buoy} = h\gamma_w$} = 0 psf

d) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$

Weight of the object (lb), W = 1000 lb

Depth Coefficient, C_D = 0.5

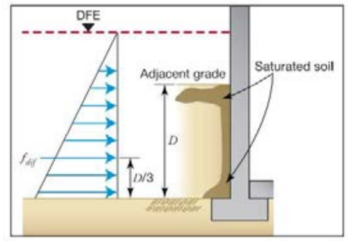
Blockage Coefficient, C_B = 1

Building Str. Coefficient, C_{Str} = 0.2

F_i = 1000 lb

B.3.2 For 1-story building, when stillwater Flood depth is 5 feet

a) Hydrostatic Load (F_h= $0.5P_hH$)



Source of Water	=	fresh water
Depth of flood, H	=	5 ft
Difference in water level in interior and exterior side, H	=	0 ft
Hydrostatic Pressure, ($P_h = \gamma H$)	=	0 psf
F_h	=	0 lb/ft

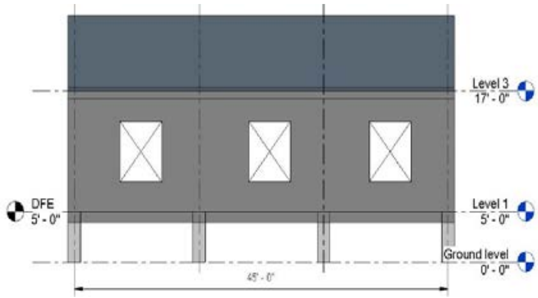


Fig. North Elevation of building

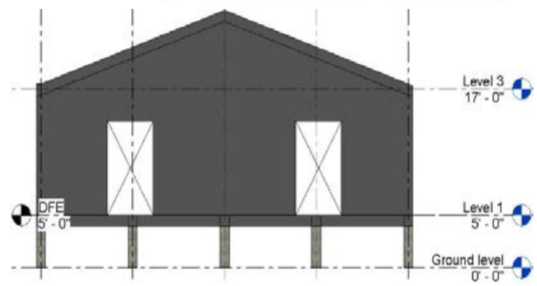
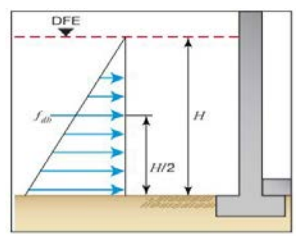


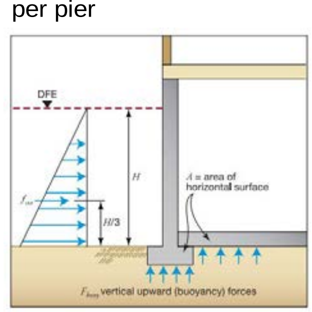
Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier

c) Hydrostatic buoyancy (F_{buoy}):



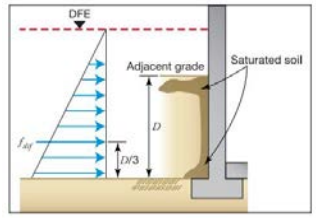
Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	0.2 ft
Uplift force on slab {$P_{buoy} = h\gamma_w$}	=	12.48 psf

d) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.3.3 For 1-story building, when stillwater Flood depth is 7 feet with maximum possible level difference of 3 ft

a) Hydrostatic Load (F_h= 0.5P_hH)



Source of Water	=	fresh water
Depth of flood, H	=	7 ft
Difference in water level in interior and exterior side, H	=	2 ft
Hydrostatic Pressure, (P_h= γH)	=	124.8 psf
F_h	=	124.8 lb/ft

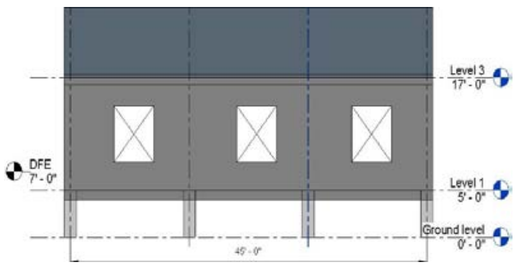


Fig. North Elevation of building

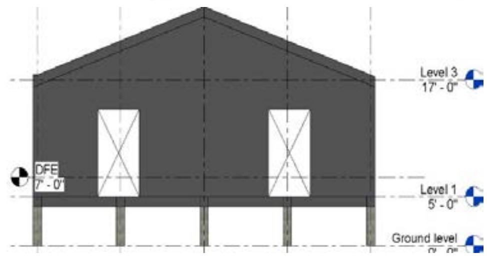
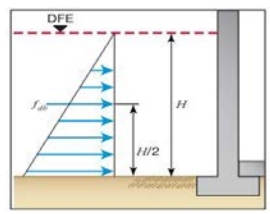


