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ADEQUACY OF EXISTING LIGHT TIMBER SHEAR WALLS TO BRACE STEEL FRAMES IN RENOVATION CONSTRUCTION

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ADEQUACY OF EXISTING LIGHT TIMBER SHEAR WALLS TO BRACE STEEL FRAMES IN RENOVATION CONSTRUCTION

by

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Abstract

ADEQUACY OF EXISTING LIGHT TIMBER SHEAR WALLS TO BRACE STEEL FRAMES IN RENOVATION CONSTRUCTION

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A lateral load resisting system (LLRS) is an interconnected framework in the building that works together to resist the lateral loads, both wind and seismic, acting on the building. A shear wall is a part of the LLRS which transfers the lateral load to the foundation of the building. Currently, the provision from IBC and ASCE 7-16 suggests the selection of adequate stiffness, dimension, and type of shear walls based on the given lateral loads. There has been a rise in the amount of building renovations since the 1980's. In 2019, expenditures in home improvements were around $262 billion with $17 billion spent in major structural work. These numbers have increased by roughly 3% in 2022. One common type of renovation is to open a floorplan by removing an interior load bearing wall and replacing it with a steel frame structure. This frame will require adequate lateral stability to be provided by the LLRS of the building structure. Currently, the AISC Steel Construction design code requires a certain amount of lateral stiffness based on the geometry of the frame and the gravity loads being carried by the frame. However, many times it is just assumed that the LLRS of the building provides this required stiffness without it being

checked. This thesis will examine the adequacy of light framed timber shear walls, designed in accordance with IBC specifications based on wind loading, to provide adequate stiffness, based on AISC specifications, to a steel frame structure used in a common retrofitting of a building. Variables related to building location which impacts gravity loads (e.g., dead load, live load and snow load) and lateral loads (e.g. wind), building size and tributary area of the steel frame were used to determine if shear wall design based on the IBC code is adequate for lateral stiffness of the steel frame in all cases. The numerical analysis showed that for each of the cases analyzed in this thesis that the timber shear walls designed in accordance with the IBC under a given wind loading will provide the required lateral stiffness to an interior steel frame used in renovations of a structure.

1.0 Introduction

Lateral Load Resisting System (LLRS) is that part of the building that resists the lateral force created by wind or earthquake. When walls, roof elements and flooring systems are constructed and behave in a way they resist these lateral forces, they are referred to as diaphragms. Shear walls are vertical diaphragms that transfer the lateral forces from horizontal diaphragms (e.g., floor system) to the foundation. They also act as structural subassembly that transfers the horizontal loads of the building to the foundation in the form of an overturning moment. These shear walls provide lateral resistance to the structure and are necessary when internal structural systems, like a steel frame, require lateral support from the larger building structure. It is common for building structures to undergo some form of renovation in their lifetime. One type of renovation is to remove a load-bearing wall and replace it with a steel frame. This frame requires a certain amount of lateral stiffness per American Institute of Steel Construction (AISC) specifications based on the geometry of the frame and gravity load the frame is supporting. Shear walls designed to meet the standards of the International Building Code (IBC) are relied on for providing this stiffness requirement.

1.1 Problem Statements and Objectives

Renovations of existing buildings are very popular. At the residential level, the expenditures in home improvements have jumped from $146 billion in 1995 to $262 billion in 2019 (Harvard Joint Center for Housing Studies, 2021) . This 2019 number was estimated to increase roughly 3% in 2022. The COVID-19 pandemic has triggered the remodeling industry as many owners have shifted to work from home and changes in lifestyle has urged the need to bring changes to their home conditions. Around $17 billion dollars was spent in 2019 in remodeling of residential houses that included major structural changes. These included addition and removal and/or relocation of kitchens, bathrooms, and bedrooms as well as load bearing and partition walls, beams, and columns (Improving America's Housing, 2021). Whenever renovation work is done, it is important to analyze the effect of the changes on the remaining structure in addition to any new load bearing elements. It is also necessary to understand how the existing structural system supports the new structural elements. For example, a steel frame that is used to support gravity loads of a removed load bearing wall requires a certain amount of lateral stiffness to be provided by the LLRS (e.g., shear walls). Often when these frames are designed and installed it is assumed that the existing shear walls of the structure will provide adequate lateral stiffness for the frame structure, but this is not checked.

This research investigates the adequacy of timber framed shear walls, designed to meet the standards of the IBC for lateral loading, to provide the required lateral stiffness of a steel frame per AISC Steel Construction specifications used in a remodel of a building. The variables tested in this research include building geometry and the wind and gravity loads. Changing the building geometry allowed for the tributary area of the frame as well as the shear walls to differ. Additionally, the location of the building was investigated due to changing wind loads and snow loads in differing parts of the country. Through all the numerical calculations performed in this research it was determined that timber framed shear walls designed to meet IBC standards provide the required stiffness for a steel frame that would be used within the building structure.

Organization of Thesis

The organization of the Thesis is as follows:

Chapter Two- Background:

It describes the history of wind load calculation, use of light frame shear walls, the theory behind the wind and the building interaction and the existing knowledge and research regarding the shear wall properties.

Chapter Three- Renovation, Rehabilitation and Remodeling in United States:

It describes the history of renovation and building codes in United States, the common challenges in renovation and its solutions.

Chapter Four- Lateral Load Resisting System:

It describes the theory behind lateral load resisting system and its components.

Chapter Five- Calculations:

The process of calculating wind loads and gravity loads are discussed.

Chapter Six- Parametric Analysis:

The matrix of cases where varying wind loads, gravity loads and building dimension is presented with results, conclusions, and design recommendations.

Chapter Seven- Conclusion:

It summarizes the results obtained, conclusions, design recommendations and future work.

2.0 Background

2.1 History of wind load calculation

It was not until the 20th century that the proper consideration of the wind loads on the buildings were classified as lateral loads. San Francisco published the first ever building code after the major earthquake of 1906. This required all the buildings to be designed for a wind load of 30psf. This was mandated to provide the least amount of lateral strength for the Bay Area buildings. Throughout the first half of the 20th century the wind design on buildings was at its infancy and structural engineers had a small understanding of this area. William J. LeMessurier, who is considered as one of the prominent American structural engineers of that time strongly criticized the negligence of lateral stability check in buildings design. This was based upon the fact how the Boston Building Code 1938 edition required 10psf design wind load for first 40 feet building height, 15psf for next 40 feet height and 20psf beyond that while The City of New York had zero wind for the height of building above 100 feet. Later, in the 1950s, LeMessurier was involved in other major revisions of the Boston Building Code which increases the load requirements.

It is considered that the modern design of a structure for wind load was started with the implementation of American National Standards Institute (ANSI) A58.1-1972. This provision acted as a quantum jump in terms of complications in comparison with the code existent at that time. But these provisions had major fault in a way it was full of ambiguities and inconsistent in the format and terminologies that resulted in misinterpretation. When it was revised in 1982 with the implementation of ANSI A58.1-1982, it was able to clearly define the wind forces accordingly with the cladding and components of the buildings. The specification was able to consider the aerodynamics of the wind pressure around the building corners, ridges, and eaves as well as the effect of average area on the wind pressure. Since it was less ambiguous, the model code organizations began to adopt it.

In 1988, the authority to update ANSI A58.1 was taken over by American Society of Civil Engineer's (ASCE). Minor changes were made in the provisions of ANSI A58.1-1982, which was included in the minimum load standards of ASCE 7-88. While in ASCE 7-93, a hurricane-coast importance factor of 1.05 was included to account for the wind speed at coastal regions and the wind-speed maps were also upgraded. In 1995, the first major changes to wind provisions were made. The "three-second-gust" wind speed was used instead of "fastest-mile" wind speed. The "fastest-mile" speed was the time it takes for a mile of wind to pass through a measuring instrument called an anemometer. Usually, it took around 30-60 seconds. In the later method, "three-second-gust" this time was changed to 3 seconds. This resulted in an increase in average wind velocity from 70 miles per hour to 90 miles per hour. To prevent the design from being of such increased wind velocity there had to be major changes in the velocity pressure exposure coefficients, gust effect factors, internal and external pressure coefficients. Topographic factor was also included in this provision to incorporate the wind speed-up over isolated hills and escarpments. To include the wind induced torsion in all buildings with mean roof height greater than 60ft, new provisions were added for the full and partial loading on the main wind force resisting system (MWFRS). Also, an analytical procedure was included to determine the external loads on the MWFRS of the low-rise buildings.

ASCE 7-98 updated the wind-speed map based on new analysis of hurricane wind speeds. Wind-directionality factor, K_d was introduced in this provision to include the directionality of the wind. Directionality indicated the fact how seldom the wind ever hits the critical direction of the building, In fact it cannot, as wind direction changes from one instant to another. This was previously considered by a low load factor of 1.3 but it was pointed that allowable stress design could not take advantage of directionality of the wind hence a load factor of (1.3/0.85(K_d)) =1.53 was introduced, increasing the wind load factor by 5%. To allow the shorelines in the hurricane prone regions to be classified as exposures C and D, there were minor changes in the definitions of exposure C and D. In this edition the wind design provisions were organized by the method of design: method 1- simplified procedure, method 2- analytical procedure and method 3- wind tunnel procedure. The "wind-borne debris region" was also defined in ASCE 7-98, where the lower 60ft of category II, III or IV buildings that were in the wind-borne debris regions were advised to be covered with impact resistant covering such as glazing that receives positive external pressure.

In ASCE 7-02, the simplified design procedure, method 1 of ASCE 7-98 was discarded and a simplified design procedure in section 1609.6 of the International Building Code (IBC) 2000 was adopted. This provision defined that a ground surface roughness within each 45-degree sector to be determined for a distance upwind of the site. Three roughness categories were defined as surface roughness category A, B and C. They were differentiated based on location (urban/suburban), Terrain features like flat, open country, grandstands, and hurricane prone areas. In this provision, wind-induced torsion was made mandatory for all buildings even below 60 feet of height. To require the roof to be designed for zero or a positive pressure a roof pressure coefficient of 0.18 was added for the windward roof compared to a high suction coefficient being provided before.

In ASCE 7-05, new pressure coefficients were provided for the mono slope, pitched and troughed roofs. For the roof top equipment, walls and for free-standing signs the force coefficients were added. In wind-borne debris region, debris resistant glazing in the lower 60 feet of the building was made mandatory.

In summary, the basic analytical procedure initiated in ANSI A58.1-1982 has remained the same for the last 25 years (Mehta, 2010), but the format of the provisions has changed over the time. There has been addition of simplified procedures and wind tunnel procedures and wide range of pressure and force coefficients for the various shapes of the buildings and structures.

2.2 Light Frame Shear Wall

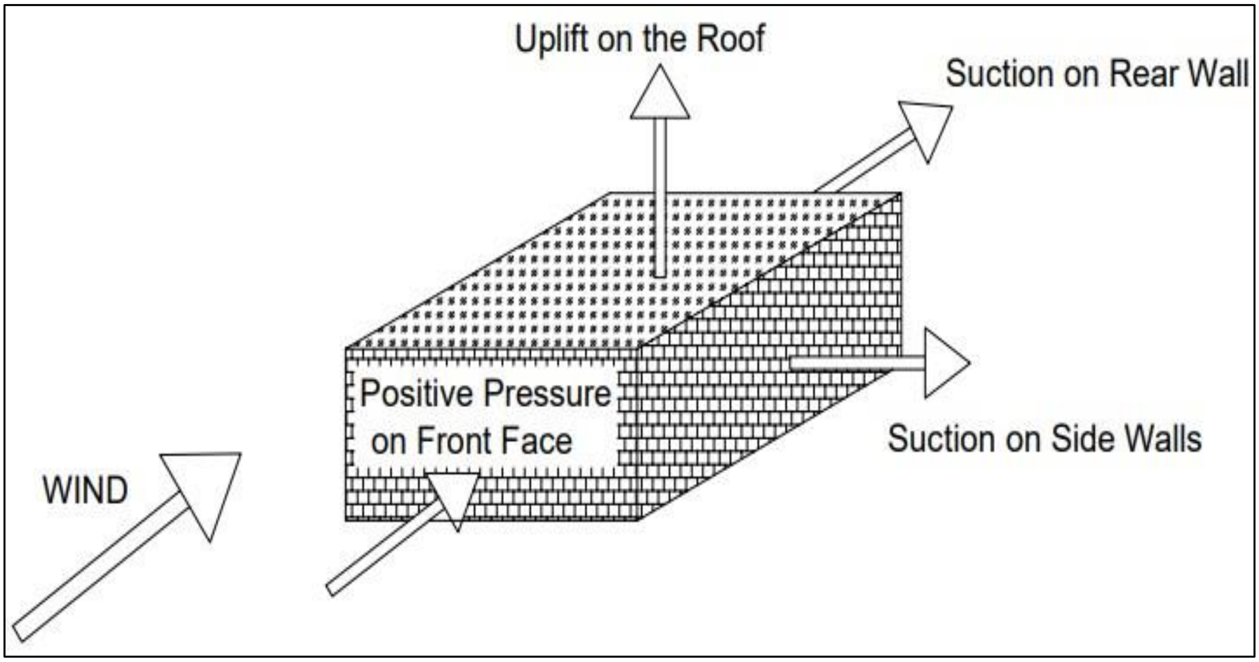
Wooden light frame buildings are the major construction type for single family and low to mid rise multi-family residential apartment buildings in North America. Wood is orthotropic material which means the stiffness and strength of the wood changes along the orthogonal direction (Beyer, Fridle, Cobeen, & Pollock, 2007). The light frame wooden buildings (LFWB) are composed of shear walls, floors, diaphragms, roof trusses, foundations, and connections (metal fasteners like nails or staples). There are two most popular types of LFWB construction designs: Standard shear walls and Midply shear walls. The Midply shear walls have sheathing placed on two sides of the midline framing while Standard shear walls have single sheathing on one or both sides of the walls. We are concerned only about the standard shear walls in this research which have studs and plates on top and bottom of the wall frame which supports the vertical loads and sheathing that transfers the lateral loads.

2.3 Interaction of Wind with Building

Whenever there is an interaction of building and wind load, there are two types of forces acting on it. There are both positive and negative pressures acting on the walls and roof of the building as shown in figure 1 below. The Windward Wall, which is the wall on which the wind is directly acting on, the pressure is considered as positive pressure. The Leeward Wall, opposite the wind direction and the Sidewalls are considered to have negative pressure or suction. Additionally, depending on the geometry of the roof there may be either downward or uplift pressure. A case of Uplift pressure on the roof is shown in figure 1.

Figure 1

Interaction of Wind with Building (American Society of Civil Engineers, 2017)



Hence, the shear walls and roof diaphragms must have enough strength to resist the applied loads to prevent lateral failure. These loads applied on the external envelope of the building are transferred to the lateral load resisting framework of the building which is finally transferred to the foundation of the structure.

Based on the exposure categories for where the building is located, the basic wind speed, topography of the location, the height of the building and internal pressure, building shape are the factors on which the stability of the LLRS is dependent. ASCE 7-16 codes has formulated these factors into mathematical values and has provided conditions on which each of them is used. Based on these codes, the interaction of the wind and the building is analyzed and better understood.

2.4 Literature Review

2.4.1 Wood Shear Wall Testing for Experimental Values Of IBC

There has been a substantial number of experimental tests conducted to determine the shear strength of wooden shear walls. The tests covered under this section were performed between 1980 to 2001. These tests are quantifying the static and dynamic characteristics to discuss the performance and behavior of shear walls.

2.4.1.1 Influence Of Dimension and Loading Condition on The Strength of Shear Walls

Experimental study was conducted by (Nelson, Wheat, & Fowler, 1985) to investigate the behavior of seven shear wall assemblies that are usually used in manufactured houses. The size and location of the shear wall within the combination, the number of glued sides of paneling and number of floor joists beneath the shear walls were taken as variables. It was found out that the walls on the windward side of the assembly had higher ultimate strengths and the connection of shear wall to the floor on windward side was typically the location of failure. The occurrence of slip between shear wall and side wall indicated that the side walls were also involved in the load transfer.

In an experiment conducted by (Patton-Mallory, Wolfe, Soltis, & Gutkowski, 1985) where 11 full size walls(three 8×8 ft, two 8×16 ft, six 8× 24 ft) and 200 small size walls(22 in.× 2 in., 4 ft, 6 ft and 8 ft) were tested under monotonic loading condition, it was found out that the racking strength was linearly proportional to the length of the wall. The stiffness of small walls was increased linearly with the wall length while the stiffness of full-size walls was increased nonlinearly with wall length. It was also found out that the sheathing layers and panel sections were contributing to increasing the racking resistance.

Investigation was continued by (Lam, Prion, & He, 1997) to determine the effect of oversized panels on the shear resistance of wooden shear walls. The OSB and plywood that were considered as the sheathing materials had dimensions of 4× 8 ft as standard size while the oversized ones were 8 ft in one dimension and multiple of 4 ft as the other. It was observed that in case of monotonic loading the walls that were sheathed with oversized panels had a significant increase in stiffness and higher lateral load carrying capacity. While in case of cyclic loading, the standard walls were dissipating more energy than the oversized walls.

In a test conducted by (Durham, Lam, & Prion, 2001), 12 shear walls were tested with both standard and oversized OSB panels. It was found that there was a 26% increase in shear capacity when the large sheathing panels were used. When the redistribution of the nails was done from interior to exterior, there was 104% increase in shear capacity. There was a 25% reduction on drifting walls with oversized panels on carrying out the shake table test.

2.4.1.2 Influence Of Fastener Type and Sheathing Material on The Wall Performance

Tests were conducted on ten 16 x 48 ft wood- framed diaphragms sheathed with particleboard by (Atherton, 1983). The main purpose of this test was to study the effect of sheathing material thickness, nail size, nail spacing, blocking, and sheathing pattern on the strength of the wooden diaphragms. It was concluded that the increased nail spacing had the most major effect on the strength of the wall while blocking had a small effect.

In a test conducted by (Schmid, Nielsen, & Linderman, 1994), three shear wall panels were tested with a height-to-width ratio of two-to-one under reversed cyclic pseudo-static loads. It was concluded that the tie-down anchors with proper bolt tightening sequence and torque were required to resist lateral displacement induced by lateral loads. It was also found out that the vertical dead loads reduced the uplift of the panels which reduced the lateral displacement occurred out of rotation of the panels.

(Ceccotti & Karacabeyli, 1996) investigated the effects of fastener type on wall performance and the effect of gypsum wallboard on the shear wall capacity. It was concluded that the gypsum wallboard increases strength and stiffness. But in comparison to OSB or plywood it decreased the ductility of the shear wall.

(Kamiya, 1996) carried out a test where the comparison of effect of fastening the studs of shear wall with bolts was compared with fastening the studs of shear walls with nails. There was a slight difference in the shear resistance of the nailed stud walls against the bolted stud walls. While in the case of high acceleration, the displacement response of the walls with nailed studs were significantly higher than the bolted studs.

(Jones, 1997) carried out experimental investigation on the effect of sill plate connection, bolt sizes and clamps on the shear wall capacity. This test evaluated the standard connection as well as the connection strengthened with reinforcing clamps. The clamps showed increase in stiffness and 26% more energy dissipation of the shear wall specimen. These clamps also increased the strength, energy absorption of wood-to-concrete and wood-to-wood connection.

(Karacabeyli, Stiemer, & Ni, 2001) initiated a new type of shear wall system. Instead of having studs on the one side, the sheathing was sandwiched in between the two studs that were rotated 90 degrees from their usual orientation. Because of this the fasteners connecting the sheathing to the studs got loaded in double shear. Then the new shear wall tested using the shake table was found to perform better than conventional shear walls.

2.4.2 Effect Of Openings on The Performance of Shear Walls

In an investigation by (He, Magnusson, Lam, & Prion, 1999) the effect of openings on the lateral resistance of the wooden shear walls with standard and oversized sheathing panels, it was found out that the application of oversized sheathing panels improved the shear wall performance. The openings had a significant decrease in strength and stiffness and changed the failure model.

2.4.3 Wooden Shear Wall Testing According to ASTM E564-06

ASTM E564-06 describes the general layout of the tests that can be conducted to evaluate the shear capacity of a wooden framed wall that was supported on a rigid foundation. The load was applied along the plane of the wall along that edge which is opposite of the rigid support and in a parallel direction. It describes the shear stiffness and shear strength of the wooden frame wall to use it as a shear wall on a rigid support. In this test the wall under consideration undergoes racking deformation. This is achieved by anchoring the bottom of the wall assembly and the load is applied on the top edge of the wall height parallel along the length of the wall. Then the measurements are taken for the forces, displacements at each load interval.

The assembly of the test consists of frame of shear wall including diagonal bracing members or reinforcements. The assembly represents the minimum acceptable stiffness of the frame and sheathing material. The performance of the wall is heavily based on the spacing of the framing, sheathing to frame connection and the anchorage connection to the floor and foundation. The connections used in the test are representative to those used in actual building construction scenarios. Similarly, the size of the connector, location of the frame and size of the test wall, number and grade of the members are representative of the actual building trends. The load that is distributed on the top of the wall simulates the load on the floor or roof of the actual building. To prevent the test assembly to displace out of the loading, the wall is supported along its top with rollers. The size of the wall changes according to the desired study. In case the performance of the wall is desired according to the curing conditions, age, moisture content or temperature, then the entire wall system is cured according to the specification of required industry. If the wall is required to be tested for certain specific environmental conditions, then they are tested in an environmental chamber.