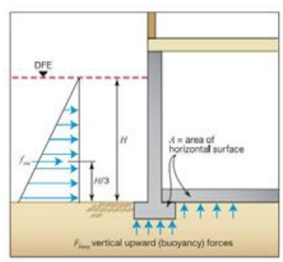
Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d ρ.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier

c) Hydrostatic buoyancy (F_{buoy}):



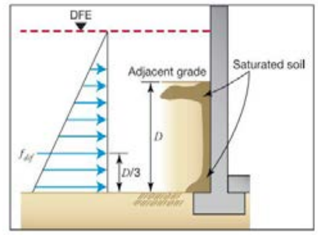
Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)$γ_w$}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	2 ft
Uplift force on slab {P_{buoy} = h$γ_w$}	=	124.8 psf

d) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.3.4 For 1-story building, when stillwater Flood depth is 9 feet with maximum possible level difference of 3 ft

a) Hydrostatic Load ($F_h = 0.5P_hH$)



Source of Water	=	fresh water
Depth of flood, H	=	9 ft
Difference in water level in interior and exterior side, H	=	3 ft
Hydrostatic Pressure, ($P_h = \gamma H$)	=	187.2 psf
F_h	=	280.8 lb/ft

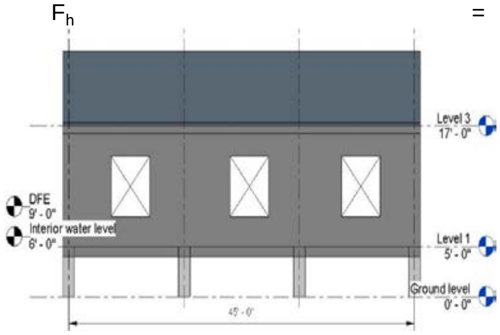


Fig. North Elevation of building

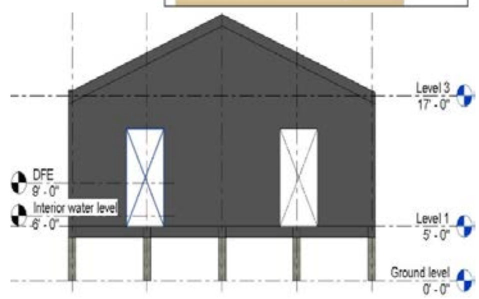
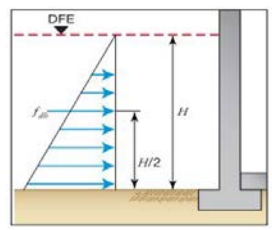


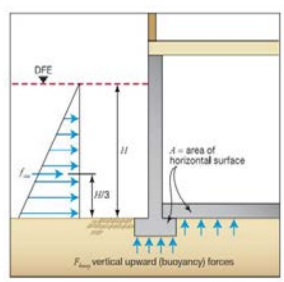
Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier

c) Hydrostatic buoyancy (F_{buoy}):



Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	3 ft
Uplift force on slab {$P_{buoy} = h\gamma_w$}	=	187.2 psf

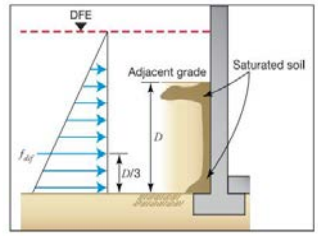
d) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$