The section recommends testing a minimum of two wall system of a particular set of specifications to determine the shear capacity. A reversed direction of load is applied for the second test of shear walls in case they are unsymmetrical. A third test is conducted if the shear strength or stiffness of the second test is not within 15% of the results of the first test in such an orientation which is like the weaker of the two tests. Then the strength and stiffness shall be average of the two weak ones. A hydraulic jack that can induce a constant displacement rate for a continuous failure is applied at the top of the wall which is parallel to the lengthwise plane of the frame. The load is applied at a constant rate of displacement to arrive at the target in a time not less than five minutes. Whenever gravity load is required, it is applied along the top of the frame or roof. The duration of application of each increment should be one minute before any recordings of the deflections are made. Apply almost 10% of the desired ultimate load for five minutes. Then remove the load and wait for five minutes and read and record the initial readings. When arriving at one third or two thirds of the estimated ultimate load, remove the load, wait for five minutes, and record the recovery of the wall. In an equivalent way, loading and unloading can be carried on until the desired ultimate load is achieved.

The desired output from this test is the shear strength, shear stiffness and shear strain. Shear strength is calculated by dividing the maximum load until failure by the length of the wall. Shear Stiffness is quantified by the racking load in terms of corresponding shear displacement. At the same time strain is the angular displacement. Racking load is estimated by using the line pressure to a calibrated loading ram or a load cell which is mounted to the loading device. The shear displacement of a wall frame is suggested to be accurate to 0.01 inches. It can be measured by two approaches: direct measurement or measuring the diagonal elongation of the frame. For the case of direct measurement, four displacements are measured in various sections of the frame: Base Slip, Uplift, Top Plate horizontal displacement and Vertical displacement. This occurs as the frame tends to rotate and translate from a rectangle to a skewed parallelogram or a rhombus. To measure the base slip, the displacement is measured at the centroid of the bottom of the plate with respect to the test machine in a direction parallel to the length of the wall. The uplift is recorded as the distance which the bottom of the stud at the end of the loaded wall is lifted off the base of the machine, indicated the degree of rigid body rotation. The top plate horizontal displacement is measured at the top of the centroid of the top plate. This is due to combination of the rigid body rotation, horizontal translation of the entire wall and shear deformation. The vertical displacement is measured at the toe of the wall with reference to the base. The total horizontal displacement of the top of the wall is computed according to the diagonal elongation approach is as shown below:

$$(c + \delta)^2 = (b + \Delta)^2 + (a^2 - \Delta^2)$$

After substituting:

$$a^2 + b^2 = c^2$$

We obtain:

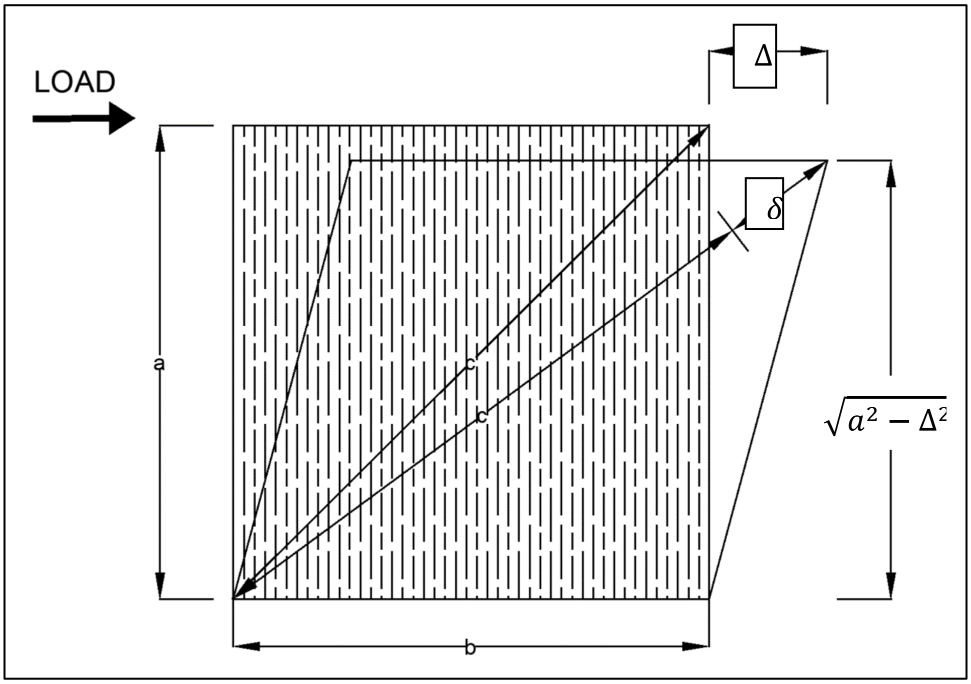
$$2c\delta + \delta^2 - 2b\Delta = 0$$

Then:

$$\Delta = \frac{(2c\delta+\delta^2)}{2b}$$

Figure 2

Horizontal Shear Displacement Calculation (ASTM E564-06)



The load deformation curve is measured either continuously or in an incremental manner. If there is a continuously measuring device, then at least five sets of readings are taken for each load cycle to create a load- deflection curve. In case the shear wall is with an opening, then the change in length of the opening provides the shear deformation for the purpose of verifying the accuracy of the racking measurement.

The Global Stiffness (G') value includes the rotation and translation of the wall or both and provides an overall evaluation of the wall performance within the assembly. It is computed as:

$$G' = \frac{P}{\Delta} \times \frac{a}{b}$$

The Internal Shear Stiffness (G_{int}) is the actual shear of a particular wall assembly due to wall shear displacement. It uses the similar expression as the global stiffness except the Δ is used that does not consider the rotation and translation. The equation for internal shear stiffness is presented as below:

$$G_{int} = \frac{P}{\Delta_{int}} \times \frac{a}{b}$$

Internal Shear Displacement using Horizontal and vertical displacement measurements (Δ_{int}): This displacement is determined as a function of the measured displacement shown in the figure-2 above.

$\Delta_{int} = \Delta_3$-$\Delta_1(\Delta_2 - \Delta_4) \times \frac{a}{b}$

Ultimate Shear Strength: It gives the ultimate shear strength of the wall and is calculated as below:

$$S_u = \frac{P_u}{b}$$

A load deflection curve showing the gravity and vertical loads along with deflection for different assemblies is plotted for a better visual representation.

2.4.4 Testing of Wood Structural Shear Panels Under Cyclic Loading

On January 17th, 1994, a 6.7 magnitude earthquake hit Northridge in the San Fernando valley California. The quake lasted for 10-20 seconds and caused a ground velocity of 4.1 miles per hour. A total of 57 people died and caused total property damage of $20 billion and more than $40 billion of economic losses, making it the costliest natural disaster in the U.S. history (California Departmet of Conservation, 2019). Buildings are induced to in-plane shear loads in one direction but changes of wind gusts cause reversal of shear load to opposite direction if the building falls into the “eye” of the storm. While most of the structures were constructed based on monotonic testing of shear wall components, this started questions about the performance of the shear walls during cyclic (reversed) loading. At the same time questions about effect of dissimilar materials to stiffness and strength of wall, shear wall displacement limits, fatigue resistance of fasteners and deformation of holders and connectors during cyclic loading were also needed to be answered. This led to generating a sequential phased displacement test procedure, which was developed jointly by U.S. and Japanese engineers in 1987 as Joint Technical Coordinating Committee on Masonry Research (TCCMAR) (Rose, 1998). This was used as a base for generating a standard cyclic load test method for testing wooden shear wall components and the factors that affect the performance.

3.0 Renovation, Rehabilitation and Remodeling in United States

3.1 Introduction and History of Renovation

Renovation is a process of repair or alteration of any building or structure that extends the structure's estimated useful life. Similarly, rehabilitation is an upgrade done on those buildings and structures which is done to meet the present need and requirements. In case of rehabilitation, the present aesthetic and general overlook of the building is preserved. Remodeling is very much like renovation as it is applied to residential construction. It is done to provide extra strength and more aesthetic beauty to the building. Those buildings that are functionally inadequate and underutilized to serve the demand of the present usually go through the process of rehabilitation and renovation.

One of the popular examples of renovation goes back to 1837 when a famous building in Boston underwent a drastic change (Newman, 2021). It was the Custom House in Boston that was built to collect tax and originally located near the water. As time passed by the harbor was filled with water and it lost its proximity to water. Finally, the building was sold to private developers and converted to an office in the 1990s. Even though the building was small, it has rejuvenated its existence and stands tall with pride and rich history.

Similarly, The Board of Trade Building in Boston was completely changed and converted into apartments. It was heavily decorated, and the nearby buildings could not match its architectural sophistication. Quickly it became popular among executives visiting the city and was crowned as the city's jewel. The interior of the building was aided by renovation projects. Inclusion of new staircase, escalators, lobby, and atrium spaces requires the shuffling of structural members like adding or extending the beam, columns and providing foundations for them. Renovation has served as more than artistic decoration and can also be applied for lateral load upgrading. It involves the installation of new shear walls, braced frames and anchoring or bracing of nonstructural elements like partitions, HVAC equipment and exterior parapets. Seismically retrofitting the buildings with fluid viscous dampers has enabled the buildings to preserve their exterior and resist the code required levels of earthquake loading.

Renovation is also done to prevent the building envelope from deteriorating. The leaking of the roofs, walls, buckling of walls, cracking, and failing of sealants are some of the most common deteriorations in old buildings. Age, prolonged exposure to sunlight and other natural elements are usually reasons for these damages. Masonry walls are most prone to mortar deterioration, efflorescence, and discoloring. If the mortar is not replaced or rejoined, then there is a high chance that the water will percolate and will get a chance to freeze within the hollow spaces and expanding ice causes further crumbling and destruction of the joints. Similarly, without proper maintenance, a small leak into the roof insulation saturated with water may corrode the structural members. By the time the rust on the ceilings is observed, a minor problem results in a major roof removal and replacement of structural members.

Rehabilitation work is heavily done to prevent structural damage and failure. Weather related deterioration and corrosion are some of the most common reasons. Corrosion of reinforcement bars in concrete slabs of parking garages are some of the most common. Highly concentrated use of deicing salts exposes the concrete decks to chloride-ion infiltration and increases concrete deterioration. This causes corrosion in reinforcement bars resulting in spalling and cracking of concrete (Wu, Shi, Gao, & Wang, 2014). Surprisingly, structural rehabilitation can also mitigate terrorist attacks on famous landmarks and save both the lives of people and the property. Violent wind and earthquakes not only damage the structure, but also distort its components. It might cause serviceability problems like inoperable windows, binding doors, vibrating floors in dance and aerobic studios, buckled floors and partitions. It’s been considered that sophisticated design methods, use of composite design materials and high strength steels are making the building frame lighter, which have lower natural frequency. Whenever the frequency created by the occupants of the building matches this natural frequency then there is a strong vibration. A lot of the time dealing with these kinds of distortions are difficult than fixing the evident damages. Similarly, government mandates also raise the requirement of renovation. A popular example is the requirement of all building and public access to comply with the guidelines of American Disabilities Act (ADA). This includes the removal of asbestos, handicapped-accessible ramps with appropriate mandated slope. Another example is the installation of interior elevators for multistory public buildings like courthouses. At the same time, buildings in New York City require certain fire-protection systems, fire alarm systems, smoke controlling devices in multistory buildings. Whenever local government or governing authority brings new mandates, there is a certain kind of renovation required in the structures.

Sometimes the buildings undergo renovations to correct the previously carried out poor and wrongful designs. For example, the Enid Haupt Conservatory in New York City, one of the largest conservatories in the country had to undergo renovations for five times to correct the problems like glass sealed away from the seals, leakage from the windows and corrosion on structural members.

The 200-year-old Octagon in Washington DC was renovated in 1954, the second-floor wood joists were replaced by steel framing and a concrete deck was made. Later, the concrete diaphragm was replaced with a system that duplicates the original flexibility property of the floor. Wooden notches and framing are highly prone to damage by notches and cuts for the installation of plumbing pipes and electrical pathways. So, one of the major examples where the renovation was done to compromise the building’s original character was the massive store archway of Fort Trumbull of New London, Connecticut which was blocked with bricks to convert into a laboratory. Years later the building was restored back to its previous condition for the purpose of being a tourist spot. Sometimes renovations are carried out to recover the effects of previously carried out incomplete renovation effects. One of such examples is a factory building in Italy in 1930s which was purchased by a telephone company. The renovation work included construction of a mezzanine, which means a short story between the ground floor and the first floor, which was done without construction of column footing. Also, to install strip windows and another floor, a substantial portion of the lateral load bracing of the building was removed. So, a renovation project was done to provide a new foundation for the mezzanine and provide lateral bracing of the building.

3.2 Common Structural Challenges and Solutions

Sometimes it is really challenging to find the design data for many of the structures which were built before World War two. These residential and commercial buildings are still being used in different capacities. Even though it is found their use is limited and some of them are empirically designed and don’t help with any of the desired analysis. Some of these structures are designed based on field experiences. And it is hard for structural engineers to figure out a way to understand them and renovate accordingly. Especially a system involving structural steel, open web steel joints and engineered woods are difficult to interpret. It was possible to understand the design designation and load carrying capacity based on the field measurements of the frames. With the specific design load tables, the capacity of the open web steel joists could be found. Similarly, the unknown sized steel beams can be measured in the field and their designation can be obtained from American Institute of Steel Construction (AISC) chapters. At the same time, engineered woods systems can be analyzed based on information contained in the manufacture’s manual.

Inadequacy of the structural roof framing and the floor are some of the major renovations that are observed in the residential buildings. In case the steel framing members are seen inadequate then they are reinforced by adding new members between the existing members. It is usually done by welding a new section or by introducing a combined action with a concrete floor. If the members are made up of reinforced concrete, then they are upgraded by adding the reinforced beams columns, reducing the span, or performing external post- tensioning for the concrete sections. Similarly, the wooden sections are strengthened by adding new members to divide the load path within the frame.

While strengthening is considered as one of the popular ways of uplifting the load carrying capacity of a structure, there are always ways to determine the additional structural capacity of the structure to support the increasing load. This is especially useful in coping with the change of design provisions of the Codes. For example, the snow loading has undergone significant changes over the years and the roofs that were previously understood to be comply with the codes are not doing so anymore. These investigations are time and capital consuming as they require a lot of designs and material testing, but they are less expensive than strengthening the members.

A lot of old buildings were not designed structurally to resist the lateral loads. Even if they had such a system, then they are far from being adequate by today's standards and codes. It has been considered the most expensive type of renovation. Installation of shear walls, braced frame, floor, and diaphragm strengthening are some of the common lateral load enhancement projects. In more complex structures there are works like isolation of the base, installation of dampers for making the structure seismic stable. Seismic retrofitting work also includes within this area where there is seismic bracing of non- structural elements like partitions and parapets.

Upgrading HVAC system creates challenges for structures. This results in creating new floors, roofs and wall openings for ducts and pipes. Th situation gets really complicated when we are trying to do that for a two-way concrete slab. Passing a large, heated tube through a beam is never a desirable action in terms of structural engineering point of view. But a lot of the time there might not be a better way and we end up making web openings in beams. Similarly, making openings in load bearing walls might get really challenging. We are always provided with certain percentage of voids in this kind of structural sections and use of lintels over these openings can keep the openings safe but sometimes we need huge openings, and, in that case, we have no option than rebuilding the entire wall.

Similarly, upgrading or installation of HVAC on the top of the roof is challenging for old buildings. This includes creation of floor and roof openings and making sure the existing frame has enough load capacity to support the newly installed units. So, the solution is to ignore the roof and the beams altogether and make a structural frame attaching it to the existing columns and then supporting the new equipment.

Removing load bearing wall is also one of the major renovation works. Usually, it is done for purposes of extending the room space like extending a portioned space into open hall. Or making a door open wide. This requires a proper reordering of the partition walls or placing of lintel sections on the remaining parts of the walls. It is important to check and reinforce the wall structures to make sure that they can carry the loads from these newly installed lintels. In case when we are removing the entire wall, then it requires the installation of new framing structures like beams and columns. This frame structure should have shear strength equal to or more than that of the shear walls. Then only in that case the frames will be able to hold the lateral force and all the gravity loads like dead load of the structure, live load from the users and components of the building.

There have been a lot of situations where failure or damage of structures have happened because of the failure of identifying the wind vulnerabilities in the buildings. One of such examples is the renovation of the Main building complex at a Fairground in Bay County, Florida. This building was planned as a critical facility for post-hurricane staging area for county supplies and hurricane recovery shelter for affected residents. There was retrofitting done to the building which included rolling doors and roof purlins in 2011. When hurricane Michael hit the building in 2018, all the seven retrofitted rolling doors failed because the door frame members did not have enough wind resistance. In another example the city hall and police station building in Bay County which was built in 1942 was retrofitted around 2001 with roll down storm shutters to provide resistance to high-speed wind. The storm shutters survived hurricane Michael but the other structures like the roof trusses for the sloped roof failed. This caused the rain to enter the building, which damaged the interior of the building. It can be concluded that the structural deficiencies of the building were not resolved before installing the roll-down shutters. These failures suggest that retrofitting one element of the building while neglecting the wind vulnerabilities in the other parts of the building does not lead to effective wind hazard mitigation. Whenever there is a requirement of retrofitting of an element the whole building must be assessed if it can achieve the performance demanded by the retrofit. To achieve this a detailed plan must be created to identify the weak and sensitive areas of the building. The following five steps are recommended by (FEMA, 2013) for improving wind resistance of existing structures:

1. Comprehensive wind vulnerability assessment of the structure: A team of highly qualified engineers perform an assessment where they review the historical information of as-built drawing, specifications, and previous repairs. They also check the structure for common high wind vulnerabilities for the roof covering, roof framing, soffits and wall covering by both destructive and non-destructive testing.
2. Assessment of alternatives for repairs, retrofit or new construction for damaged structures.
3. Comparing semi retrofit vs full retrofit.

4. Applying best wind retrofit practices.
5. Developing backup plan for failures.

3.3 History of Building Codes in Renovation

The International Building Code which was published in the year 2000 is followed all over the United States. It replaces all the provisions from the past like The Uniform Building Code, The BOCA National Building Code and The Standard Building Code. It was becoming a nightmare of regulations for the American Institute of Architects. In 1991 it passed a resolution that would support a single code.

There have been significant changes to the code throughout the period. One of the major changes was the statement of IBC Chapter 34 that stated any addition or alteration shall not be allowed to increase the force in any structural elements by more than 5 percent unless the element complies of the new construction and it should not decrease the strength below the level required by the code. Later in 1997, the State of New Jersey adopted its first rehabilitation subcode which was later followed by the state of Maryland, Rhode Island, and other states. The major ideas of these codes were to reclaim and reutilize the existing building structures. Quickly these codes became more vigorous, and the International Code Council intrigued to develop the International Existing Building Code. The first edition of IEBC was started in 2003 which followed the tri-annual cycle of IBC expansion. For quite some decades the designers had the option to choose either IBC chapter 34 or IEBC. Later in 2009, the IBC chapter 34 was revised to be well matched with those of IEBC and since 2015 chapter 34 has been terminated. So currently, the IEBC has been the apotheosis of all the building renovation provisions. It offers the best comprehensive set of rehabilitation provisions.

Since 2018, IEBC has been the most significant hub for all the provisions that may be used in the renovations that incorporate one- and two-family townhouses and dwellings no higher than three stories above. The code provisions of the IEBC have three major methods of compliance. They are Prescriptive, Work Area and Performance. The prescriptive method contains provisions that are like those in early editions of IBC Chapter 34 and deals with alterations, additions, change of occupancy and renovation of historic buildings. The performance method deals with repairs, alternations, addition and change of occupancy and works for buildings in certain existing occupancies: Group A, B, E, I-2, M, R and S. The performance method uses a point system, in which separate scores are given in terms of fire safety, means of egress, general safety and total building score is determined. This evaluation process determined the building's consent with the codes.

4.0 Lateral Load Resisting System

4.1 Introduction

Lateral Loads means the combined effect of the wind load and the earthquake load. The effect produced by the wind is external and depends on the shape and dimension of the building. Opposite, the earthquakes cause internal inertial forces and causes damage. The intensity of the damage depends on the building's mass, type of construction and the soil upon which the building rests. From the beginning of human civilization till this date, the severity that has been brought upon by the high wind has been immense and we have seen enormous damage to property and fatalities. Hurricanes have been the reason for the top five most expensive natural disasters in the United States' history. The biggest one is the Hurricane Katrina which resulted into $125 billion loss and 1836 total fatalities (Wikipedia, 2022). The amount of damage incurred by the earthquakes and the amount of research funds allocated towards earthquake damages mitigation are dwarfed in comparison to that of wind. Understanding the effect of wind load on buildings is still a part of the challenge and we still have a lot of knowledge to gain. Research done under this section often leads to revision of the existing building codes.

The standard codes are the International Building code and ASCE 7 which has U.S. maps of basic wind speeds and has annual return interval that correlates with the risk category of the structure. So far, we have understood that the high wind causes four types of damage to the building. They are as follows:

1. Collapse: This means the building disintegrates just like the way a house of cards falls apart.
2. Partial damage: This means only the weak part of the building gets damaged.
3. Overturning: This happens when the building overturns, but it does not necessarily break.

4. Sliding: In this case the structure moves away from its original position.

Among these several types, the partial damage is the one that occurs the most. High wind usually damages the areas of the walls and roofs located near the corners. Therefore, the areas like roofing, members of secondary structural function and walls are designed for higher wind load capacity than the other areas of the building. During the event of the hurricanes, there is partial failure cladding of the walls, roofing, windows, and doors because of the wind and the material debris that is flying in the air. Once certain areas of the buildings are damaged and exposed, then the rain and high gusts of wind get inside which results in a huge increase in internal air pressure resulting in destruction of major internal structural components. One such example is the interior of the famous six story headquarters of Burger King in Miami. The interior was totally wrecked, and it took almost a year for the relocation of the store. The wind damaged the weak areas like window frames that resulted in access to rainwater and hence causing damage of $25 to $30 million.

This incident proved the idea of wind forces always acting horizontally on the projected area of the building wrong. These days the codes take into consideration that the wind load distribution is more complex. These loads either are pushing against the structure or are applying suction both externally to the building and internally within the building.

4.2 Load Resisting Mechanism

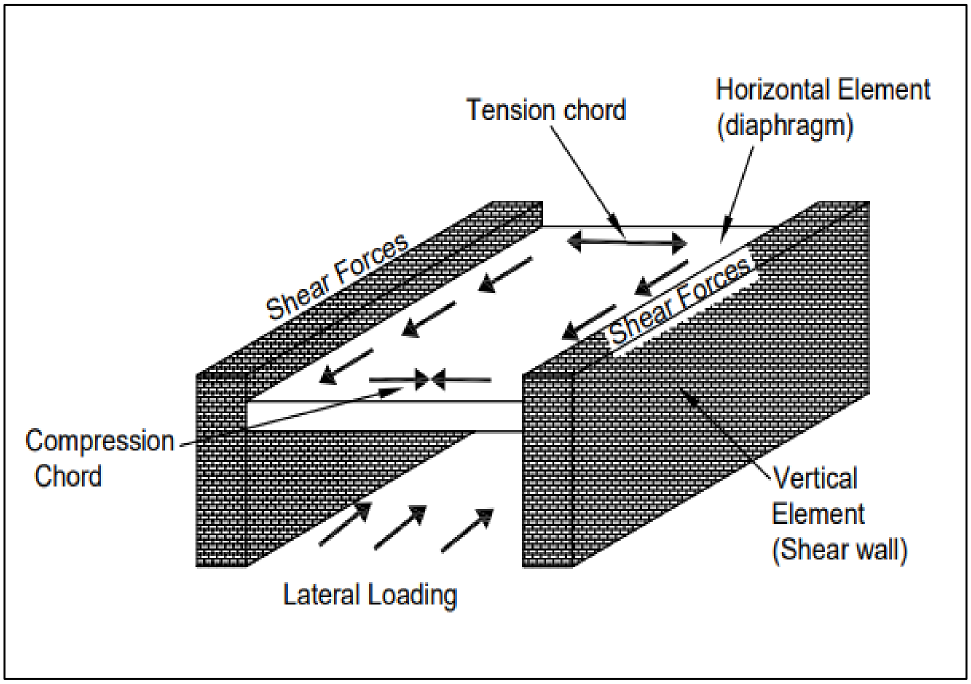
Lateral Load Resisting System (LLRS) means that structure of the building that is designed in resisting the forces exerted by the wind and the seismic loads. The most prominent building codes like ASCE and IBC mandate this system as an important design criterion. It has always been challenging to determine a building’s lateral load resisting capacity.

A typical LLRS has both vertical and horizontal elements that are interconnected and whenever any load acts upon them, both these systems can act together. As shown in figure 3, whenever there is any lateral load acting on the body, it first acts on the horizontal element which is called as diaphragms, diaphragms transfer this load to the vertical element which can be shear walls or columns. The diaphragms can be present in the form of beams, chords, or flanges as well. Diaphragm might have plywood, wood, steel or concrete deck and steel bracing and metal that fastens the roof. There are two types of diaphragms based on blocking of the panels: blocked and unblocked diaphragms. Blocked diaphragm is the one where all the panel edges are nailed to common framing, this helps in transferring the shear from one panel to another, hence it overall increases the shear capacity and rigidity of the diaphragm. While unblocked diaphragms have panels usually around 4 feet apart, these panels are not connected to each other but join lengthwise or breadthwise to a common frame (APA The Engineered Wood Association, 2003). Unblocked diaphragms are mostly common in residential construction. They might be both rigid and flexible depending on the type of material they are made of. It has been found out that the flexible diaphragm is not able to distribute torsional stresses to vertical members well. Hence rigid diaphragms are preferred.

So, from the very beginning the above concept was understood and the importance of vertical component was realized. The most common approach to make the vertical component stronger was to make the unreinforced masonry unit thicker and more solid. Of course, the thicker wall was better resistant to wind load compared to smaller wall, but it was calamitous for seismic load.

Figure 3

Lateral Load Resisting Mechanism (Newman, 2021)



Traditionally, a lot of wooden framed buildings also relied on exterior shear walls and some interior partitions walls for lateral stability. Most of these walls were not designed nor reinforced but their stability was acceptable in most cases of wind and seismic events. A lot of significant studies have proven that sustainability of buildings upon horizontal lateral forces is based on its stiffness and the horizontal diaphragm's aspect ratio. Another popular method for lateral stability for the steel structures was the partially restrained connections. They used to be called wind connections and they used to be very rigid and resisted lateral loads in a similar fashion to moment connections. At the same time, they were flexible enough for the beams to behave like simply supported members under the action of gravity loads. These connections were made either with hot rolled angles or tee sections which were attached to the beams and columns with rivets. The downside of this method was it was a thorough design and not very simpler methods were present to analyze it. (Roeder, 1995) conducted lateral load performance of steel framed and semirigid connections, he found out that the frames were unsuccessful in providing the seismic resistance the current building codes were requiring. He found out that all these buildings had been enclosed with fireproofing concrete and it had unreinforced masonry walls that were stronger than the frame itself.

As of now, the steel framed buildings systems have been developed very well and they rely on fully restrained moment connections. These connections are made by rivets, bolts, and welds. Based on feasibility and performance welded moment connection has been widely popular these days. This type of connection has excellent energy dissipation, and it can create plastic hinges.

4.3 Lateral Load Resisting System Upgrading

Chapter 34 of International Building Code, Existing Structures from 2000 until 2012 had provisions related to seismic retrofitting of buildings, but it had not been cleared at which situations the retrofit was required. Later, the International Existing Building Code (IEBC) specified the circumstances where the building should comply with the wind and seismic provisions and be concerned when it does not. It has specified where the entire building should be upgraded and other cases where only certain members should be upgraded.

The following are the scenarios where the entire building should be evaluated and most likely to be upgraded:

1. Whenever repairs of any substantial structural damage are done to the vertical element of the lateral load resisting structure.
2. When repairs of the substantial structures of the gravity load carrying members are done. Considering these members were damaged by lateral loads.
3. When repairs of the disproportional earthquake damages in buildings are done.
4. Whenever there is an alteration work for structural irregularity as defined in ASCE-7, that would result in decrease in the capacity of the lateral load resisting element.

5. Whenever there is an alteration work that would involve a significant structural change, then the entire structure system should be able to resist the criteria of IBC wind and decreased level of seismic forces mentioned in IBC.
6. When a building is undergoing changes in occupancy that has chance of categorizing the building into higher risk category for wind or seismic loading conditions, then the existing lateral load resisting system should be able to withstand the full IBC- level wind or seismic loading conditions.
7. When a building or structure which has been upgraded into risk category 4 because of change of occupancy and has been providing operational access to an adjacent building, then in this case it should be able to withstand the full IBC-level wind and seismic forces.
8. When an additional structure is added to an existing building (which was not structurally independent) then the combined building should be able to resist the full IBC wind and seismic forces.
9. When buildings are relocated, then they should be able to withstand the full-IBC level wind and seismic forces.

At the same time there are scenarios where only certain components or structural members require lateral retrofitting and only certain structural members or frames that require lateral load resistance evolution.

The following are the scenarios where the certain members or components should be evaluated and most likely to be upgraded:

1. There should be bracing of unreinforced masonry bearing walls when there is reroofing of more than 25 percent of the building is done. This proposed work is classified as Alteration Level 1,2 or 3 by IBC.

2. The connection to the roof framing and roof to wall connections and the structural capacity of the diaphragm must be evaluated when there is more than 50 percent of the roof diaphragm of the building which is in the high wind area is exposed during the reroofing job.
3. The wall anchors should be placed at roof level when there is Alteration level 3-IBC work is going on in the building with reinforced masonry wall or concrete walls. In case the walls are unreinforced masonry walls then the anchors should be provided at each floor level.
4. Any unreinforced masonry partition or nonstructural walls, parapets that are related to work classified as Alteration level 3 should be reinforced, anchored, braced, or removed to withhold out of plane seismic forces.

Chapter 7 of the IBC defines Alteration Level 1 as work that involves replacement or covering of existing materials, components (structural, mechanical, or electrical) using the new materials while maintaining the same function. Alteration Level 2 is defined as the work that reconfigures up to 50 percent of the area of the building while Alteration Level 3 involves reconfiguration of space which is more than 50 percent of the area of the building. Usually, these reconfigurations are related to extending the open floor spaces, sprinkler system or addition of staircases or fire escapes.

4.4 Common Scenarios for Upgrading the Lateral Load Resisting System

Upgrading the LLRS of a building means strengthening its horizontal and vertical lateral load resisting members. It can be done in multiple ways by adding new elements, reinforcing the existing ones or if the structure is very deficient then we replace it with a completely new element. Meanwhile these structure elements like shear walls and frames are upgraded, it is also equally important to check their foundations as well, making sure they can sustain the new conditions of loading or upgrades with reinforcements.

So far, the engineers have narrowed down certain building conditions and patterns that have repeatedly failed in high wind loads and are the first in the list that require periodic check, renovation, and reinforcement. Some of them are listed below:

1. Buildings which have tall height, having big open floor areas, having areas of sharp corners, floors with soft stories (soft stories are usually the ground floors with just columns and no shear walls, usually created for open spaces for parking) which have lower lateral stiffness compared with other higher floors. Also, the buildings with irregular configurations have varying stiffness.
2. Buildings in areas where the soil has higher chances of liquefaction.
3. Buildings that have weaker or no connections between the walls and floors.
4. Buildings with shear walls made up of unreinforced masonry walls, which have cracks and fractures.
5. Buildings where there is inadequate anchorage and support with the foundations and in the case of concrete buildings with insufficient slab framing.

Meanwhile, most of the failure cases have been identified because of blowing off the roof decks and shear walls. The common reason for this is inadequacy of load path and connection failure. Some other common reasons for these are workmanship error and reduced capacity due to termites and corrosion (FEMA, 2007). At the same time, it is important to understand the damages and vulnerability that can be brought by failure of some of the non-structural building elements like chimneys, parapet walls and balconies as well. They always have chances of toppling, falling over, damaging the structural elements, and causing damage to people and other properties. Hence, their upgrades should not be ignored.

4.5 Upgrading of Diaphragm

The type of floor and the roof diaphragm plays a vital role in determining the distribution of lateral loads to the other components of the lateral load resisting system. For flexible diaphragms like wooden and metal decks they transfer lateral loads to other vertical elements based on their tributary area. For example, in a building which is made up of three wooden shear walls where two walls are facing each other opposite and one shear wall is at the center line, the center line wall will resist one third of the total acting lateral load. In the case of rigid diaphragms, for example consider a concrete floor, it distributes the lateral loads to the vertical components in proportion to their relative rigidity. For the same example with three equal shear walls, the middle walls will take one third of the horizontal load. In comparison with rigid diaphragms the flexible diaphragms are considered weak as they cannot sustain high torsion and might get damaged in extreme wind speeds. Hence it is important to understand the function and type of the existing diaphragm before upgrading the vertical components of LLRS of the building. In case the existing diaphragm is deficient for the design lateral load conditions then it needs to be reinforced or changed. Reinforcing a diaphragm depends on its material composition and its weakness. The deficiency of a diaphragm may be because of inadequate stiffness or proper connection.

Historically, most of the wooden framed buildings had wooden boards as diaphragms that are nailed to the wooden joists or girders that rest on wooden columns. These wooden boards often had weak shear strength and were highly flexible and their overall connection to the walls was very weak. In present, The Table A108.1(1) of IEBC Appendix A has assigned substantial strength values to these materials. They range from 300 lb/ft to 1800 lb/ft for shear in roofs with straight sheathing and for floors with diagonal sheathing, respectively. The connection between the wood joists and the walls was quite simple in the past. They were just placed into pockets into the masonry walls without any kind of anchorages. In the event of fire, it was considered appropriate for the destroyed joists to fall without crushing the walls with it. For that reason, the joists were cut at an angle popularly called fire cut to help them rotate away from the walls. Present design philosophy is exactly opposite where we want a strong connection between walls and diaphragms. Addition of anchors between the walls and the floors or in between the walls and roof is the current way of lateral retrofitting.

These anchors have the following functions:

1. They provide tensile strength to the diaphragm by connecting it to the walls hence necessary bonding is provided in case of strong wind events like hurricanes and tornados. ASCE 7-16 Section 12.11 provides required analysis procedures.
2. Exchange of forces is possible by these anchors. These anchors have necessary shear strength to them for lateral load transfer.
3. If their purpose is to transfer gravity loads downward, then they are designed to resist downward shear forces as well.

   (FEMA, 2007) has specified three types of anchors in between diaphragms and unreinforced masonry walls/wooden shear walls.

1. Drilled dowels for shear tie. They have an allowable shear capacity of 1000 lbs and no tension capacity.
2. Drilled bend dowels placed in adhesive anchors at an angle of 22.5 degrees. They have an allowable shear of 1000 lbs. and tension of 1200 lbs.
3. Through bolt anchors with allowable shear of 1000 lbs. and allowable tension of 1200 lbs.

In site load testing is done if the anchors are found to have insufficient capacities, in that case "TMS (The Masonry Society) 402- Building Code Requirements and Specifications for Masonry Structures" is used to upgrade the anchors as per requirements.

4.6 Adding Wooden Shear Wall and Braced Frame

One of the most common ways of strengthening the Lateral Load Resisting System is adding new shear wall or braced frames. The new wall might work as a complement to the existing structural members or might work as providing vertical rigidity to the building. In the case of rigid diaphragms, a study of the structural rigidities must be done to determine the portion of the lateral loading that this new member will accommodate. When the walls are made for providing sole rigidity of the building then the existing rigid elements that are nonstructural must be separated from the building. The primary advantage of adding a vertical lateral load resisting member is that it helps in decreasing the diaphragm span which helps in reducing the diaphragm strengthening process. The advantage of having new vertical members is to have clear space without changing or exposing the existing elements. At the same time, in case the space is a problem, then there is always a way to replace the existing members and partition walls. Additionally, where there is no requirement of additional strengthening of the existing framing then adding braced frame is an economical option. Usually in case of light framed buildings the most communal problem is not with the capacity of the shear walls but with the connections in between the walls and tiers, Tie-down connectors and steel straps are economic and easy methods for proper load transfer between the diaphragms and shear walls.

4.7 Reinforcing Existing Wooden Shear Walls

As mentioned above, sometimes addition of a new vertical lateral load resisting structural member is economical compared to reinforcing an existing member. But sometimes reinforcing is the only option. Reinforcing the shear walls upgrades the shear stiffness and strength of the members, upgrades their ductility, and expands their energy dissipation capacity (Chang, 2015). Some of the methods to reinforce the existing wooden shear walls are as follows (Dobrila & Premrov, 2003):

4.7.1 Using Additional Board Panels Over Existing Wooden Shear Panels

This is one of the simplest methods where the effect of the additional board depends upon the coefficient of stiffness of the fasteners. The ductility of the combined board and wall decreases hence additional board panels is not a good procedure if we are looking for an enhanced dynamic behavior of the wall. But the additional panel increases the stiffness and shear resistance of the walls by a huge margin.

4.7.2 Carbon Fiber or High Strength Synthetic Fiber Reinforcement

Carbon fibers are the answer to both high shear resistance and ductility. Tests conducted by (Bergmeister & Luggin , 2001) and (Tingley & Kent, 2001) showed that fiber reinforcement increased the stiffness and shear strength of beams by 22% and 5% in comparison to non-reinforced beams. The use of Fiber Reinforced Plastics (FRP) also increased the bending resistance for glulam beams (Stevens & Criner, 2000). It was found out that FRP glulam beams are practical, less expensive and can be used in bridges for bigger dimensions and spans. The use of high strength fibers for strengthening and repairs of the timber shear walls is being widely used and accepted. With development of new methods for large scale production is making these kinds of fibers less expensive and more innovative ideas for their use in timber shear walls.

4.7.3 Diagonal Steel Members

This is a common method of sharing the shear force of the shear wall with the diagonal frame. The connection of the shear wall with the frame is important and should be optimized in these methods. This method is adopted in severe cracks in the shear walls. A certain portion of the horizontal force on the shear walls is transferred to a tensile diagonal of steel member after the first set of cracks appear. It was found out that the shear resistance of the reinforced panels increased by almost 77% after the diagonal steel panels were installed with significant increase in ductility (Dobrila & Premrov, 2003).

4.7.4 Energy Dissipation Devices

Energy dissipation devices like fluid inertial dampers can be used for energy dissipation which is the result of lateral wind load. These dampers have a piston rod that carries the recoiling force to the moving plates. These moving plates are surrounded by parallel fixed plates with viscous fluids in between them and all of them are inside a cylinder. The fluids' shearing stress weakens the rod's movement and hence the lateral load acting on the rod is hugely reduced. This as a result reduces the deflection in the shear wall hence decreasing the requirement of reinforcements. The dampers are manufactured to resist the forces ranging to 10 to 2000 kips or even higher. They are selected and connected to the existing shear walls according to the requirement. Also, the highlighting advantage of these dampers is they can be installed in the bracing at locations away from the gravity frames, which makes it easier in case of repairing and safe even if they fail.

5.0 Calculations

5.1 Wind Load Calculation

According to ASCE 7-16, Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers, 2017), the following represent the four different methods a designer can choose to determine the wind load for a Main Wind Force Resisting System (MWFRS):

1. Directional procedure for the buildings of all heights.
2. Envelope procedure for low-rise buildings.
3. Directional procedure for building appurtenances and other structures.
4. Wind tunnel procedure for all buildings and other structures.

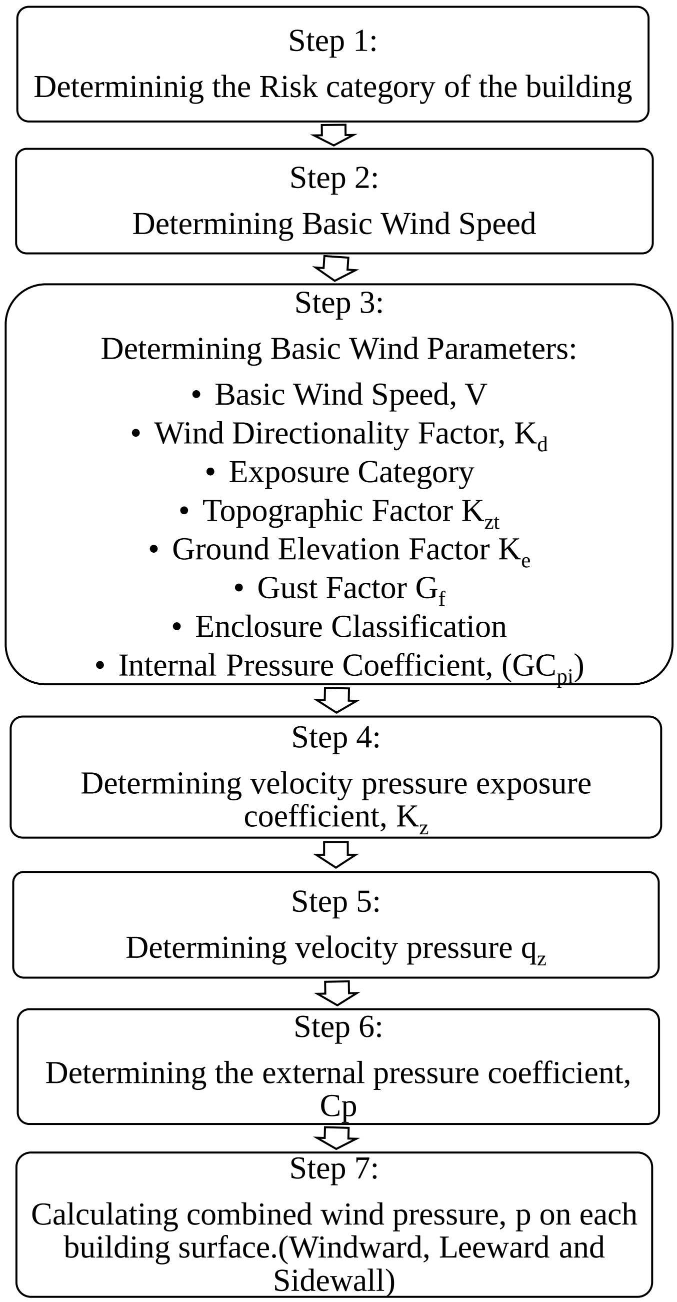
The wind load on the MWFRS in this research is determined in accordance with the Directional Procedure described in chapters 26-31, ASCE 7-16. The following is a brief discussion of the steps mentioned in figure 4 below.