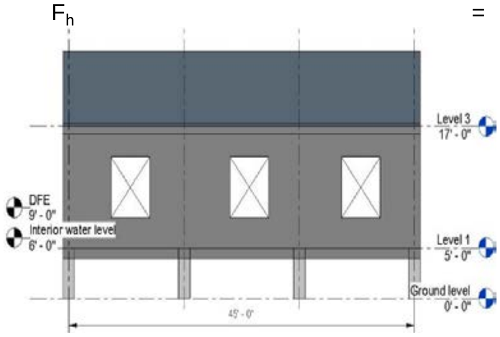
Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.3.5 For 1-story building, when stillwater Flood depth is 11 feet with maximum possible level difference of 3 ft

a) Hydrostatic Load ($F_h = 0.5P_hH$)

Source of Water	=	fresh water
Depth of flood, H	=	11 ft
Difference in water level in interior and exterior side, H	=	3 ft
Hydrostatic Pressure, ($P_h = \gamma H$)	=	187.2 psf
F_h	=	280.8 lb/ft





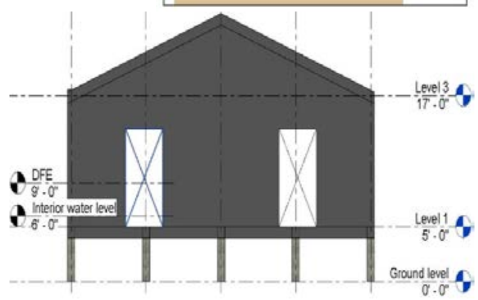
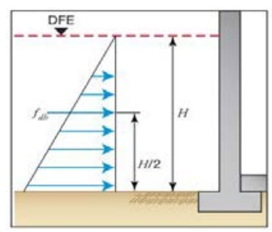


Fig. North Elevation of building

Fig. East Elevation of building

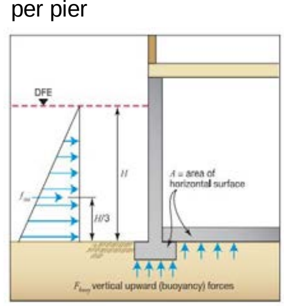
b) Hydrodynamic Load (F_h)

Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier



c) Hydrostatic buoyancy (F_{buoy}):

Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	3 ft
Uplift force on slab {$P_{buoy} = h\gamma_w$}	=	187.2 psf

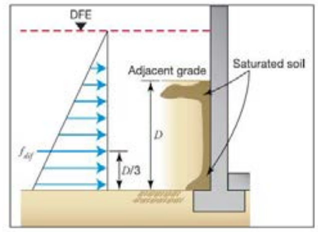


d) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.3.6 For 2-story building, when stillwater Flood depth is 3 feet

a) Hydrostatic Load (F_h= $0.5P_hH$)



Source of Water	=	fresh water
Depth of flood, H	=	3 ft
Difference in water level in interior and exterior side, H	=	0 ft
Hydrostatic Pressure, (P_h= γH)	=	0 psf
F_h	=	0 lb/ft

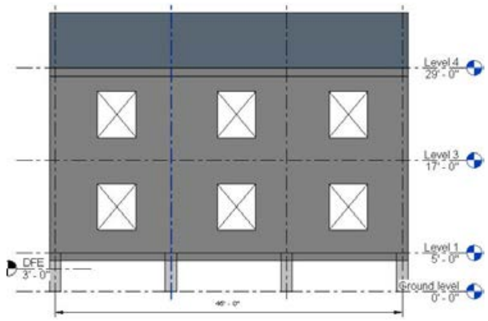


Fig. North Elevation of building

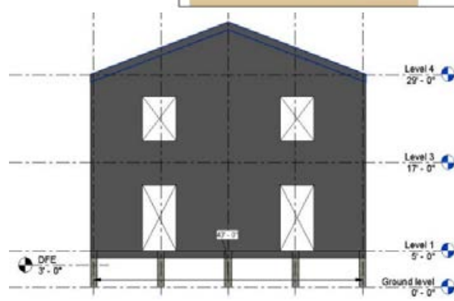
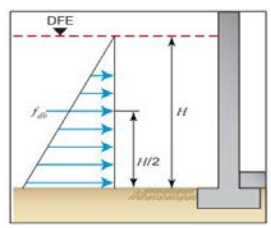