Step 1: Determining the risk category of the building:

Section 1.5 Risk categorization, ASCE 7-16 classifies buildings and other structures based on risk to human health, life, and wellness in case of their destruction or failure of their occupancy or use. To apply various load conditions like flood, wind, snow, and earthquake these buildings and structures are categorized into various risk categories. This division is done in Table 1.5- ASCE 7-16. Buildings that represent minimal risk to human life in the case of collapse are categorized as risk category I. Buildings and structures that are categorized as essential facilities and whose failure poses a major risk to a large group of human life are categorized as risk category IV. These include buildings that assemble, fabricate, handle, and stores hazardous substances like fuels, chemicals and toxic wastes of quantities exceeding the threshold quantity established by the appropriate jurisdiction. Risk category III are those structures and buildings that also pose a substantial risk to human life and are not included in risk category IV. All buildings and structures except for those listed in Risk Categories I, III and IV fall under Category II.

Figure 4

Directional Procedure for Wind Load Calculation of MWFRS



These buildings with different risk categories have corresponding importance factors, which are shown in Table 1.5-2, ASCE 7-16. In this research, snow importance factor (I_s) has been applied in from the same table in calculation of snow load. For this research we have selected our sample building under Risk Category II.

Step 2: Determining the basic wind parameters for the applicable risk category:

- Basic Wind Speed:

Section 26.5.1, ASCE 7-16 provides the basic wind speed, V, which is used in the design of the wind load on buildings and other structures. They can be obtained from the wind hazard maps provided in the figures 26.5-1 and 26.5-2. These wind hazard maps are sub divided which can be used for determining the wind speed for the above-mentioned risk categories buildings. For risk category I buildings, the wind hazard maps provided in figure 26.5-1A and 26.5-2A are used while risk category II buildings and structures uses wind hazard maps from figures 26.5-1B and 26.5-2B. The risk category III buildings and structures uses wind hazard maps from figures 26.5-1C and 26.5-2C while the risk category IV buildings and structures use wind hazard maps from figures 26.5-1C and 26.5-2C. The direction of the wind obtained is assumed to be flowing in the horizontal plane in any direction. In case of any records or experience when the wind speeds are higher than the one mentioned in these figures then we use those higher values. In the wind hazard maps, there are areas like mountain terrain and gorges which are known as special wind regions. These regions are separated and examined for unusual wind conditions. The authority having the jurisdiction are allowed to adjust the wind speed values based on meteorological information and estimate basic wind speed in accordance with section 26.5.3.

Section 26.4.1 of the code describes the sign convention for wind pressure. The wind pressure is taken as positive when it is acting on the surface while it is taken as negative when acting away from the surface. Section 26.4.2 of the code describes the critical load conditions where the external and internal pressures are combined to determine the final load value. In case of calculating the wind pressure acting on the opposite faces of the building surfaces, the section

26.4.3 of the code is applied, which calculates the design wind load for the Main Wind Force Resisting System, Cladding and Covering by doing the algebraic sum of the pressure acting on opposite faces of the buildings.

- Wind Directionality Factor:

The wind directionality factor K_d is included in calculation of the wind load and can be determined from Table 26.6-1 of the code. This table categorizes the structures into buildings, arched roofs, domes, and many other types and provides corresponding K_d factors. The wind directional factor of 0.85 is taken into consideration for the main wind force resisting system.

- Exposure Categories:

For assigning the surface roughness, the exposure of the building or structure is determined. To determine exposure, two upwind sectors that are extending 45° on each side of the selected wind direction is taken. The exposure of these two sectors is determined according to sections 26.7.2 and 26.7.3 of the code. Then the exposure that results in the highest wind load is used to calculate the wind load in that direction.

According to section 26.7.2 of the code, the ground surface roughness within each of the 45° sector shall be obtained for the distance upwind of the site. These categories are as listed below:

a. Surface Roughness B: This includes urban, suburban, wooded and terrain with closely spaced hinderances which are of single-family sized houses or bigger.
b. Surface Roughness C: This includes open terrains with disintegrated hindrances that are of height less than 30ft. This category includes open fields.
c. Surface Roughness D: This includes flat, unobstructed areas and water bodies. This includes smooth mud flats, salt flats and unbroken ice surfaces.

According to section 26.7.3 of the code, the exposure categories are divided as below:

a. Exposure B: Those buildings and structures which have a mean roof height less than or equal to 30 feet and for which the surface roughness B exists in the upwind direction for a distance more than 1500 feet fall into this exposure category. If the height of the building or structures with mean roof height is more than 30 feet and surface roughness B exists in the upwind direction for a distance greater than 2600 feet or 20 times the height of the building or structure whichever is greater, they fall into this category.
b. Exposure C: Those structures that don't fall into exposure category B and D fall into this category.
c. Exposure D: In this exposure category, surface roughness D exists on the upwind direction for a distance greater than 5000 feet or 20 times the building or structure height, whichever is higher. This exposure category also applies in the situation where the ground surface roughness immediately upwind of the site is B or C, and site is within distance of 600 feet or 20 times the building or structure height whichever is greater.

- Topographic Factor:

A Topographic Factor (K_{zt}) is given by figure 26.8-1, ASCE 7-16 and is used in the calculation of design wind load because of the wind speed-up effect. The empirical equation for calculating the topographic factor is as below:

$K_{zt}= (1+K_1 \times K_2 \times K_3)^2$……………. *(Section 26.8-2, ASCE 7-16)*

where,

K_1= Factor to consider the shape of topographic feature and maximum speed-up effect.

K_2= Factor to consider the decrease in speed-up with distance upwind or downwind of crest.

K_3= Factor to consider the decrease in speed-up with the height above the local terrain.

In case the site conditions and locations of the buildings and other structures do not meet all the conditions then we take K_{zt} as 1.

- Ground Elevation Factor: According to Section 26.9 of the code, the ground elevation factor is taken as K_e=1 for all the elevations.
- Gust Effect and Gust Factor:

Tall buildings are flexible and as their height increases, they are slenderer, and their fundamental frequency keeps on decreasing. This fluctuates the wind pressure on its surface and creates a buffeting action. This buffeting action of the wind is traditionally known as gust effect and is treated by gust loading factor. In this method the equivalent-static wind loading is equal to the mean wind force multiplied by the Gust Loading Factor (Kareem & Zhou, 2003). According to Section 26.11.1 of the code, the gust-effect factor for a rigid building or other structure is taken as 0.85.

- Enclosure Classification:

The amount of the internal pressure of a building is dependent on the size and location of the opening in the building with respect to the wind direction. The amount of opening in the envelope determines the enclosure classification. Openings are those holes which allow air to flow through the building during the design wind. Section C 26.12 of the code defines an enclosed building as such there are not enough openings in the exterior part of the building that allows enough air into the building. Partially enclosed buildings are such where there are sufficient openings in the windward side of the building envelope to allow for the air to enter the building, but there are not sufficient openings in other portion of the building to allow air flow out of the building without having internal pressure build. In case of partially open buildings there are sufficient openings in the building to allow for the air to flow into the building and sufficient openings in the building to allow air to flow out of the building with some internally developed pressure. Open buildings are those where the air can enter and exit the building without significant accumulation of internal pressure.

- Internal Pressure Coefficients:

Section C 26.13 of the code provides the internal pressure coefficient values. These values are obtained from the wind tunnel tests conducted for the low-rise buildings and are also valid for buildings of any height. For enclosed buildings, the value of GC_{pi} = +0.18 or -0.18. In the case of a partially enclosed building the building has significant openings and internal pressure is affected by the exterior pressure at the opening. It has a high value of GC_{pi} = +0.55 or -0.55.

Step 4: Determining velocity pressure exposure coefficient:

The velocity pressure exposure coefficient (K_z or K_h and K_e) is defined by section 26.10.1 of the code. The exposure coefficients are classified according to exposure categories presented in section 26.7.3 of the code. The exposure coefficient (K_e) depends on the ground elevation above the sea level and is defined by Table 26.9-1 of the code. The following formula is applied. For calculation of K_e.

$K_e = e^{-0.0000362\, z_g}$ …………………. *(Section 26.9, ASCE 7-16)*

where,

z_g= gradient height or ground elevation above the sea level in feet.

Conservatively, K_e is taken as 1 and applied for most of the cases. The value of K_z depends on the height of the building above the ground level(z) and exposure category and is defined by Table 26.10-1 of the code. In case the value of the height is different from the table, empirical formulae are provided to determine the K_z as mentioned below.

$K_z \;=\; 2.01(\frac{z}{z_g})^{\frac{2}{\alpha}}$ for 15ft ≤ z ≤ z_g……………… *(Section C26.10-1, ASCE 7-16)*

$K_z \;=\; 2.01(\frac{15}{z_g})^{\frac{2}{\alpha}}$ for z ≤ 15 ft………………….... *(Section C26.10-2, ASCE 7-16)*

where,

α and z_g are terrain exposure constants and are provided in Table 26.11-1 of the code.

Step 5: Determining velocity pressure:

The velocity pressure(q_z) is determined at height z above the ground is calculated according to the following equation:

$q_z = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2$ …………… *(Section 26.10-1, ASCE 7-16)*

where,

$q_z =$ velocity pressure at height *z* of each level from ground (psf).

K_z = velocity pressure exposure coefficient, Section 26.10.1.

K_{zt} = topographic factor, Section 26.8.2.

K_d = wind directionality factor, Section 26.6.

K_e = ground elevation factor, Section 26.9.

V = basic wind speed, Section 26.5.

Risk category for determination of wind load on rooftop structures, rooftop equipment and other building supplements shall take the risk category greater in between the two:

a. Risk category of the building on which the equipment is placed.

b. Risk category for facility to which the equipment provides the designated service.

Step 6: Determining the external pressure coefficient C_P or C_N:

External pressure coefficient C_P is obtained from figure 27.3-1 of the code, for enclosed and partially enclosed buildings. Regarding sign convention, (+) sign signifies the pressure pushing onto the surface and (-) sign signifies the pressure acting away from the surface (i.e., suction).

Table 1

Wall Pressure Coefficients (Figure 27.3-1, ASCE 7-16)

Surface	Ratio of l/b	C_P	Use with
Windward wall	1.5	0.8	q_z

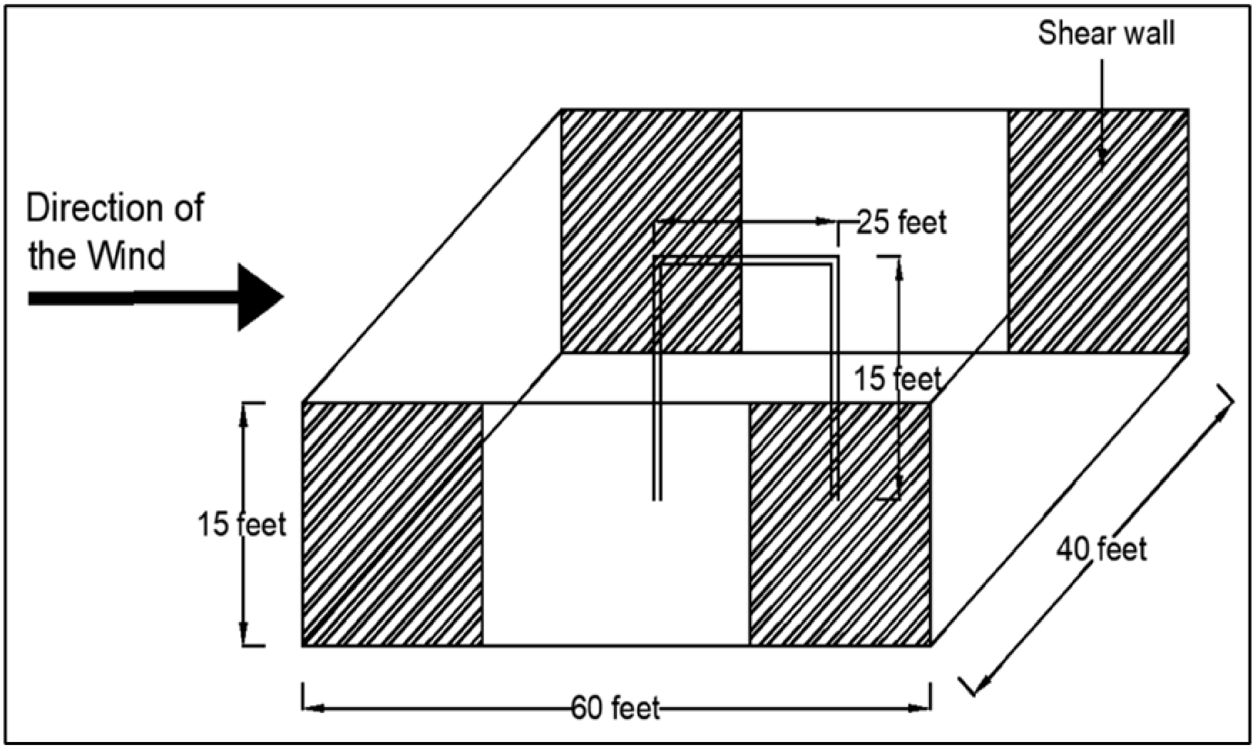
Leeward wall	1.5	-0.3	q_h
Sidewall	1.5	-0.7	q_h

Notes:

a) l=length of the building, b= breadth of the building, h= height of the building/shear wall

Figure 5

Diagram of a Sample Building with Dimension Parameters



Step 7: Calculating the combined wind pressure, p on each building surface (windward, leeward, sidewall)

The design wind pressure for the MWFRS of buildings of all heights in lb/ft^2 was given by Section 27.3 of the code. The wind pressure for the enclosed and partially enclosed rigid and flexible building is obtained by the empirical equation 27.3-1 of the code as below:

$p = q \times \mathrm{G} \times C_p - q_i \times (GC_{pi})$………………. *(Section 27.3, ASCE 7-16)*

where,

q = q_z for windward walls calculated at height z above the ground.

q =q_h for leeward walls, sidewalls and roofs calculated at mean roof height, h.

q_i =$q_h$for windward walls, sidewalls, leeward walls, and roofs of enclosed buildings.

G = Gust effect factor obtained from Step 3.

C_p = External pressure coefficient obtained from Step 6.

GC_{pi} = Internal pressure coefficient obtained from Step 3.

The wind pressure for the Open Buildings with mono slope, Pitched or Troughed Free Roofs was obtained by the empirical equation 27.3-2, ASCE 7-16 as below:

$p = q_h \times G \times C_N$………………… *(Section 27.3-2, ASCE 7-16)*

where,

q_h = velocity pressure at mean roof height h using the exposure defined in section 26.7.3 of the code.

G = Gust effect factor obtained from Step 3.

C_N = Net pressure coefficient obtained from figure 27.3-4 through 27.3-7 of the code.

5.2 Gravity Loads

Among all the varieties of gravity loads, our research considers the following most common types of gravity loads that a residential building is exposed to.

5.2.1 Dead Load

Dead loads are weights of all the construction materials that are incorporated into a building like walls, floors, ceilings, roofs, stairways, finishes, cladding, and similar structural components including the weight of the cranes and system of machines that handles the construction materials. It considers the actual weights of materials and construction which are approved by the authority having jurisdiction in case the definite information is not provided. During designing phase, the weight of fixed service equipment including the maximum weight of the contents of the fixed service equipment are included. Some of the fixed service equipment is electrical feeder lines, heating, ventilating, and air conditioning systems. According to Section 1.3.6, ASCE 7-16, Variable components of the fixed service equipment like liquid contents, movable trays are not used to counteract forces of overturning, sliding and uplift conditions.

All the landscaping and hardscaping materials like soil, plants, walkways, fences, and drainage layer material on the top of the roof are considered as dead loads. For this purpose, the weight is calculated by considering both fully saturated soil with the draining layer and the fully dry soil with draining layer. Also, if the solar panels and ballasted systems that are not permanently attached to the roof are considered as dead loads.

5.2.2 Live Load

Those loads that are acted by the use and inhabitance of the building or other structures excluding the environmental loads and the loads added during the construction are known as live loads. Construction loads include the weight of the construction workers, machines, and appliances. Environmental loads include snow load, wind load, rain load, earthquake load, flood load and dead load.

In the process of selecting the occupancy and the inhabitance for the design of a building, it should be taken care that there is a possibility of changes in the occupancy of the structure. In that case the load relating to the previous case should not be considered. The owner of the building should also be careful that the live load greater than the one approved by the authority having jurisdiction is not in action on any part of the building.

Table 4.3-1, ASCE 7-16 provides the minimum uniformly distributed live loads and minimum concentrated live loads for various occupancy. These values were selected by a group of 25 structural engineers after a series of experiments and reaffirmation (Corotis, Harris, & Fox, 1981). They also confirmed that the floors that were measured for a live load survey were usually below present design value. They recommended that the buildings must be designed to resist maximum loads for some reference period and designated T for 50 years. From the same table the value of uniform live load (L) for residential occupancy for private room with corridors serving is taken as 40 psf.

5.2.3 Roof Live Load

These loads are acting vertically upon the surface of roof. These are those loads that are acted during the process of maintenance of the building by the workers, which includes the materials and machines. It covers those non occupancy related objects that are moveable like plants, saturated landscaping materials and decorative objects. At the same time, it also includes occupancy related live loads such as rooftop decks, assembly areas and vegetative landscapes. Since the roof that we have assumed for the purpose of our research is flat, the value of live roof load (Lr) is taken as 25psf.

6.0 Parametric Analysis

6.1 Matrix of Cases

6.1.1 Building Dimensions and Properties

In this chapter, the values of different variables that are either fixed or varied in the analysis of shear walls are discussed in an elaborate way. Following the mandates of the International Building Code (IBC) section 2305, General Design Requirement for Lateral Force-Resisting Systems, the shear walls, or diaphragms designed and discussed under this research are conducted under the provisions of Sections 2305, 2306 and 2307 of American Wood Council (AWC) Special Design Provisions for Wind and Seismic (SDPWS). ASCE 7-16, Directional Procedure for Wind Load on Buildings: Main Wind Force Resisting System is applied for calculation of deflection on the wooden shear wall.

A building of 40 feet by 60 feet dimension is assumed to be in a Risk Category "II” considering our building represents low risk to human life in the event of failure according to Table 1.5-1, ASCE 7-16. The dimension of the building was chosen near to the average size of a U.S. Single-Family House which was 2473 square feet for 2020 (Residential Buildings Factssheet, 2021). The building has sheer walls of a height of 15 feet on all four sides of the building and has only one story. Since the mean roof height of the building is less than 30 feet, the building falls into the Exposure Category “B”. Also, the ground surface roughness of “B” is assumed to prevail in the upwind direction of the building at 1500 feet, which also makes the building into exposure Category “B”, according to Section 26.7.3 Exposure Categories, ASCE 7-16. The shear wall is assumed to be made up of the panel type Sheathing of Structural I category from Table 2305.2(2), IBC 2021.

Figure 6

Dimension of The Building in An Isometric View

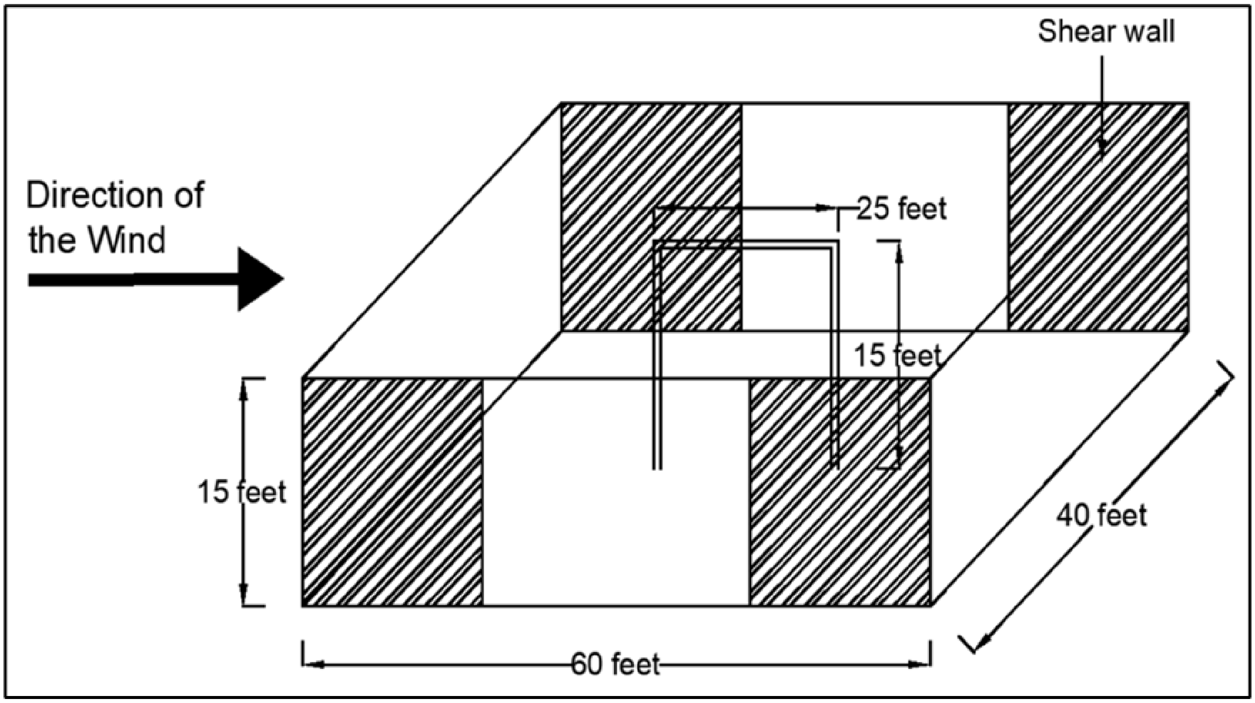
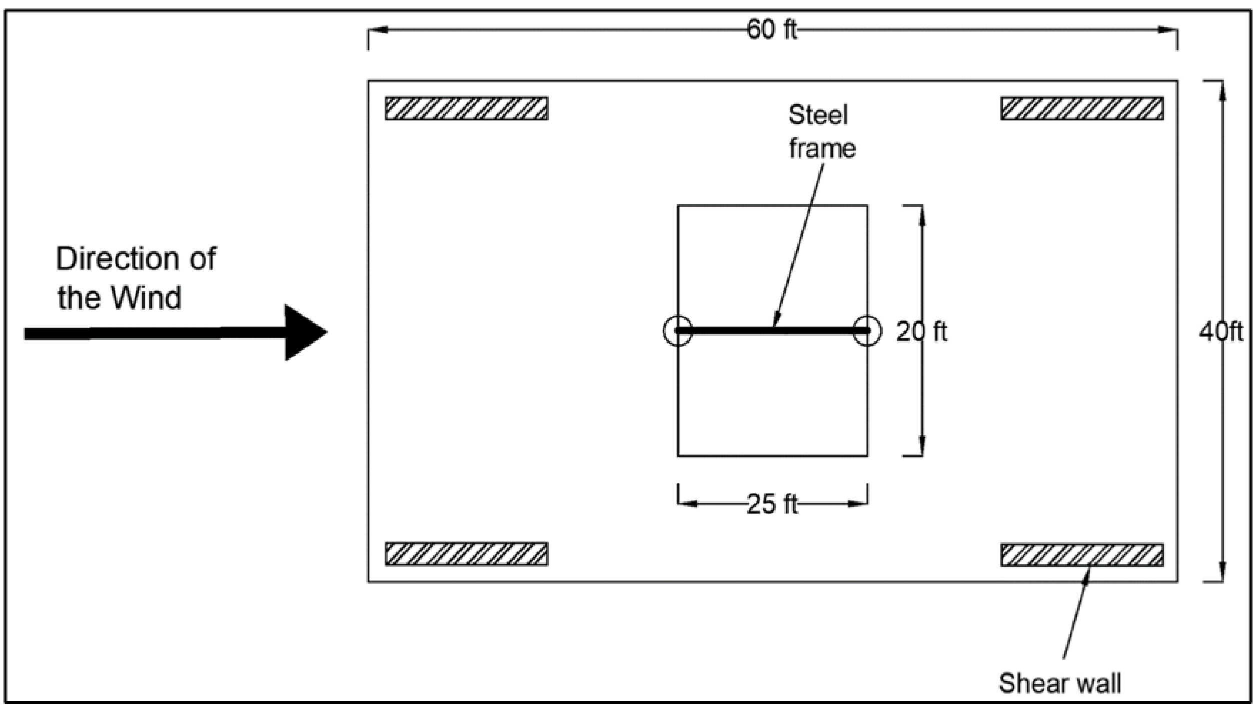


Figure 7

Dimension of The Building in Plan View



6.1.2 *Basic Wind Speed and Ground Snow Load*

The basic wind speed was obtained from Section 26.5.1 ASCE 7-16. The direction of the wind was assumed to be in horizontal direction. It is important to note, wind is taken as a parameter that creates stiffness demand. The following website was also used to extract a 50-year Mean Recurrence Interval wind speed in unit of mph (miles per hour) and ground snow load in (pound per square feet) ATC Hazards by Location (atcouncil.org). Table 3 shows the 13 locations in United States and the corresponding basic wind speed and ground snow load taken into consideration for this research.

Table 2

Locations and Their Corresponding Wind Speed and Ground Snow Load

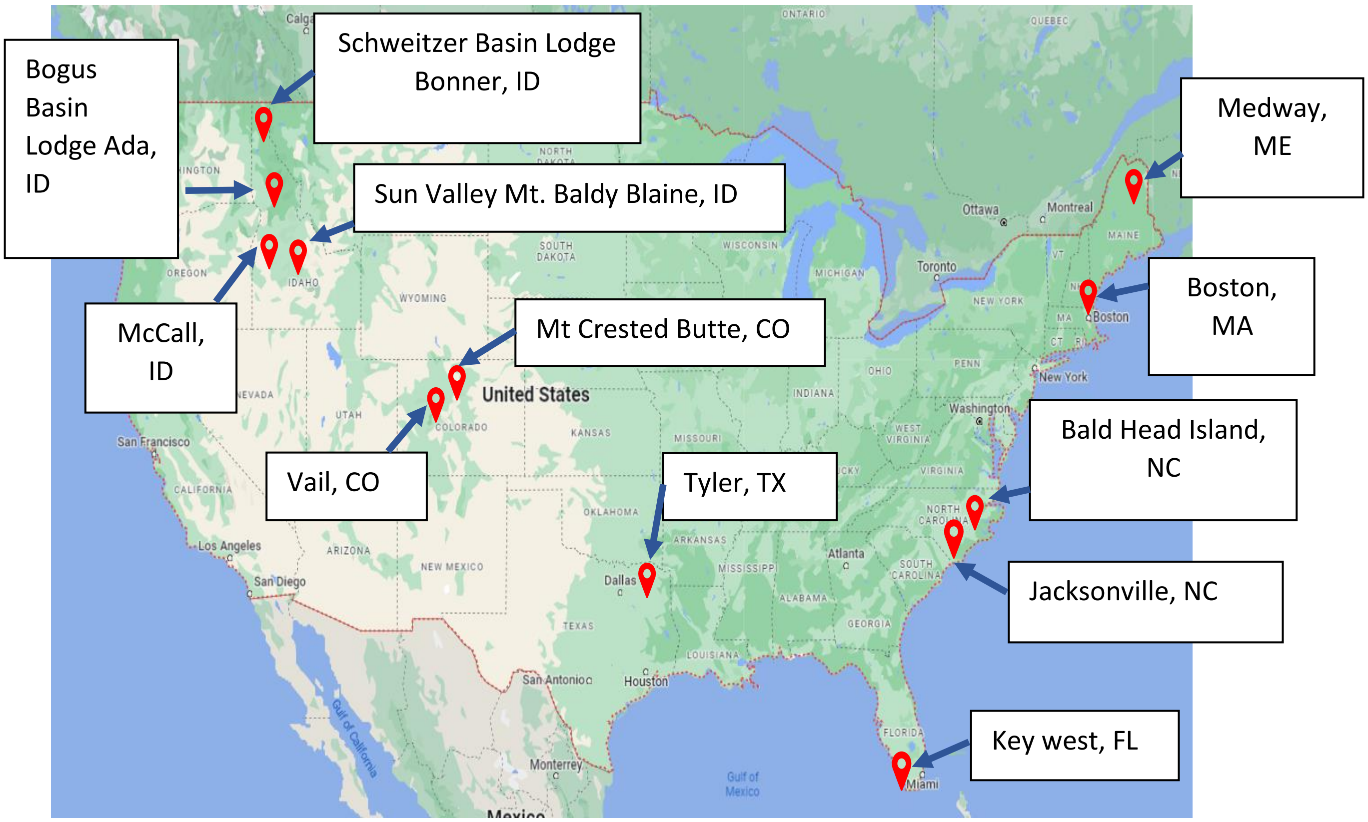
Locations	Basic Wind Speed (mph)	Ground Snow load (psf)
Key west, FL	171	0
Whittier, AK	139	300
Bald Head Island, NC	140	10
Jacksonville, NC	129	10
Boston, MA	110	40
Schweitzer Basin Lodge, ID	97	243
Medway, ME 04460,	98	100
Mt Crested Butte, CO	99	155
Vail, CO 81657	100	175
Sun Valley Mt. Baldy	97	197
McCall, ID 83638,	96	157
Bogus Basin Lodge Ada, ID	96	137
Tyler, TX	97	5

The location was chosen based on the areas with wind speed above 95 mph to the maximum of 171 mph in the entire United States for the mean recurrence period of 50 years. At the same time ground snow load was also taken into consideration as having minimum of 5 psf to the maximum of 300 psf in the United States for a mean recurrence period of 50 years.

The following map shows the locations taken into consideration for this research.

Figure 8

Map showing 13 locations for wind speed and ground snow load



6.1.3 Total Wind Pressure Calculation and Parameters Involved

Some of the standard parameters used for the purpose of calculation of Total wind pressure are as summarized in Table 3 below.

Table 3

Standard Parameters for Total Wind Pressure Calculation

C.N.	Parameters	Symbol	Value	Unit	Reference
1	Air density of the standard atmosphere	D_o	0.08	lb/ft^3	At 101.325 kPa and 15-degree Celsius, density of air is 0.0765 pounds/ft^2 (wikipedia.org)
2	Power law coefficient	α	7		Table 26.11, Terrain Exposure Constants
3	Gradient Height	Z_g	1200	ft	
4	Velocity Pressure Coefficient	K_{air}	74.79	$lbft/ft^2$	Equation C26.10.2, Refer to Note 1
5	Height of each level from the ground	z_t	15	ft	
6	Height of the building	h	15	ft	
7	Wind directionality factor	K_d	0.85		Section 26.6
8	Ground elevation factor	Ke	1		Section 26.9
9	Gust effect factor	G_f	0.85		C26.11, Refer to Note 2
10	Enclosure classification		Enclosed		Section 26.12-Assumption, Refer to Note 3
11	Internal pressure coefficients	GC_{pi}	+/-0.18		Section 26.13, Refer Note 4

Notes:

1. The dynamic velocity pressure was obtained by using Bernoulli's Dynamic pressure law:

It states that the pressure within the fluid (gas or liquid) decreases whenever there is an increase in speed of a moving fluid.

Mathematically (American Society of Civil Engineers, 2016):

$P\ or\ K_{air} = 0.5 \times density(p) \times velocity(v)^2$

where,

$P = K_{air}$= velocity pressure coefficient (lbf/ft^2)

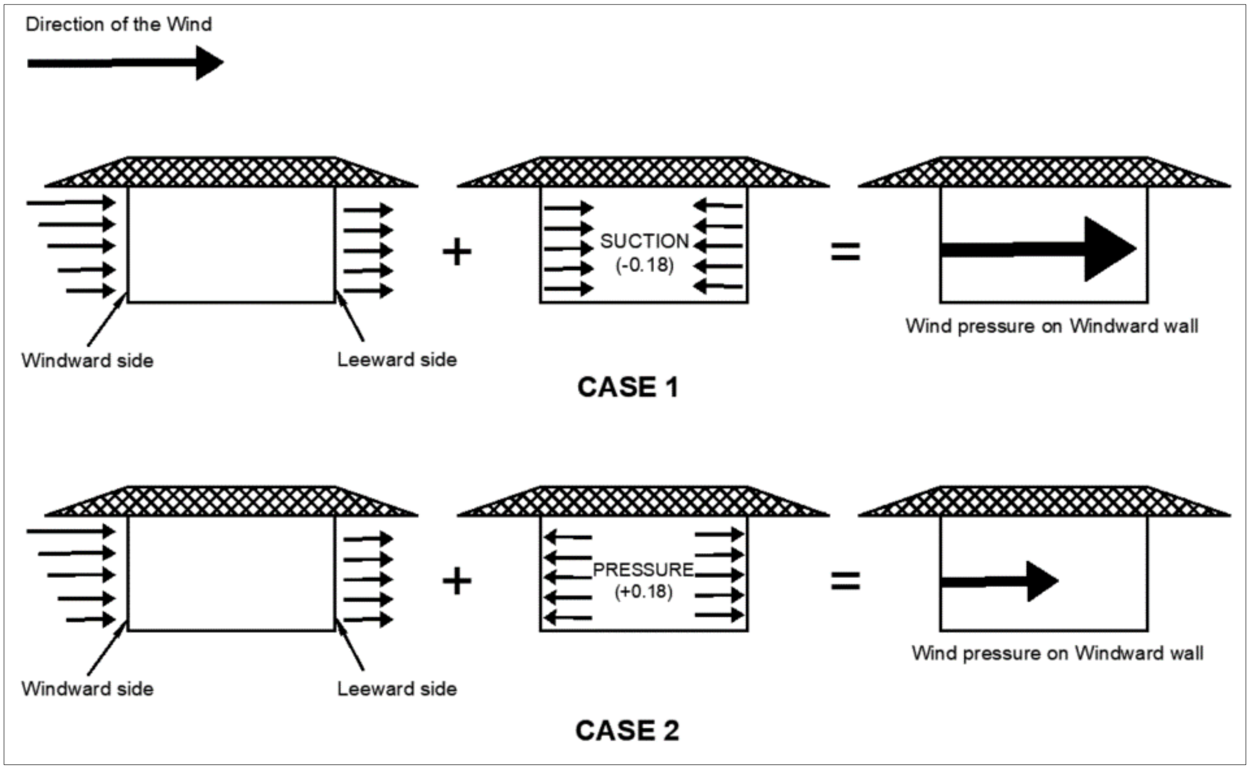
p= density of air=0.002378 lb/ft^3

v= {basic wind speed (mph) x 1.46667} ft/sec

2. A conservative value of the Gust effect factor of 0.85 was applied for rigid buildings.
3. Internal pressure inside a building is highly dependent upon the size and location of the opening in relation to the direction of the wind. The amount of this opening determines the enclosure classification. An enclosed building is one which has sufficient openings in the exterior envelope such that it allows a considerable amount of air inside the building. Partially enclosed are those which have sufficient openings to let the air pass inside but insufficient openings to let the air go outside. Partially open buildings are those that allow air to flow into the building and sufficient opening for the air to flow out of the building.
4. The internal pressure coefficient depends upon the enclosure classification of the building envelope. For an enclosed building, the internal pressure coefficient is -0.18 for suction and +0.18 for internal pressure. Figure 8 below shows the mechanism of how the internal suction and pressure inside a building affects the total wind pressure for the shear walls. For the first case, the internal pressure gets cancelled for suction pressure thereby the total wind pressure for the windward and the leeward wall gets summed. In Second case, the internal pressure that exerts out into the shear walls cancels the pressure acted upon the windward wall, hence decreasing pressure on windward wall.

Figure 9

Internal Pressure Inside an Enclosed Building



Then, the velocity pressure exposure coefficient is calculated and is dependent on the exposure category which is determined from Section 26.10. As mentioned in section 6.1.1 above the exposure category is assumed as "B". The height of the single-story building was assumed as 15 feet and hence following equation is applied:

$$K_z = 2.01(\frac{z}{z_g})^{\frac{2}{\alpha}}$$ …………….. *(Section 26.10, ASCE 7-16)*

where,

K_z= velocity pressure exposure coefficient

z= height of each level from the ground

z_g= gradient height

α= power law coefficient

The value of K_z was obtained as 0.57 for all the 13 location cases.

Then, the velocity pressure (q_z) is calculated at height z, by using the section 26.10.2 as follows:

$q_z = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2$…………………. *(Equation 26.10-1, ASCE 7-16)*

where,

K_z= velocity pressure exposure coefficient

K_{zt}= topographic factor

K_d= wind directionality factor

K_e= ground elevation factor

V = basic wind speed

q_z= velocity pressure at height z

Now the total wind pressure (p) for the windward, leeward and sidewall is calculated by the equation as follows:

$p = q \times G_f \times C_p - q_i \times (GC_{pi})$………………… *(Equation 27.3, ASCE 7-16)*

where,

q = q_z for windward wall calculated at height z above the ground.

q =q_h for leeward wall, sidewall and roof calculated at mean roof height h.

q_i =$q_h$for windward wall, sidewall, leeward wall, and roof of enclosed buildings.

G_f = Gust effect factor obtained from Section 26.11.1. G_f = 0.85

C_p = External pressure coefficient obtained from Table 1

GC_{pi} = Internal pressure coefficient, -0.18 for the case of suction and +0.18 for the pressure as shown in figure 8 above.

The total wind pressure to be considered for shear wall design are summarized in Table 4 as follows:

Table 4

Total Wind Pressure Calculated for Each Location

C.N.	Location	Total wind pressure considered to design the shear wall(psf)
1	Key west, FL	34.19
2	Whittier, AK 99693, USA	22.59
3	Bald Head Island, NC 28461, USA	22.92
4	Jacksonville, NC	19.46
5	Boston, MA, USA	14.15
6	Schweitzer Basin Lodge Bonner, ID, USA	11.00
7	Medway, ME 04460, USA	11.23
8	Mt Crested Butte, CO, USA	11.46
9	Vail, CO 81657, USA	11.69
10	Sun Valley Mt. Baldy Blaine, ID, USA	11.00
11	McCall, ID 83638, USA	10.78
12	Bogus Basin Lodge Ada, ID	10.78
13	Tyler, TX	11.00

6.1.4 Snow Load Calculation and Parameters Involved

Section 7.3, ASCE 7-16 provides the equation to calculate flat roof snow load p_f in lb/ft^2 as follows:

p_f= 0.7$\times$ $C_e \times C_t \times I_s \times p_g$ ………………. *(Equation 7.3-1, ASCE 7-16)*

where,

p_f= flat roof snow load (psf).

C_e= Exposure Factor= 0.9 for Surface Roughness Category "B" for Fully Exposed Roof, Table 7.3-1

C_t = Thermal Factor=1.0, Table 7.3-2

I_s = Importance Factor=1, for risk category II, Table 1.5-2.

p_g = Ground snow load= (0 to 300) psf according to Table 3, Section 7.2-1.

The minimum snow load for low slope roofs (p_m) according to Section 7.3.4 of the code is as follows:

a. For p_g is 20 lb/ft^2 or less

$$p_m = I_s \times p_g$$

b. For p_g exceeds 20 lb/ft^2

$$p_m = 20 \times I_s$$

Then the snow load taken into consideration is the minimum of p_g and p_m. Table 5 shows the snow load calculated for each case.

Table 5

Snow Load Calculation

Case no.	Exposure factor (C_e)	Thermal factor (C_t)	Importance factor (I_s)	Ground snow load (P_g) (lb/ft^2)	Flat roof snow load (P_f) (lb/ft^2)	Minimum Snow load for low slope roofs (p_m)			Snow load (lb/ft^2)
						If p_g is 20 lb/ft^2 or less	If p_g exceeds 20 lb/ft^2	Min. snow load (p_m) lb/ft^2	
1	0.90	1	1	0	0.00	0	0	0	0.00
2	0.90	1	1	300	189.00	0	20	20	189.00
3	0.90	1	1	10	6.30	10	0	10	10.00
4	0.90	1	1	10	6.30	10	0	10	10.00
5	0.90	1	1	40	25.20	0	20	20	25.20
6	0.90	1	1	243	153.09	0	20	20	153.09
7	0.90	1	1	100	63.00	0	20	20	63.00
8	0.90	1	1	155	97.65	0	20	20	97.65
9	0.90	1	1	175	110.25	0	20	20	110.25
10	0.90	1	1	197	124.11	0	20	20	124.11
11	0.90	1	1	157	98.91	0	20	20	98.91
12	0.90	1	1	137	86.31	0	20	20	86.31
13	0.90	1	1	5	3.15	5	0	5	5.00

6.1.5 Shear Wall Design

There are two favored design methods under the American Wood Council's (AWC) 2015 Special Design Provisions for Wind and Seismic (SDPWS) Section 4.3.3.3.4. One of the methods that we have used in this research is the Equal Deflection Method which is based on segmented shear wall approach. In this method, the entire shear wall system is divided into segments of height equal to the height of the floor. In these segments there are separations of doors and windows. The total length of the full height wall is then responsible to resist the shear force. The other method is the perforated shear wall design method. In this method, the openings around the shear wall like doors and windows transfer the load around them to the full height segments. The difference between the Equal Deflection and the Perforated Shear wall method is that in case of perforated shear wall method the smaller segments of openings around doors and windows also take part in resisting the lateral load while only the full height walls take part in lateral load resistance in segmented shear wall method. All the cases that have been considered in designing the shear walls analyzed in this research has been explained in the steps below:

Step 1: Calculation of the wind load on the windward wall:

The total wind load (p) by Allowable Stress Design (ASD) on the Main Wind Force Resisting System (MWFRS) is obtained from the previous section 6.1.3 and multiplied by the area (A) of the windward wall as shown in the equation below:

Total ASD load on the MWFRS = $A \times p$

The Total ASD shear load on MWFRS for each of the locations is summarized in Table 6 below.