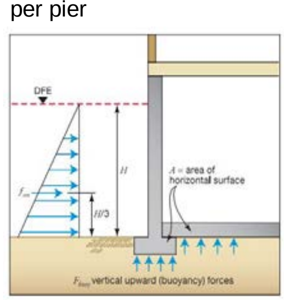
Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.3	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.3	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	126.1 psf	
Hydrodynamic Pressure (P_d) -Z	=	129.35 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	126.1 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	129.35 lb	per pier

c) Hydrostatic buoyancy (F_{buoy}):



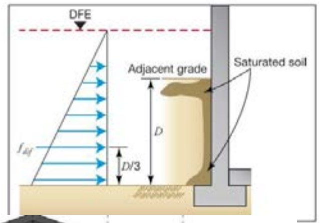
Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	882.5143 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	0 ft
Uplift force on slab {P_{buoy} = $h\gamma_w$}	=	0 psf

d) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	0.5
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	1000 lb

B.3.7 For 2-story building, when stillwater Flood depth is 5 feet

a) Hydrostatic Load (F_h= 0.5P_hH)



Source of Water	=	fresh water
Depth of flood, H	=	5 ft
Difference in water level in interior and exterior side, H	=	0 ft
Hydrostatic Pressure, (P_h= γH)	=	0 psf
F_h	=	0 lb/ft

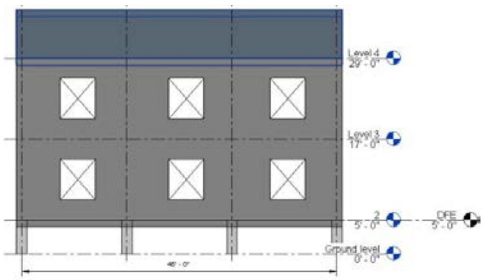


Fig. North Elevation of building

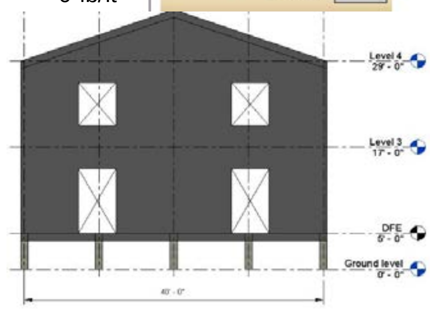
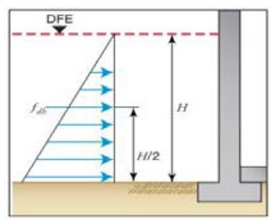


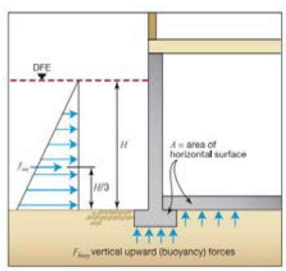
Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier

c) Hydrostatic buoyancy (F_{buoy}):



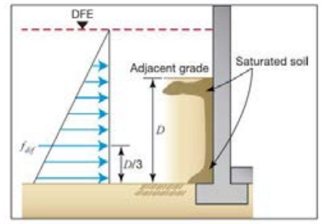
Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	0.2 ft
Uplift force on slab {P_{buoy} = hγ_w}	=	12.48 psf

d) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.3.8 For 2-story building, when stillwater Flood depth is 7 feet with maximum possible level difference of 3 ft

a) Hydrostatic Load (F_h= $0.5P_hH$)



Source of Water	=	fresh water
Depth of flood, H	=	7 ft
Difference in water level in interior and exterior side, H	=	2 ft
Hydrostatic Pressure, (P_h= γH)	=	124.8 psf
F_h	=	124.8 lb/ft

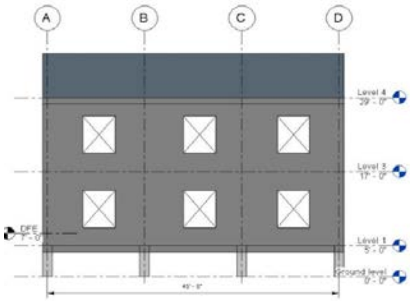


Fig. North Elevation of building

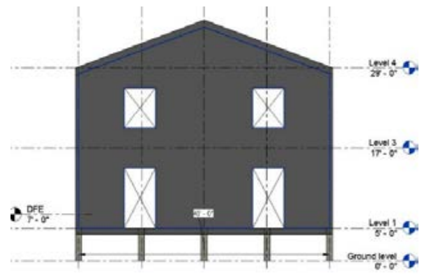
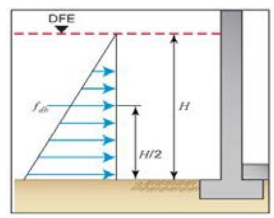