Table 6

Total ASD Shear Load on MWFRS and Required Shear Wall Capacity

Case no.	Location	Total wind pressure to be considered to design the shear wall(psf)	Total area on windward side of the wall	Total ASD shear load (wind load) on the MWFRS for the floor in calculation(lbs)	Total length of full height shear wall(ft)	Required shear capacity(plf)
1	Key west, FL	34.19	600	20514.93	17.2	1192.73
2	Whittier, AK 99693, USA	22.59	600	13555.25	17.2	788.10
3	Bald Head Island, NC 28461, USA	22.92	600	13750.99	17.2	799.48
4	Jacksonville, NC	19.46	600	11675.01	17.2	678.78
5	Boston, MA, USA	14.15	600	8489.13	17.2	493.56
6	Schweitzer Basin Lodge Bonner, ID, USA	11.00	600	6601.18	17.2	383.79
7	Medway, ME 04460, USA	11.23	600	6737.98	17.2	391.74
8	Mt Crested Butte, CO, USA	11.46	600	6876.20	17.2	399.78
9	Vail, CO 81657, USA	11.69	600	7015.81	17.2	407.90
10	Sun Valley Mt. Baldy Blaine, ID, USA	11.00	600	6601.18	17.2	383.79
11	McCall, ID 83638, USA	10.78	600	6465.77	17.2	375.92
12	Bogus Basin Lodge Ada, ID	10.78	600	6465.77	17.2	375.92
13	Tyler, TX	11.00	600	6601.18	17.2	383.79

Step 2: Assuming the breadth of each of the section of the shear wall:

The IBC 2021, Table 2503.3.4 gives the minimum shear wall aspect ratio (height/width) for the wind loading as 3.5:1. The height of the shear wall is 15 feet for all the cases which give a minimum breadth of shear wall section as 4.29 ft. (15 feet/3.5=4.29 feet). Hence, the breadth of shear wall was taken as 4.3 feet. Also, the number of sections was assumed to be 4, which gave the total length of shear wall as:

Total length of full height shear wall= 4.3 feet $\times$ 4 = 17.2 feet

Step 3: Calculating the Required Shear Wall Capacity:

The required shear wall capacity was obtained by dividing the total wind load on the windward side of the building by the total length of the full height shear wall as shown in last column of the table 6 above.

Step 4: Selecting the shear wall from APA Table 1 for allowable shear panels.

The shear wall that can resist the shear force as calculated in step 3 above can be selected from Table 1, Allowable shear for APA panel shear walls with framing of Douglas-Fir for wind or seismic loading or International Building Code (Table 2306.4.1) (APA The Engineered Wood Association, 2007). It provides two panel grades for the shear walls, one with APA Structural grade 1 and another with APA rated sheathing/ APA rated siding. For this research purpose, panels of APA Structural Grade I with framing of Douglas-Fir was chosen. Also, these panels are applied directly to the framing. Then according to IBC Table 2306.4.1, the allowable shear capacity is increased by 40% for the wind design as shown in table 7 as wind is a temporary maximum. All the studs used on these panels are of Douglas Fir wood which has same dimension of 2 $\times$ 6 inches and Modulus of Elasticity of 1,410,000 psi. The cross-sectional area of the two studs each on side of the shear panel is 16.50 in^2.

Table 7

Shear Panel and Nail Properties

C.N	Required shear capacity of wall (plf)	Allowable shear capacity (plf)	Allowable shear capacity increased by 40%	Minimum Nominal Panel thickness(T) (in)	Minimum Nail Penetration (in)	Nail size (in)	Nail Spacing (in)	Panel Span rating	Rigidity of Panel G_t(lb/in)
1	1192.73	870	1218	15/32	1.50	0.15	2	32/16	83,500
2	788.10	610	854	3/8	1.38	0.13	2	24/0	77,500
3	799.48	610	854	3/8	1.38	0.13	2	24/0	77,500
4	678.78	505	707	7/16	1.38	0.13	3	24/0	77,500
5	493.55	360	504	3/8	1.38	0.13	4	24/0	77,500
6	383.79	280	392	15/32	1.38	0.13	6	32/16	83,500
7	391.74	280	392	15/32	1.38	0.13	6	32/16	83,500
8	399.78	340	476	15/32	1.50	0.15	6	32/16	83,500
9	407.90	340	476	15/32	1.50	0.15	6	32/16	83,500
10	383.79	280	392	15/32	1.38	0.13	6	32/16	83,500
11	375.92	280	392	15/32	1.38	0.13	6	32/16	83,500
12	375.92	280	392	15/32	1.38	0.13	6	32/16	83,500
13	383.79	280	392	15/32	1.38	0.13	6	32/16	83,500

Step 5: Calculation of Deflection according to Section 2305.3 IBC:

The deflection in the wooden framed shear wall was calculated according to Section 23-2, AWC (American Wood Council) SDPWS (Special Design Provisions for Wood and Seismic) as shown below. The total deflection is the sum of the deflection due to bending, deflection due to shear, deflection due to nail slip and total vertical elongation of wall anchorage system.

$\Delta_{SW} = \frac{8VH^3}{EAB} + \frac{VH}{4G_T} + 0.75HE_N + D_AH/B$……………………….. (*Equation 23-2, IBC)*

where,

A= area of the end-post cross section (in^2).

B= shear wall length (feet).

D_A= total vertical elongation of wall anchorage system (in).

E= modulus of elasticity of end posts (psi).

E_N= staple/nail slips (in).

G_T= rigidity of panel (lbs./in).

H= shear wall height (ft).

V= induced unit shear (lbs./ft).

Δ_{SW}= maximum shear wall deflection by elastic analysis (in).

The deflection due to bending was calculated by the expression $(\frac{8VH^3}{EAB})$. The deflection due to shear is calculated by expression $(\frac{VH}{4G_T})$. The expression $(0.75HE_N)$ calculates deflection due to nail slip and the expression $(\frac{D_AH}{B})$ calculates deflection due to vertical elongation of wall hold down. The deflection at shear wall of each wall segment of was calculated and shown in the Table 8.

Table 8

Calculation of Deflection of Shear Walls

Case	Shear wall segments	Breadth (ft)	Height (ft)	Required shear capacit (plf)	Modulus of elasticity(psi)	Cross section area of stud(ft^2)	Rigidity of panel(G_t) (lbs/inch)	V_{nail} [1]	Nail slip (in)	Hold down slip(inch)[2]	Allowable tensile capacity (lbf)[2]	Design uplift force[3]	D_A(inch)[3]	Deflection due to bending	Deflection due to shear	Deflection due to nail slip	Vertical elongation of wall anchorage	Total deflection Δ (inch)
1	1	4.3	15	1192.73	1410000	16.5	83500	198.79	0.040	0.033	3375	17890.93	0.175	0.322	0.054	0.445	0.610	1.430
	2	4.3	15	1192.73	1410000	16.5	83500	198.79	0.040	0.033	3375	17890.93	0.175	0.322	0.054	0.445	0.610	1.430
	3	4.3	15	1192.73	1410000	16.5	83500	198.79	0.040	0.033	3375	17890.93	0.175	0.322	0.054	0.445	0.610	1.430
	4	4.3	15	1192.73	1410000	16.5	83500	198.79	0.040	0.033	3375	17890.93	0.175	0.322	0.054	0.445	0.610	1.430
2	1	4.3	15	788.10	1410000	16.5	77500	131.35	0.011	0.033	3375	11821.44	0.116	0.213	0.038	0.127	0.403	0.781
	2	4.3	15	788.10	1410000	16.5	77500	131.35	0.011	0.033	3375	11821.44	0.116	0.213	0.038	0.127	0.403	0.781
	3	4.3	15	788.10	1410000	16.5	77500	131.35	0.011	0.033	3375	11821.44	0.116	0.213	0.038	0.127	0.403	0.781
	4	4.3	15	788.10	1410000	16.5	77500	131.35	0.011	0.033	3375	11821.44	0.116	0.213	0.038	0.127	0.403	0.781
3	1	4.3	15	799.48	1410000	16.5	77500	133.25	0.012	0.033	3375	11992.14	0.117	0.216	0.039	0.133	0.409	0.796
	2	4.3	15	799.48	1410000	16.5	77500	133.25	0.012	0.033	3375	11992.14	0.117	0.216	0.039	0.133	0.409	0.796
	3	4.3	15	799.48	1410000	16.5	77500	133.25	0.012	0.033	3375	11992.14	0.117	0.216	0.039	0.133	0.409	0.796
	4	4.3	15	799.48	1410000	16.5	77500	133.25	0.012	0.033	3375	11992.14	0.117	0.216	0.039	0.133	0.409	0.796
4	1	4.3	15	678.78	1410000	16.5	77500	169.69	0.025	0.033	3375	10181.69	0.100	0.183	0.033	0.276	0.347	0.839
	2	4.3	15	678.78	1410000	16.5	77500	169.69	0.025	0.033	3375	10181.69	0.100	0.183	0.033	0.276	0.347	0.839
	3	4.3	15	678.78	1410000	16.5	77500	169.69	0.025	0.033	3375	10181.69	0.100	0.183	0.033	0.276	0.347	0.839
	4	4.3	15	678.78	1410000	16.5	77500	169.69	0.025	0.033	3375	10181.69	0.100	0.183	0.033	0.276	0.347	0.839
5	1	4.3	15	493.55	1410000	16.5	77500	164.52	0.022	0.033	3375	7403.31	0.072	0.133	0.024	0.251	0.253	0.661
	2	4.3	15	493.55	1410000	16.5	77500	164.52	0.022	0.033	3375	7403.31	0.072	0.133	0.024	0.251	0.253	0.661
	3	4.3	15	493.55	1410000	16.5	77500	164.52	0.022	0.033	3375	7403.31	0.072	0.133	0.024	0.251	0.253	0.661
	4	4.3	15	493.55	1410000	16.5	77500	164.52	0.022	0.033	3375	7403.31	0.072	0.133	0.024	0.251	0.253	0.661
6	1	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	2	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	3	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	4	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
7	1	4.3	15	391.74	1410000	16.5	83500	195.87	0.038	0.033	3375	5876.15	0.057	0.106	0.018	0.425	0.200	0.749
	2	4.3	15	391.74	1410000	16.5	83500	195.87	0.038	0.033	3375	5876.15	0.057	0.106	0.018	0.425	0.200	0.749
	3	4.3	15	391.74	1410000	16.5	83500	195.87	0.038	0.033	3375	5876.15	0.057	0.106	0.018	0.425	0.200	0.749
	4	4.3	15	391.74	1410000	16.5	83500	195.87	0.038	0.033	3375	5876.15	0.057	0.106	0.018	0.425	0.200	0.749
8	1	4.3	15	399.78	1410000	16.5	83500	199.89	0.040	0.033	3375	5996.68	0.059	0.108	0.018	0.452	0.205	0.782
	2	4.3	15	399.78	1410000	16.5	83500	199.89	0.040	0.033	3375	5996.68	0.059	0.108	0.018	0.452	0.205	0.782
	3	4.3	15	399.78	1410000	16.5	83500	199.89	0.040	0.033	3375	5996.68	0.059	0.108	0.018	0.452	0.205	0.782
	4	4.3	15	399.78	1410000	16.5	83500	199.89	0.040	0.033	3375	5996.68	0.059	0.108	0.018	0.452	0.205	0.782
9	1	4.3	15	407.90	1410000	16.5	83500	203.95	0.043	0.033	3375	6118.44	0.060	0.110	0.018	0.480	0.209	0.817
	2	4.3	15	407.90	1410000	16.5	83500	203.95	0.043	0.033	3375	6118.44	0.060	0.110	0.018	0.480	0.209	0.817
	3	4.3	15	407.90	1410000	16.5	83500	203.95	0.043	0.033	3375	6118.44	0.060	0.110	0.018	0.480	0.209	0.817
	4	4.3	15	407.90	1410000	16.5	83500	203.95	0.043	0.033	3375	6118.44	0.060	0.110	0.018	0.480	0.209	0.817
10	1	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	2	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	3	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	4	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
11	1	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
	2	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
	3	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
	4	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
12	1	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
	2	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
	3	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
	4	4.3	15	375.92	1410000	16.5	83500	187.96	0.033	0.033	3375	5638.75	0.055	0.101	0.017	0.375	0.192	0.686
13	1	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	2	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	3	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717
	4	4.3	15	383.79	1410000	16.5	83500	191.89	0.036	0.033	3375	5756.84	0.056	0.104	0.017	0.400	0.196	0.717

Notes:

1. Shear capacity of each nail, $V_{nail} = \frac{v}{\frac{12}{nail\ spacing}}$

   where, v= required shear capacity

2. Hold down slip and Allowable tensile capacity for the holders are obtained from the manufacturer's manual.

3. $D_A = \frac{Hold\ Down\ Slip}{Allowable\ tensile\ capacity} \times Design\ Uplift\ force$

Then the weighted average deflection and total stiffness provided by the shear wall in each case was calculated as shown in Table 9 below.

Table 9

Calculation of Weighted Average Deflection and Total Stiffness Provided

Case number	Wall Segment	Length (ft)	Total Length	R*	∑R	R/∑R	V**	Total V	v*** (lbf/ft)	DeflectionΔ (in)	Weighted average deflection Δ(in)	Total Stiffness provided (lbs/in)
1	1	4.3	17.2	0.96	3.822	0.25	5128.73	20514.93	1192.73	1.43	1.43	14343.51
	2	4.3		0.96		0.25	5128.73		1192.73	1.43		
	3	4.3		0.96		0.25	5128.73		1192.73	1.43		
	4	4.3		0.96		0.25	5128.73		1192.73	1.43		
2	1	4.3	17.2	0.96	3.822	0.25	3388.81	13555.25	788.10	0.78	0.78	17348.78
	2	4.3		0.96		0.25	3388.81		788.10	0.78		
	3	4.3		0.96		0.25	3388.81		788.10	0.78		
	4	4.3		0.96		0.25	3388.81		788.10	0.78		
3	1	4.3	17.2	0.96	3.822	0.25	3437.75	13750.99	799.48	0.80	0.80	17266.21
	2	4.3		0.96		0.25	3437.75		799.48	0.80		
	3	4.3		0.96		0.25	3437.75		799.48	0.80		
	4	4.3		0.96		0.25	3437.75		799.48	0.80		
4	1	4.3	17.2	0.96	3.822	0.25	2918.75	11675.01	678.78	0.84	0.84	13914.10
	2	4.3		0.96		0.25	2918.75		678.78	0.84		
	3	4.3		0.96		0.25	2918.75		678.78	0.84		
	4	4.3		0.96		0.25	2918.75		678.78	0.84		
5	1	4.3	17.2	0.96	3.822	0.25	2122.28	8489.13	493.55	0.66	0.66	12847.87
	2	4.3		0.96		0.25	2122.28		493.55	0.66		
	3	4.3		0.96		0.25	2122.28		493.55	0.66		
	4	4.3		0.96		0.25	2122.28		493.55	0.66		
6	1	4.3	17.2	0.96	3.822	0.25	1650.29	6601.18	383.79	0.72	0.72	9209.10
	2	4.3		0.96		0.25	1650.29		383.79	0.72		
	3	4.3		0.96		0.25	1650.29		383.79	0.72		
	4	4.3		0.96		0.25	1650.29		383.79	0.72		
7	1	4.3	17.2	0.96	3.822	0.25	1684.50	6737.98	391.74	0.75	0.75	8997.10
	2	4.3		0.96		0.25	1684.50		391.74	0.75		
	3	4.3		0.96		0.25	1684.50		391.74	0.75		
	4	4.3		0.96		0.25	1684.50		391.74	0.75		
8	1	4.3	17.2	0.96	3.822	0.25	1719.05	6876.20	399.78	0.78	0.78	8788.42
	2	4.3		0.96		0.25	1719.05		399.78	0.78		
	3	4.3		0.96		0.25	1719.05		399.78	0.78		
	4	4.3		0.96		0.25	1719.05		399.78	0.78		
9	1	4.3	17.2	0.96	3.822	0.25	1753.95	7015.81	407.90	0.82	0.82	8583.15
	2	4.3		0.96		0.25	1753.95		407.90	0.82		
	3	4.3		0.96		0.25	1753.95		407.90	0.82		
	4	4.3		0.96		0.25	1753.95		407.90	0.82		
10	1	4.3	17.2	0.96	3.822	0.25	1650.29	6601.18	383.79	0.72	0.72	9209.10
	2	4.3		0.96		0.25	1650.29		383.79	0.72		
	3	4.3		0.96		0.25	1650.29		383.79	0.72		
	4	4.3		0.96		0.25	1650.29		383.79	0.72		
11	1	4.3	17.2	0.96	3.822	0.25	1616.44	6465.77	375.92	0.69	0.69	9424.32
	2	4.3		0.96		0.25	1616.44		375.92	0.69		
	3	4.3		0.96		0.25	1616.44		375.92	0.69		
	4	4.3		0.96		0.25	1616.44		375.92	0.69		
12	1	4.3	17.2	0.96	3.822	0.25	1616.44	6465.77	375.92	0.69	0.69	9424.32
	2	4.3		0.96		0.25	1616.44		375.92	0.69		
	3	4.3		0.96		0.25	1616.44		375.92	0.69		
	4	4.3		0.96		0.25	1616.44		375.92	0.69		
13	1	4.3	17.2	0.96	3.822	0.25	1650.29	6601.18	383.79	0.72	0.72	9209.10
	2	4.3		0.96		0.25	1650.29		383.79	0.72		
	3	4.3		0.96		0.25	1650.29		383.79	0.72		
	4	4.3		0.96		0.25	1650.29		383.79	0.72		

Notes:

1. R*= $\frac{length\ of\ wall\ segment}{length\ of\ longest\ wall\ segment}$

2. V**=$\frac{length\ of\ wall\ segment}{total\ length\ of\ wall\ segments} \times Total\ ASD\ shear\ load\ on\ MWFRS$

3. v***= $\frac{total\ ASD\ shear\ load\ on\ MWFRS}{total\ length\ of\ wall\ segments}$

4. $\text{Weighted average deflection} = 4 * \frac{lenght\ of\ segment}{total\ length\ of\ wall\ segments} \times deflection\ in\ each\ segment$

6.1.6 Calculation of Gravity Load

The gravity loads were computed by Section 2.4.1, ASCE 7-16. The values of the components in this load combination are the same for all the 13 cases.

Table 10

Calculation of Gravity Load

Case no.	Dead load of roof(D) (psf)	Live load on roof (L)(psf)	Live roof load (L_r) (psf)	Snow load(S) (psf)	Load combinations							Gravity load(psi)
					1	2	3	4	5	6	7	
1	27	0	25	0.00	27.00	27.00	52.00	45.75	27.00	45.75	16.20	52.00
2	27	0	25	151.20	27.00	27.00	216.00	168.75	27.00	168.75	16.20	216.00
3	27	0	25	8.00	27.00	27.00	52.00	45.75	27.00	45.75	16.20	52.00
4	27	0	25	8.00	27.00	27.00	52.00	45.75	27.00	45.75	16.20	52.00
5	27	0	25	20.16	27.00	27.00	52.20	45.90	27.00	45.90	16.20	52.20
6	27	0	25	122.47	27.00	27.00	180.09	141.82	27.00	141.82	16.20	180.09
7	27	0	25	50.40	27.00	27.00	90.00	74.25	27.00	74.25	16.20	90.00
8	27	0	25	78.12	27.00	27.00	124.65	100.24	27.00	100.24	16.20	124.65
9	27	0	25	88.20	27.00	27.00	137.25	109.69	27.00	109.69	16.20	137.25
10	27	0	25	99.29	27.00	27.00	151.11	120.08	27.00	120.08	16.20	151.11
11	27	0	25	79.13	27.00	27.00	125.91	101.18	27.00	101.18	16.20	125.91
12	27	0	25	69.05	27.00	27.00	113.31	91.73	27.00	91.73	16.20	113.31
13	27	0	25	4.00	27.00	27.00	52.00	45.75	27.00	45.75	16.20	52.00

We took the values of dead load of the roof and live roof load from the residential structural guide-second edition, which is recommended by the United States department of housing and urban development policy for load combination calculation. According to the residential structural guide 2017 (Coulbourne consulting, 2017), table 3.2, the dead load of the roof was considered as 27 psf. Table 3.4 from the same guide gave us live roof load of 25 psf. Meanwhile the live load on the roof was taken as zero. We have taken the same values of dead load, live roof load and live load on the roof for all the 13 cases which are shown in Table 10. The snow loads were taken from table 5, the load combination for allowable stress design was applied and maximum load combination was taken as gravity load. The load combinations followed are:

1. D
2. D+L
3. D+ $(L_r\ or\ S\ or\ R)$
4. D+ 0.75L+0.75$(L_r\ or\ S\ or\ R)$
5. D+ 0.6W
6. D+ 0.75L+0.75(0.6W) + 0.75$(L_r\ or\ S\ or\ R)$
7. 0.6D+0.6W

where,

D = dead load

L = live load

L_r = roof live load

S = snow load

R = rain load

W = wind load

6.1.7 *Calculating the Required Stiffness of the Bracing and Computing Stiffness Ratio*

In this step, the required stiffness of the bracing of 25 feet long and 15 feet height steel frame was computed. The frame for which the calculation was done is shown in figure 10 below. For this purpose, point bracing method was applied. Point bracing controls the movement at the braced points without direct interaction with adjoining braced points.

First, the total linear load (per feet) over the top of the frame was calculated by multiplying maximum gravity load obtained in Table 10 by the tributary width of the building as shown in the equation below:

$$Total\ linear\ load\ on\ top\ of\ frame(per\ ft) = \frac{Gravity\ load \times (\frac{Width\ of\ the\ building}{2})}{1000}$$

Then the largest axial strength required by a column of this frame within the unbraced length adjacent to the point brace is obtained by multiplying the linear load on the top of the frame by the width of the frame and dividing it by 2 as shown by the equation below:

$$P_r = \frac{Total\ linear\ load\ on\ top\ of\ the\ frame\ \times width\ of\ the\ frame}{2}$$

Where,

P_r= Required axial strength of a column within unbraced length

The required strength of end and the intermediate point of bracing (P_{br}) is obtained by multiplying (P_r) by 0.01.

$P_{br} = P_r \times 0.01$

Now, by using equations A-6-4b, ASCE 7-16, Specifications for structural steel buildings, the required stiffness of the brace is obtained for ASD approach.

$\beta_{br} = \Omega \times (\frac{8 \times P_r}{L_{br}})$……………….. *(Equation A-6-4b, ASCE 7-16)*

For ASD, $\Omega = 2.00$

$L_{br} = Height\ of\ the\ floor(15 feet) \times 12$

Figure 10

Top View Showing the Steel Frame in the Building

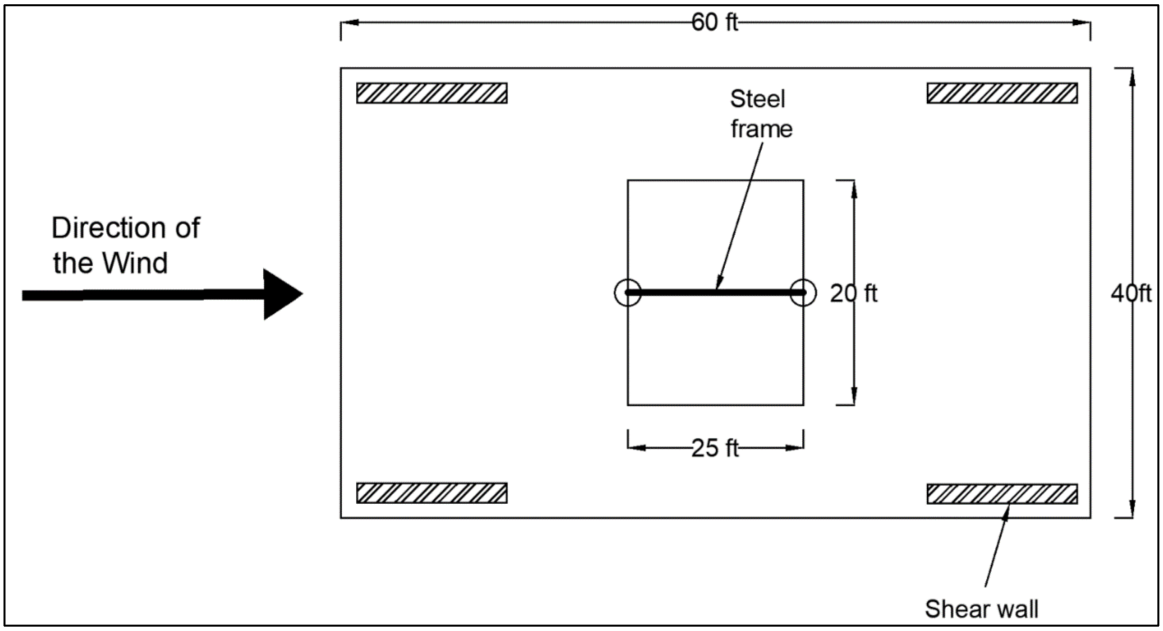
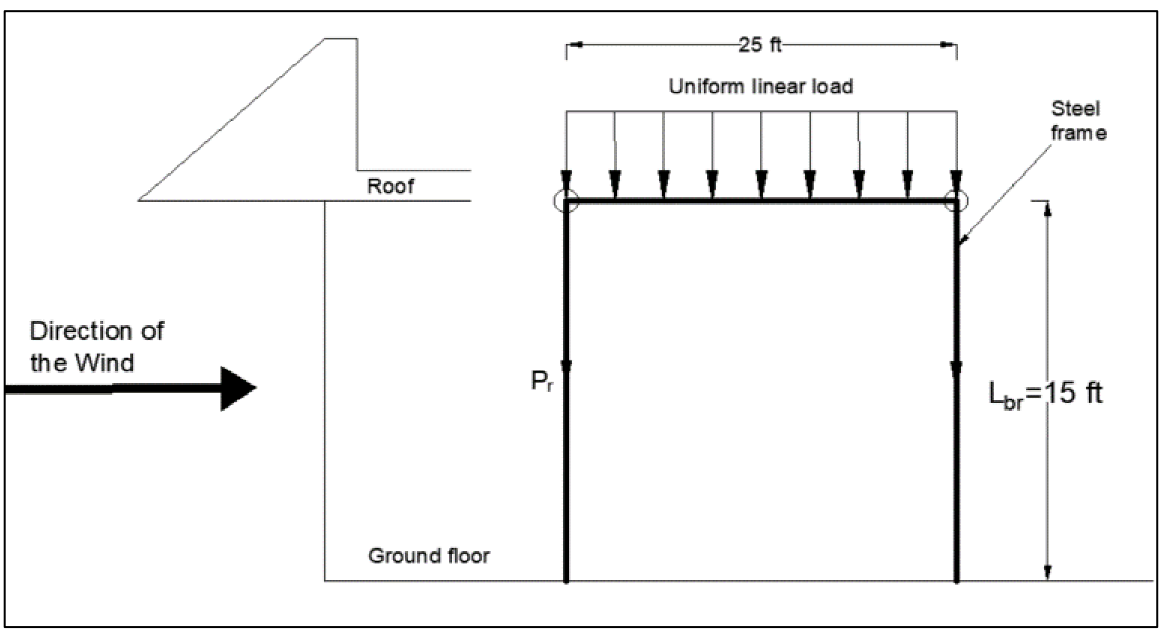


Figure 11

Cross-Sectional View Showing the Steel Frame in the Building



The required stiffness for bracing the frame from table 11 was divided with stiffness provided by the shear walls from Table 9 to compute stiffness ratio as follows:

Table 11

Summary of Required Stiffness for Bracing the Frame in all Cases

Case no.	Width of the frame(ft)	Total linear load over the top of frame (per linear ft)	Largest required axial strength of column within unbraced length (P_r) (kips)	Required strength of end and intermediate point of bracing(P_{br})(kips)	Required stiffness for bracing the frame (lbs/in)
1	25	1.04	13.00	0.13	1155.56
2	25	4.32	54.00	0.54	4800.00
3	25	1.04	13.00	0.13	1155.56
4	25	1.04	13.00	0.13	1155.56
5	25	1.04	13.05	0.13	1160.00
6	25	3.60	45.02	0.45	4002.00
7	25	1.80	22.50	0.23	2000.00
8	25	2.49	31.16	0.31	2770.00
9	25	2.75	34.31	0.34	3050.00
10	25	3.02	37.78	0.38	3358.00
11	25	2.52	31.48	0.31	2798.00
12	25	2.27	28.33	0.28	2518.00
13	25	1.04	13.00	0.13	1155.56

$$Stiffness\ Ratio = \frac{Total\ stiffness\ provided\ by\ shear\ walls}{Stiffness\ required\ for\ bracing\ steel\ frame}$$

As shown in Table 12 below, it was found that the stiffness ratio was greater than 1 for all the 12 cases.

Table 12

Computation of Stiffness Ratio

Case no.	Required stiffness for bracing the frame (lbs/in)	Total stiffness provided by shear walls (lbs/in)	Total stiffness provided/Stiffness required	Is stiffness ratio >1?
1	1155.56	14,343.51	12.41	Yes
2	4800.00	17,348.78	3.61	Yes
3	1155.56	17,266.21	14.94	Yes
4	1155.56	13,914.10	12.04	Yes
5	1160.00	12,847.87	11.08	Yes
6	4002.00	9,209.10	2.30	Yes
7	2000.00	8,997.10	4.50	Yes
8	2770.00	8,788.42	3.17	Yes
9	3050.00	8,583.15	2.81	Yes
10	3358.00	9,209.10	2.74	Yes
11	2798.00	9,424.32	3.37	Yes
12	2518.00	9,424.32	3.74	Yes
13	1155.56	9,209.10	7.97	Yes

6.1.8 Effect of Tributary Area Ratio on the Stiffness Ratio

From section 6.1.5, the wind load on the windward direction of the building is directly proportional to the total stiffness provided by the shear walls. Higher wind load will result in selection of the shear wall with higher stiffness and vice-versa.

$wind\ load\ \propto Total\ Stiffness\ provided\ by\ shear\ walls$

From section 6.1.6, total gravity load is computed from snow load and from section 6.1.7, increase in gravity load increases the stiffness requirement of the braced frame. This justifies snow load is directly proportional to stiffness required for bracing the steel frame.

$$snow\ load \propto Stiffness\ required\ for\ bracing\ the\ steel\ frame$$

Now, we take a ratio of wind load to snow load and from the above two relations, their ratio is proportional to the ratio of stiffness provided to stiffness required. The ratio of stiffness provided to stiffness required is equal to stiffness ratio.

$$\frac{Wind\ Load}{Snow\ Load} = \frac{Total\ Stiffness\ provided\ by\ shear\ walls}{Stiffness\ required\ for\ bracing\ steel\ frame} = Stiffness\ Ratio$$

The wind load and snow load can be broken down into its corresponding tributary area and load per square feet as follows:

$$\frac{Wind\ Load}{Snow\ Load} = \frac{Tributary\ area\ of\ wind \times wind\ load\ per\ square\ feet}{Tributary\ area\ of\ snow \times snow\ load\ per\ square\ feeet}$$

$$Stiffness\ ratio = Tributary\ area\ ratio \times \frac{wind\ load\ per\ square\ feet}{snow\ load\ per\ square\ feeet}$$

Now, to analyze the effect of tributary area of the snow load on the stiffness ratio, tributary area of wind divided by tributary area of snow is taken as tributary area ratio and we multiply the tributary area ratio by 1, 2, 0.5, 0.25, 0.167 and 0.125. We should understand that lower values of tributary area ratio mean larger tributary area of snow.

To understand this better, let's calculate tributary area ratio for the base case. The building is of dimension 40 feet width by 60 feet length and 15 feet height. The tributary area of the wind is the area of the wall in the windward direction, which is 15 ft × 40 ft or 600 square feet. The tributary area for the snow load (as snow load is the only variable in gravity loads which differs in all 13 cases of location) is half of the width of the building multiplied by the length of the steel frame, 20 ft x 25 ft which equals to 500 square feet. This gives the tributary area ratio of wind load to snow load as 600 divided by 500 as 1.2. Multiplying this ratio by 1, 0.5, 0.25, 0.17, 0.125 increases tributary area of snow load, meaning we are increasing the snow load. However, multiplying this ratio by 2 means we are decreasing the tributary area of snow. Their influence on the stiffness ratio is discussed in the headings below.

6.1.9 Tributary Area Ratio Multiplied By 2

The tributary area ratio was multiplied by 2 to see the effect of the decrease in tributary area of snow load and eventually snow load on the stiffness ratio. The following Table 13 summarizes the values of Wind load/Snow load and its corresponding stiffness values for all the 13 cases.

Table 13

Tributary Area Ratio Multiplied By 2

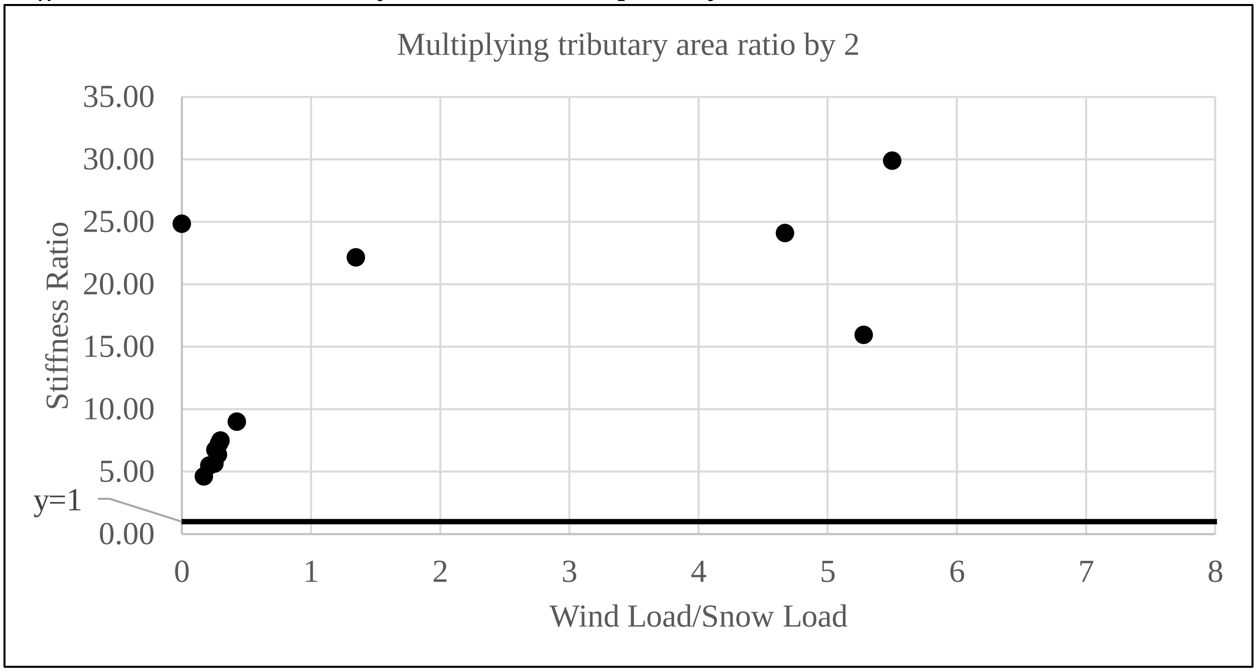
Case No.	Wind load/Snow load	Stiffness Ratio
1	#DIV/0!*	24.83
2	0.29	7.23
3	5.50	29.88
4	4.67	24.08
5	1.35	22.15
6	0.17	4.60
7	0.43	9.00
8	0.28	6.35
9	0.25	5.63
10	0.21	5.48
11	0.26	6.74
12	0.30	7.49
13	5.28	15.94

Note: #DIV/0! Is the case where Wind load is divided by zero Snow load

It was found out that when decreasing the size of the building, the stiffness ratio was more than one for all 13 cases, which means the stiffness provided by the walls was greater than the stiffness required for bracing the steel frames. The following Figure 12 shows the stiffness values are above 1 for all the 13 cases when the tributary area ratio was multiplied by 2.

Figure 12

Stiffness Ratio when Tributary Area Ratio Multiplied By 2



6.1.9.1 Tributary Area Ratio Multiplied By 1

The tributary area ratio was multiplied by 1 to see the effect of using standard size of building on the stiffness ratio.

Table 14

Tributary Area Ratio Multiplied By 1

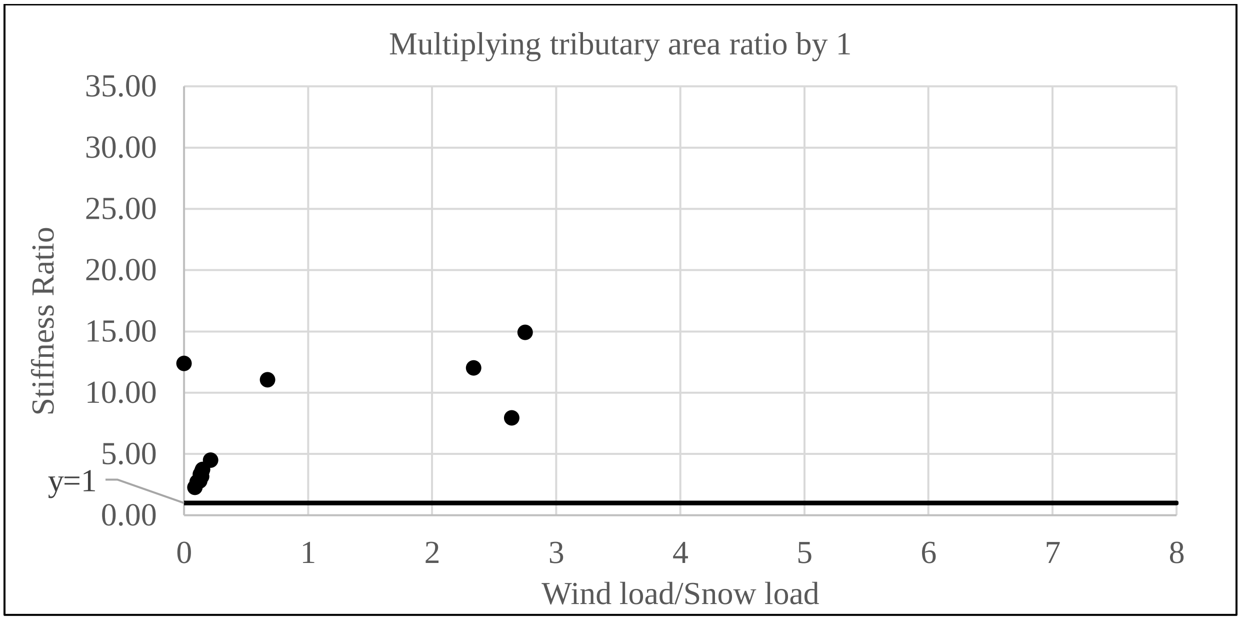
Case No.	Wind load/Snow load	Stiffness Ratio
1	#DIV/0!*	12.41
2	0.14	3.61
3	2.75	14.94
4	2.34	12.04
5	0.67	11.08
6	0.09	2.30
7	0.21	4.50
8	0.14	3.17
9	0.13	2.81
10	0.11	2.74
11	0.13	3.37
12	0.15	3.74
13	2.64	7.97

Note: #DIV/0! Is the case where Wind load is divided by zero Snow load

Table 14 summarizes the values of wind load/snow load and its corresponding stiffness values for all the 13 cases. Multiplying the tributary area by 1 means analyzing the standard size of the building with standard tributary areas, the stiffness ratio was more than one for all the cases, which means the stiffness provided by the walls was greater than the stiffness required for bracing the steel frames. Figure 13 indicates the stiffness ratio values are above 1 for all the 13 cases when the tributary area ratio was multiplied by 1.

Figure 13

Stiffness Ratio when Tributary Area Ratio Multiplied By 1



6.1.9.2 Tributary Area Ratio Multiplied By 0.5

The tributary area ratio was multiplied by 0.5 to see the effect on the stiffness ratio for the building which has double the tributary area of snow or double the snow load. The following Table 15 summarizes the values of Wind load/Snow load and its corresponding stiffness values for all the 13 cases. It was found out that the building which has double the tributary area of snow or double the snow load, the stiffness ratio was more than one for all 13 cases, which means the stiffness provided by the walls was greater than the stiffness required for bracing the steel frames. Figure 14 indicates the stiffness ratio values are above 1 for all the 13 cases when the tributary area ratio was multiplied by 0.5.