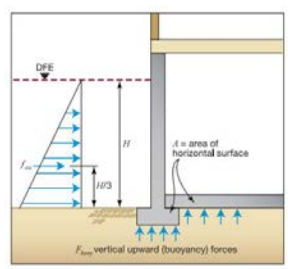
Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier

c) Hydrostatic buoyancy (F_{buoy}):



Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	2 ft
Uplift force on slab {P_{buoy} = hγ_w}	=	124.8 psf

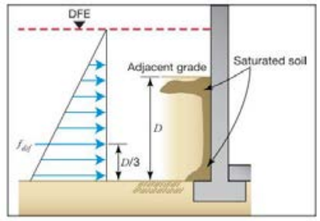
d) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.3.9 For 2-story building, when stillwater Flood depth is 9 feet with maximum possible level difference of 3 ft

a) Hydrostatic Load (F_h= 0.5P_hH)

Source of Water	=	fresh water
Depth of flood, H	=	9 ft
Difference in water level in interior and exterior side, H	=	3 ft
Hydrostatic Pressure, (P_h= γH)	=	187.2 psf
F_h	=	280.8 lb/ft



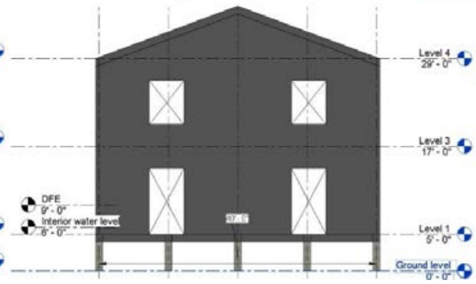
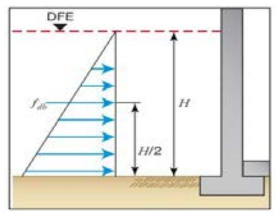


Fig. North Elevation of building

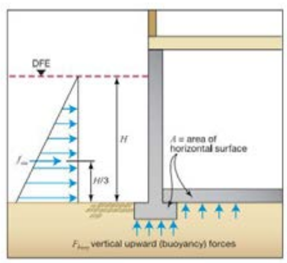
Fig. East Elevation of building

b) Hydrodynamic Load (F_h)



Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier

c) Hydrostatic buoyancy (F_{buoy}):



Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	3 ft
Uplift force on slab {P_{buoy} = hγ_w}	=	187.2 psf

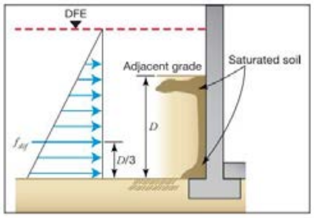
d) Debris Impact Loads (F_i): W.V.C_D.C_B.C_{Str}

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.3.10 For 2-story building, when stillwater Flood depth is 11 feet with maximum possible level difference of 3 ft

a) Hydrostatic Load ($F_h = 0.5P_hH$)

Source of Water	=	fresh water
Depth of flood, H	=	11 ft
Difference in water level in interior and exterior side, H	=	3 ft
Hydrostatic Pressure, ($P_h = \gamma H$)	=	187.2 psf
F_h	=	280.8 lb/ft



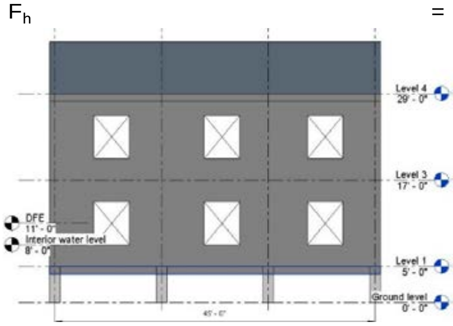


Fig. North Elevation of building

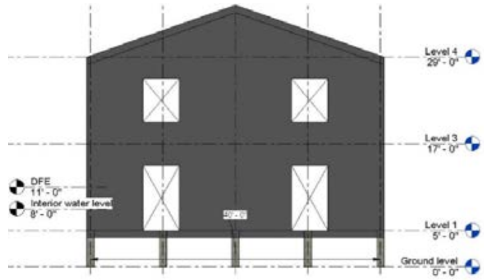
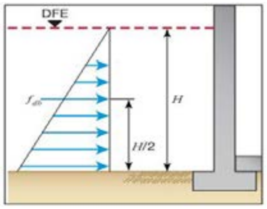


Fig. East Elevation of building

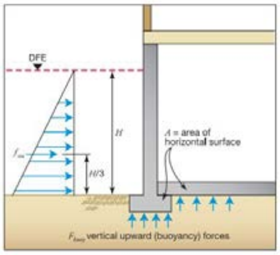
b) Hydrodynamic Load (F_h)

Velocity of flow (fps)	=	10	
Width in X Direction	=	40 ft	
Drag coefficient (C_d) X direction	=	1.25	
Width in Z Direction	=	45 ft	
Drag coefficient (C_d) Z direction	=	1.25	
Hydrodynamic Pressure (P_d)	=	$C_d\rho.V^2/2$	
Hydrodynamic Pressure (P_d) -X	=	121.25 psf	
Hydrodynamic Pressure (P_d) -Z	=	124.375 psf	
For Piles,	=		
Width of piles in X Direction	=	1 ft	
Width of piles in Z Direction	=	1 ft	
Hydrodynamic Pressure (P_d) -X	=	121.25 lb	per pier
Hydrodynamic Pressure (P_d) -Z	=	124.375 lb	per pier



c) Hydrostatic buoyancy (F_{buoy}):

Depth of Pile below Ground Level	=	15 ft
Uplift force due to buoyancy {Fbuoy = (*Vol*)γ_w}	=	980.5714 lb (per pile)
Difference in flood depth in interior and ex side above slab invert level (h)	=	3 ft
Uplift force on slab {$P_{buoy} = h\gamma_w$}	=	187.2 psf



d) Debris Impact Loads (F_i): $W.V.C_D.C_B.C_{Str}$

Weight of the object (lb), W	=	1000 lb
Depth Coefficient, C_D	=	1
Blockage Coefficient, C_B	=	1
Building Str. Coefficient, C_{Str}	=	0.2
F_i	=	2000 lb

B.4 Wind load calculation

Following assumptions are made to calculate wind load on the building based on ASCE (ASCE/SEI 7-10, 2010).

Table B.1 Assumptions made (1 and 2 story- elevated)

Details	Assumptions
Wind speed, V(mph)	150
Exposure Category	B
Topographic Factor K_1	0
Topographic Factor K_2	0
Topographic Factor K_3	0
Directionality Factor K_d	0.85

Table B.2 Detail results (1 story- elevated)

Details	Results
Exposure Constant, α	7
Exposure Constant, Z_g	1200
Gust effect factor, G	0.85
Topographical factor, K_{zt}	1
Mean roof height, h(ft)	19.167
Velocity pressure exposure coefficients (Table 27.3-1 ASCE 7/10), K_h	0.616
External pressure coefficient -Windward (Table 27.4-1 ASCE 7/10) C_p	0.8
Velocity pressure calculated using (Eq 27.3-1 ASCE 7/10) at mean roof height, q_h (psf)	30.18

Table B.3 Geometry results (1 story- elevated)

Floor level	Height (ft)	K_z	Width (X), ft	Width (Z), ft	Leeward C_p (X)	Leeward C_p (Z)
Roof	23.333	0.652	166.67 ft^2 (area)	0 ft^2 (area)	0.475	0.5
2nd floor	15	0.575	45.667	40.667	0.475	0.5
1st floor	5	0.575	45.667	40.667	0.475	0.5

Table B.4 Floor force results (1 story- elevated)

Floor level	Velocity pressure, q_z (psf)	Windward pressure, qGC_p (psf)	Leeward pressure X, q_iGC_{pi} (psf)	Leeward pressure Z, q_iGC_{pi} (psf)	Force X (lb)	Force Z (lb)
Roof	31.924	21.709	12.196	12.826	5650.679	0
2^{nd} floor	28.138	19.134	12.196	12.826	12021.02	7297.614
1^{st} floor	28.138	19.134	12.196	12.826	9555.512	10946.42

Table B.5 Detail results (2 story-elevated)

Details	Results
Exposure Constant, α	7
Exposure Constant, Z_g	1200
Gust effect factor, G	0.85
Topographical factor, K_{zt}	1
Mean roof height, h(ft)	29.167
Velocity pressure exposure coefficients (Table 27.3-1 ASCE 7/10), K_h	0.695
External pressure coefficient -Windward (Table 27.4-1 ASCE 7/10) C_p	0.8
Velocity pressure calculated using (Eq 27.3-1 ASCE 7/10) at mean roof height, q_h (psf)	34.026

Table B.6 Geometry results (2 story-elevated)

Floor level	Height (ft)	K_z	Width (X), ft	Width (Z), ft	Leeward C_p (X)	Leeward C_p (Z)
Roof	33.333	0.722	166.67 ft^2 (area)	0 ft^2 (area)	0.475	0.5
3^{rd} floor	25	0.665	45.667	40.667	0.475	0.5
2^{nd} floor	15	0.575	45.667	40.667	0.475	0.5
1^{st} floor	5	0.575	45.667	40.667	0.475	0.5

Table B.7 Floor force results (2 story-elevated)

Floor level	Velocity pressure, q_z (psf)	Windward pressure, qGC_p (psf)	Leeward pressure X, q_iGC_{pi} (psf)	Leeward pressure Z, q_iGC_{pi} (psf)	Force X (lb)	Force Z (lb)
Roof	35.349	24.037	13.75	14.461	6297.882	0
3rd floor	32.56	22.141	13.75	14.461	13595.629	8357.401
2nd floor	28.138	19.134	13.75	14.461	13372.772	15341.746
1st floor	28.138	19.134	13.75	14.461	10029.579	11506.309

B.5 Gravity load calculation

Table B.8 Gravity load calculation (1 story and 2 story- elevated)

Floor	Post Dead load (psf)	Live load (psf)
Roof	10	20 (Roof LL)
2nd floor (only for 2 story)	10	40 (Reducible, LL)
1st floor	10	40 (Reducible, LL)

B.6 Overturning Moment Calculation

B.6.1 Overturning Moment calculation for 1 story building

Stillwater depth, ft (Direction)	Moment per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75Fa - 0.6D < 0 to avoid overturning			
	0.6D + Fa uplift	0.6W (Lateral)	0.75Fa	Total
3 (x)	18619.2	3421.3929	436.55625	-14761.2509
3 (z)	14711.46667	2189.284133	388.05	-12134.1325
5 (x)	8314.734375	3421.3929	1166.015625	-3727.32585
5 (z)	6569.666667	2189.284133	1036.458333	-3343.9242
7 (x)	-76978.26563	3421.3929	4465.565625	84865.2242
7 (z)	-60822.33333	2189.284133	4336.008333	67347.6258
9 (x)	-124363.2656	3421.3929	9338.165625	137122.824
9 (z)	-98262.33333	2189.284133	9208.608333	109660.226
11 (x)	-124363.2656	3421.3929	13911.81563	141696.474
11 (z)	-98262.33333	2189.284133	13782.25833	114233.876

B.6.2 Overturning Moment calculation for 2 story building

Stillwater depth, ft (Direction)	Moment per linear foot in Respective- Direction (lb-ft) 0.6W + 0.75Fa - 0.6D < 0 to avoid overturning			
	0.6D + Fa uplift	0.6W (Lateral)	0.75Fa	Total
3 (x)	44523.45563	8859.453	218.278125	-35445.7245
3 (z)	35179.02667	6621.8368	194.025	-28363.1649
5 (x)	34218.99	8859.453	583.0078125	-24776.5292
5 (z)	27037.22667	6621.8368	518.2291667	-19897.1607
7 (x)	-51074.01	8859.453	2232.782813	62166.2458
7 (z)	-40354.77333	6621.8368	2168.004167	49144.6143
9 (x)	-98459.01	8859.453	4669.082813	111987.546
9 (z)	-77794.77333	6621.8368	4604.304167	89020.9143
11 (x)	-98459.01	8859.453	6955.907813	114274.371
11 (z)	-77794.77333	6621.8368	6891.129167	91307.7393

B.7 Sliding Forces Calculation

B.7.1 Sliding Forces calculation for 1 story building

Stillwater depth, ft (Direction)	Load per linear foot in respective Direction (lb) (-)0.45(0.6D+Fy) + 0.6W + 0.75Fa <0 (safe)			
	(-)0.45(0.6D+Fy)	0.6W	0.75Fa	Total
3 (x)	-372.384	323.64798	141.8625	93.12648
3 (z)	-331.008	243.2537867	129.35	41.59578667
5 (x)	-166.2946875	323.64798	227.34375	384.6970425
5 (z)	-147.8175	243.2537867	207.2916667	302.7279533
7 (x)	1539.565313	323.64798	502.81875	2366.032043
7 (z)	1368.5025	243.2537867	487.4541667	2099.210453
9 (x)	2487.265313	323.64798	801.69375	3612.607043
9 (z)	2210.9025	243.2537867	791.0166667	3245.172953
11 (x)	2487.265313	323.64798	983.56875	3794.482043
11 (z)	2210.9025	243.2537867	977.5791667	3431.735453

B.7.2 Sliding Forces calculation for 2 story building

Stillwater depth, ft (Direction)	Load per linear foot in respective Direction (lb) (-)0.45(0.6D+Fy) + 0.6W + 0.75Fa <0 (safe)			
	(-)0.45(0.6D+Fy)	0.6W	0.75Fa	Total
3 (x)	-890.4691125	554.9697	141.8625	-193.6369125
3 (z)	-791.5281	469.44608	129.35	-192.73202
5 (x)	-684.3798	554.9697	227.34375	97.93365
5 (z)	-608.3376	469.44608	207.2916667	68.40014667
7 (x)	1021.4802	554.9697	502.81875	2079.26865
7 (z)	907.9824	469.44608	487.4541667	1864.882647
9 (x)	1969.1802	554.9697	801.69375	3325.84365
9 (z)	1750.3824	469.44608	791.0166667	3010.845147
11 (x)	1969.1802	554.9697	983.56875	3507.71865
11 (z)	1750.3824	469.44608	977.5791667	3197.407647

B.8 Uplift Forces calculation

B.8.1 Uplift forces calculation for 1 story building

Stillwater depth, ft (Direction)	Load per linear foot in Respective- Direction (lb) 0.6W + 0.75 Fa +0.6D > 0 to avoid uplift			
	0.6D (+ve)	0.6W (uplift)	0.75Fa (uplift)	Total
3 (x)	1158.4575	-331.25775	-330.9375	496.26225
3 (z)	1029.74	-294.451333	-294.166667	441.122
5 (x)	1158.4575	-331.25775	-788.91375	38.286
5 (z)	1029.74	-294.451333	-701.256667	34.032
7 (x)	1158.4575	-331.25775	-4579.71375	-3752.514
7 (z)	1029.74	-294.451333	-4070.85667	-3335.568
9 (x)	1158.4575	-331.25775	-6685.71375	-5858.514
9 (z)	1029.74	-294.451333	-5942.85667	-5207.568
11 (x)	1158.4575	-331.25775	-6685.71375	-5858.514
11 (z)	1029.74	-294.451333	-5942.85667	-5207.568

B.8.2 Uplift forces calculation for 2 story building

Stillwater depth, ft (Direction)	Load per linear foot in Respective- Direction (lb) 0.6W + 0.75 Fa +0.6D > 0 to avoid uplift			
	0.6D (+ve)	0.6W (uplift)	0.75Fa (uplift)	Total
3 (x)	2309.75775	-468.53745	-330.9375	1510.2828
3 (z)	2053.118	-416.477733	-294.166667	1342.4736
5 (x)	2309.75775	-468.53745	-788.91375	1052.30655
5 (z)	2053.118	-416.477733	-701.256667	935.3836
7 (x)	2309.75775	-468.53745	-4579.71375	-2738.49345
7 (z)	2053.118	-416.477733	-4070.85667	-2434.2164
9 (x)	2309.75775	-468.53745	-6685.71375	-4844.49345
9 (z)	2053.118	-416.477733	-5942.85667	-4306.2164
11 (x)	2309.75775	-468.53745	-6685.71375	-4844.49345
11 (z)	2053.118	-416.477733	-5942.85667	-4306.2164

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