Table 15

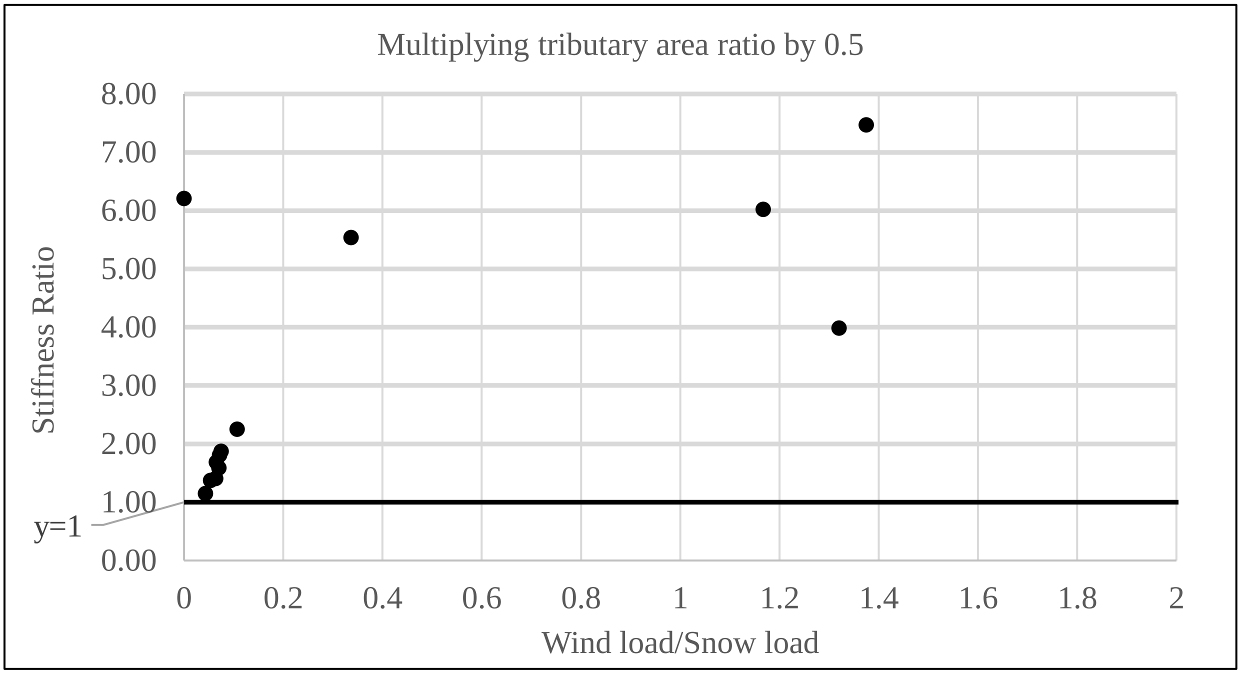
Tributary Area Ratio Multiplied By 0.5

Case No.	Wind load/Snow load	Stiffness Ratio
1	#DIV/0!*	6.21
2	0.07	1.81
3	1.38	7.47
4	1.17	6.02
5	0.34	5.54
6	0.04	1.15
7	0.11	2.25
8	0.07	1.59
9	0.06	1.41
10	0.05	1.37
11	0.07	1.68
12	0.07	1.87
13	1.32	3.98

Note: #DIV/0! Is the case where Wind load is divided by zero Snow load

Figure 14

Stiffness Ratio when Tributary Area Ratio Multiplied By 0.5



6.1.9.3 Tributary Area Ratio Multiplied By 0.25

The tributary area ratio was multiplied by 0.25 to see the effect on the stiffness ratio when tributary area of snow and snow load is increased by four times. The following Table 16 summarizes the values of Wind load/Snow load and its corresponding stiffness values for all the 13 cases. It was found out that the stiffness ratio was more than one for 6 cases. The Stiffness ratio was less than one for 7 cases. The stiffness ratio of less than one means the stiffness provided by the shear walls was less than the stiffness required for bracing the steel frames.

Table 16

Tributary Area Ratio Multiplied By 0.25

Case No.	Wind load/Snow load	Stiffness Ratio
1	#DIV/0!*	3.10
2	0.04	0.90
3	0.69	3.74
4	0.58	3.01
5	0.17	2.77
6	0.02	0.58
7	0.05	1.12
8	0.04	0.79
9	0.03	0.70
10	0.03	0.69
11	0.03	0.84
12	0.04	0.94
13	0.66	1.99

Note: #DIV/0! Is the case where Wind load is divided by zero Snow load

According to Figure 15 the stiffness ratio is more than one for 6 cases and less than one for 7 cases. But when we portrayed this scenario of quadrupled tributary area of snow load, we found out that the width of the tributary area was of a big dimension. It is to be noted that we are restricted to only increasing the width of the tributary area as the other dimension of 20 feet is result of the width of the building and the width of the building should remain unchanged, as it is responsible to determine the wind pressure and sets out criteria for selection of shear wall. As shown in figure 16, the width of the tributary area/frame was obtained as 100 feet.

Figure 15

Stiffness Ratio when Tributary Area Ratio Multiplied By 0.25

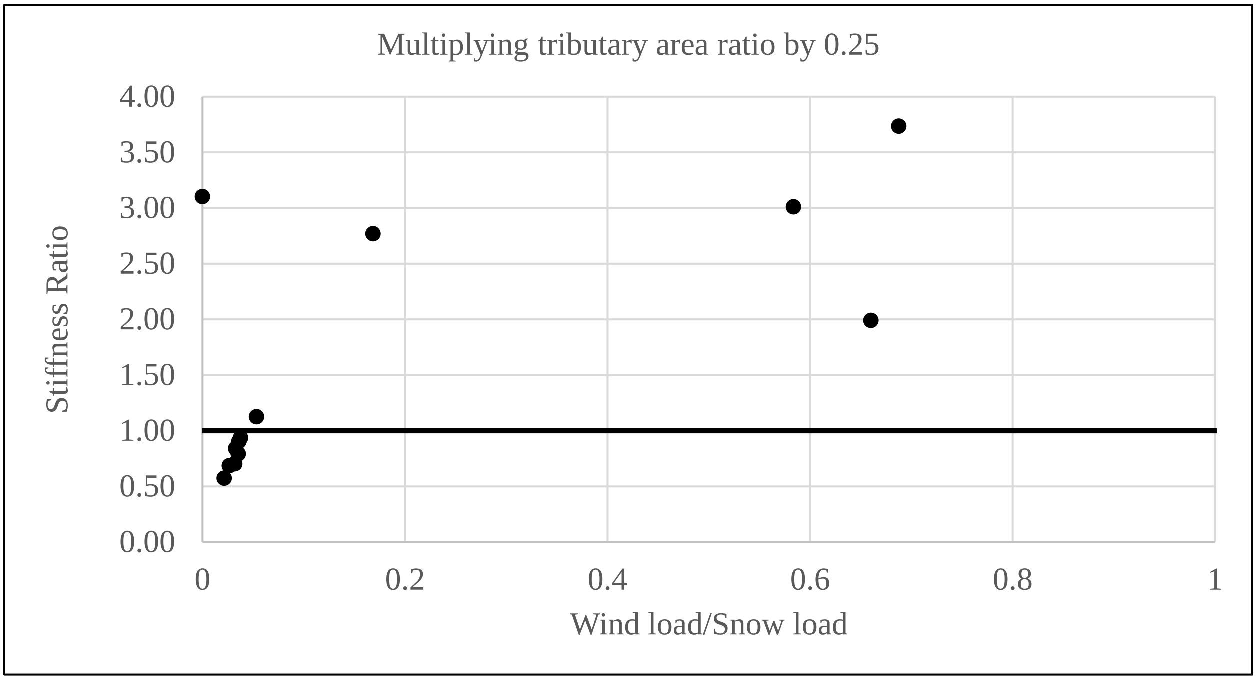
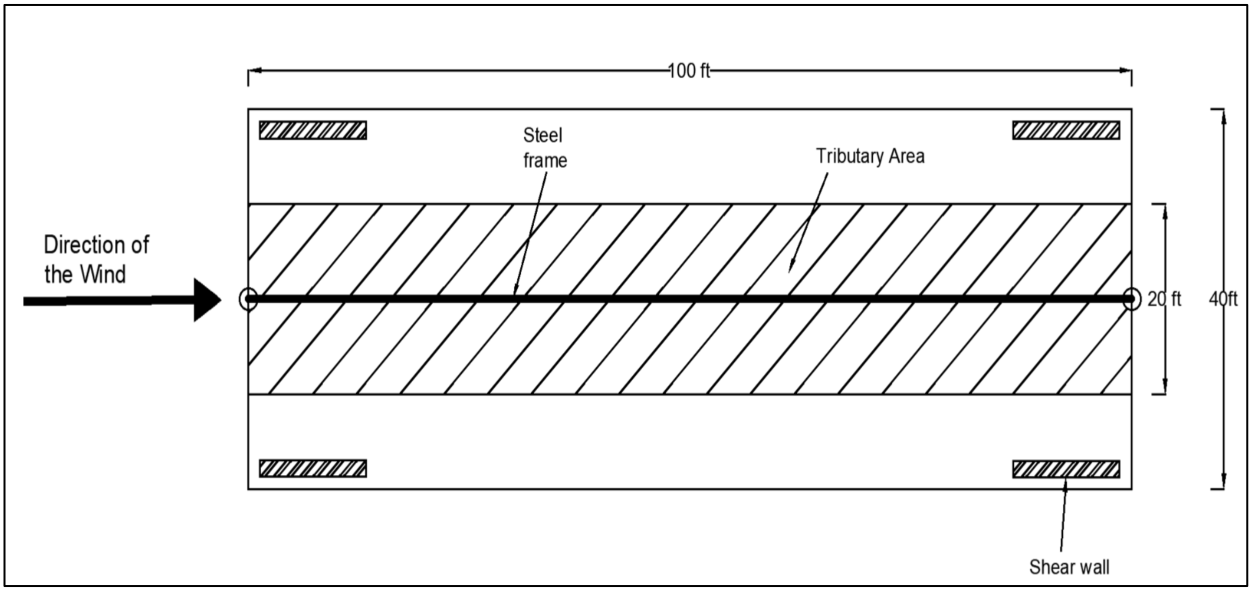


Figure 16

Width of frame when tributary area ratio multiplied by 0.25

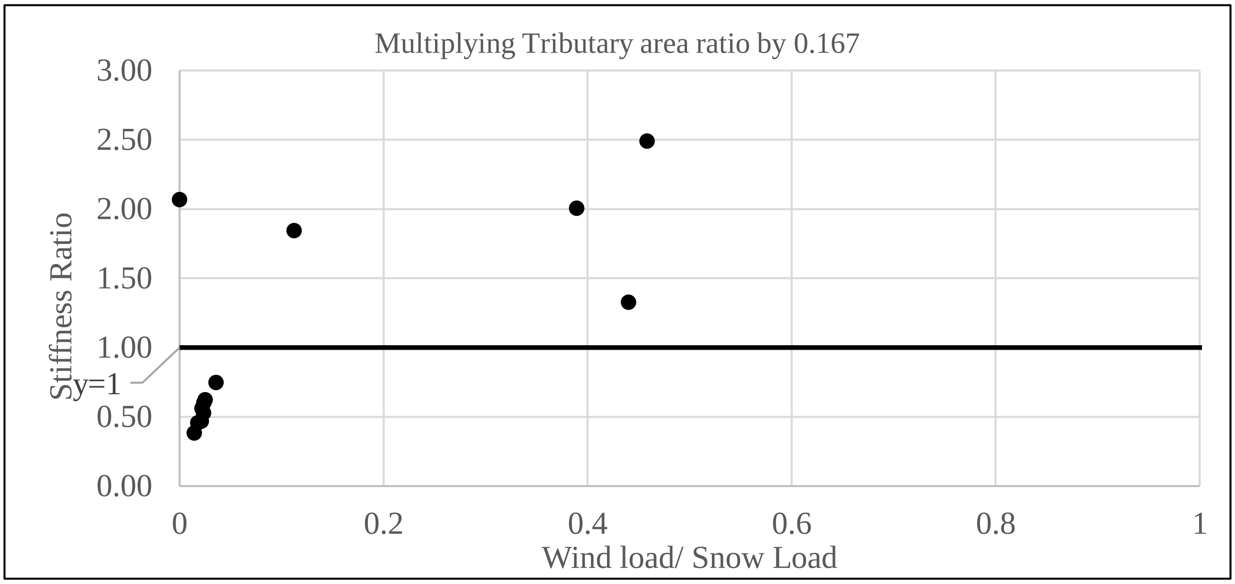


6.1.9.4 Tributary Area Ratio Multiplied By 0.167

The tributary area ratio was multiplied by 0.167 to see the effect on the stiffness ratio when tributary area of snow or snow load is multiplied by 6. Table 17 summarizes the values of Wind load/Snow load and its corresponding stiffness values for all the 13 cases. It was found out that the stiffness ratio was more than one in 5 cases and less than one in 8 cases. The stiffness ratio of less than one means the stiffness provided by the shear walls was less than the stiffness required for bracing the steel frames.

Figure 17

Stiffness *R*atio when *T*ributary *A*rea *R*atio *M*ultiplied *B*y 0.167



According to Figure 17 the stiffness ratio is more than one for 5 cases and less than one for 8 cases. But when we portrayed this scenario of multiplying the tributary area of snow load by six times, we found out that the width of the tributary area was of a big dimension. It is to be noted that we are restricted to only increasing the width of the tributary area as the other dimension (20 feet) is result of the width of the building (40 feet). The width of the building should remain unchanged, as it is responsible to determine the wind pressure and sets out criteria for selection of shear wall. Figure 18 shows the width of the frame was obtained as 150 feet for this case.

Table 17

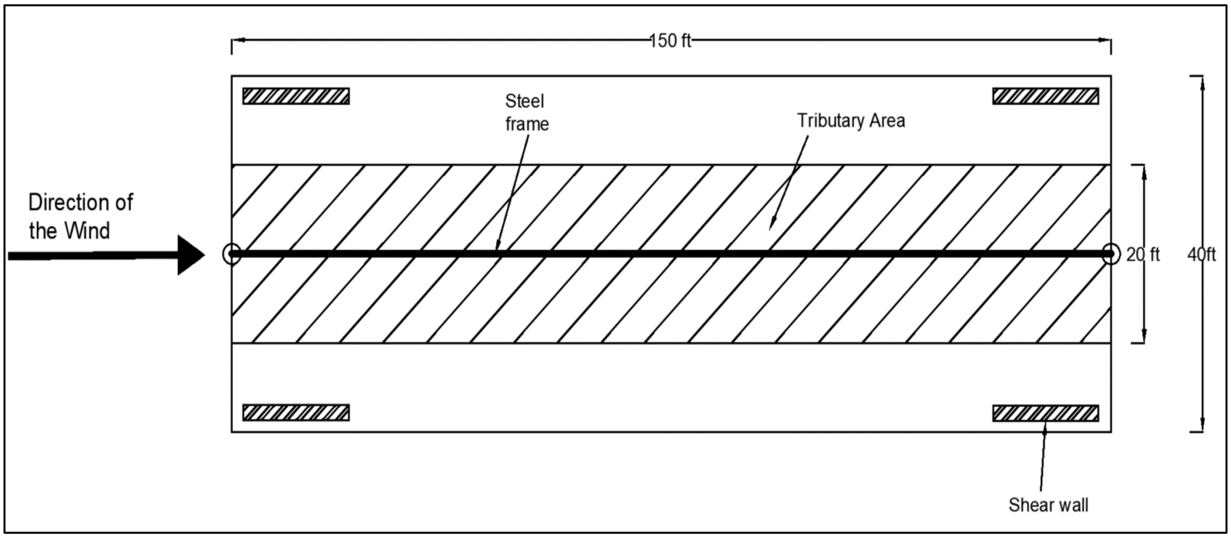
Tributary Area Ratio Multiplied By 0.167

Case No.	Wind load/Snow load	Stiffness Ratio
1	#DIV/0!*	2.07
2	0.02	0.60
3	0.46	2.49
4	0.39	2.01
5	0.11	1.85
6	0.01	0.38
7	0.04	0.75
8	0.02	0.53
9	0.02	0.47
10	0.02	0.46
11	0.02	0.56
12	0.02	0.62
13	0.44	1.33

Note: #DIV/0! Is the case where Wind load is divided by zero Snow load

Figure 18

Width of frame when Tributary Area Ratio Multiplied By 0.167



6.1.9.5 Tributary Area Ratio Multiplied By 0.125

The tributary area ratio was multiplied by 0.125 to see the effect on the stiffness ratio of the building when the tributary area of snow or snow load is multiplied by 8. Table 18 summarizes the values of Wind load/Snow load and its corresponding stiffness ratio values for all the 13 cases.

Table 18

Tributary Area Ratio Multiplied by 0.125

Case No.	Wind load/Snow load	Stiffness Ratio
1	#DIV/0!*	1.55
2	0.02	0.45
3	0.34	1.87
4	0.29	1.51
5	0.08	1.38
6	0.01	0.29
7	0.03	0.56
8	0.02	0.40
9	0.02	0.35
10	0.01	0.34
11	0.02	0.42
12	0.02	0.47
13	0.33	0.99

Note: #DIV/0! Is the case where Wind load is divided by zero Snow load

It was found out that the stiffness ratio was more than one for 4 cases less than one for 9 cases. But when we portrayed this scenario of multiplying the tributary area of snow load by eight times, we found out that the width of the tributary area was of a big dimension. It is to be noted that we are restricted to only increasing the width of the tributary area as the other dimension (20 feet) is result of the width of the building (40 feet). The width of the building should remain unchanged, as it is responsible to determine the wind pressure and sets out criteria for selection of shear wall. Figure 20 shows the width of the frame was obtained as 200 feet for this case.

Figure 19

Tributary Area Ratio Multiplied By 0.125

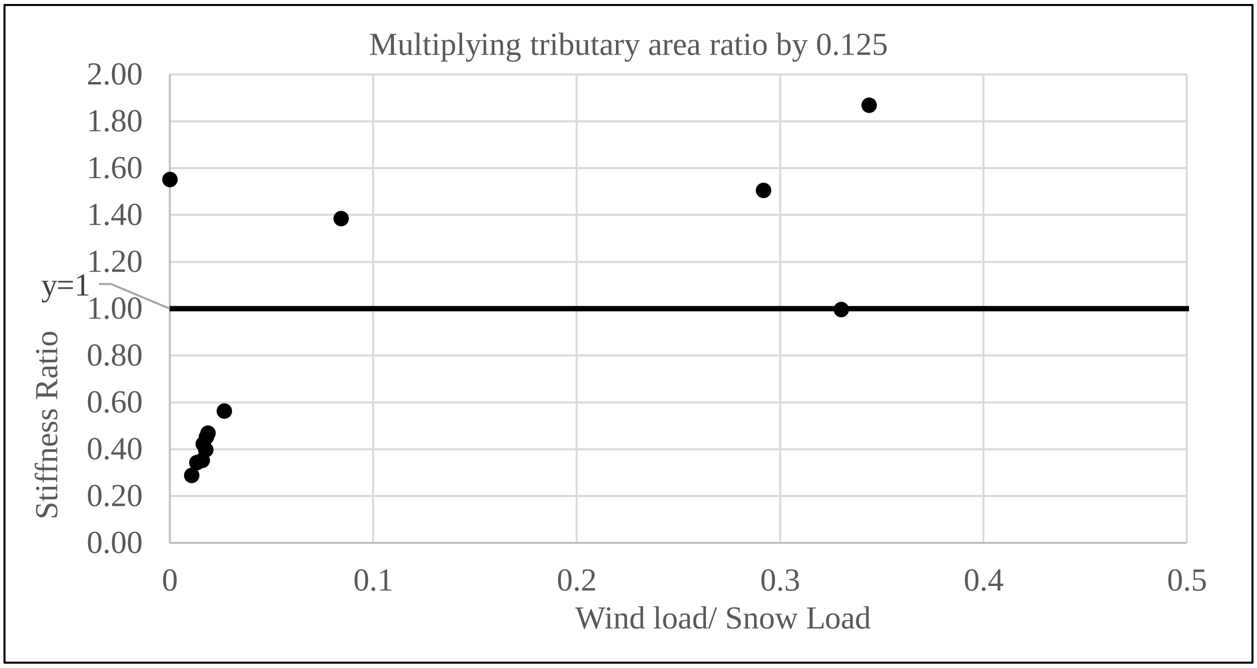
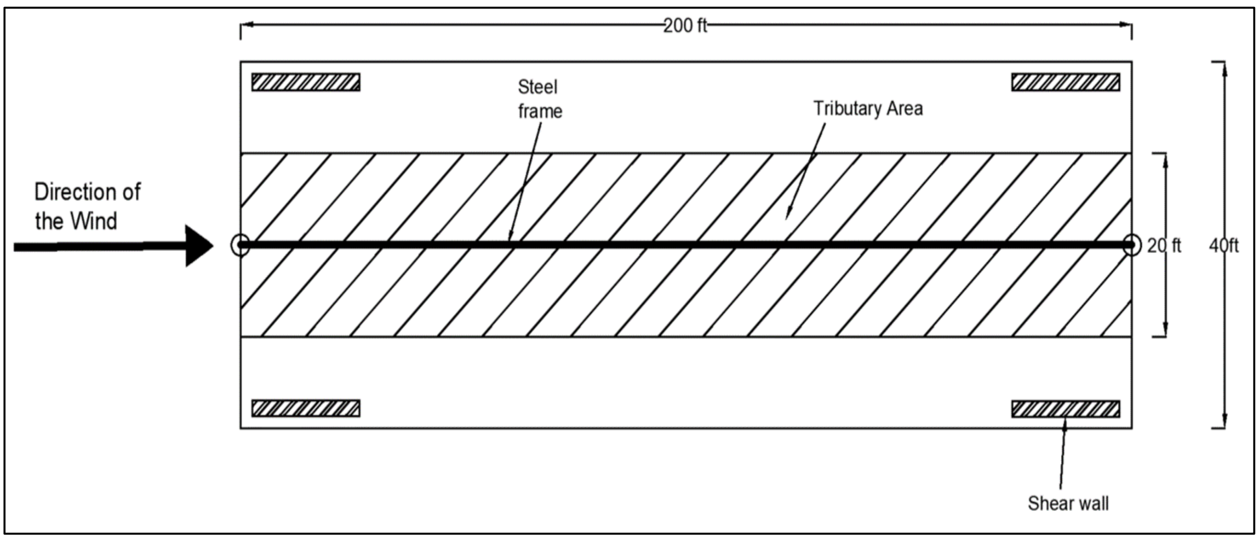


Figure 20

Width of Frame when Tributary Area Ratio Multiplied By 0.125

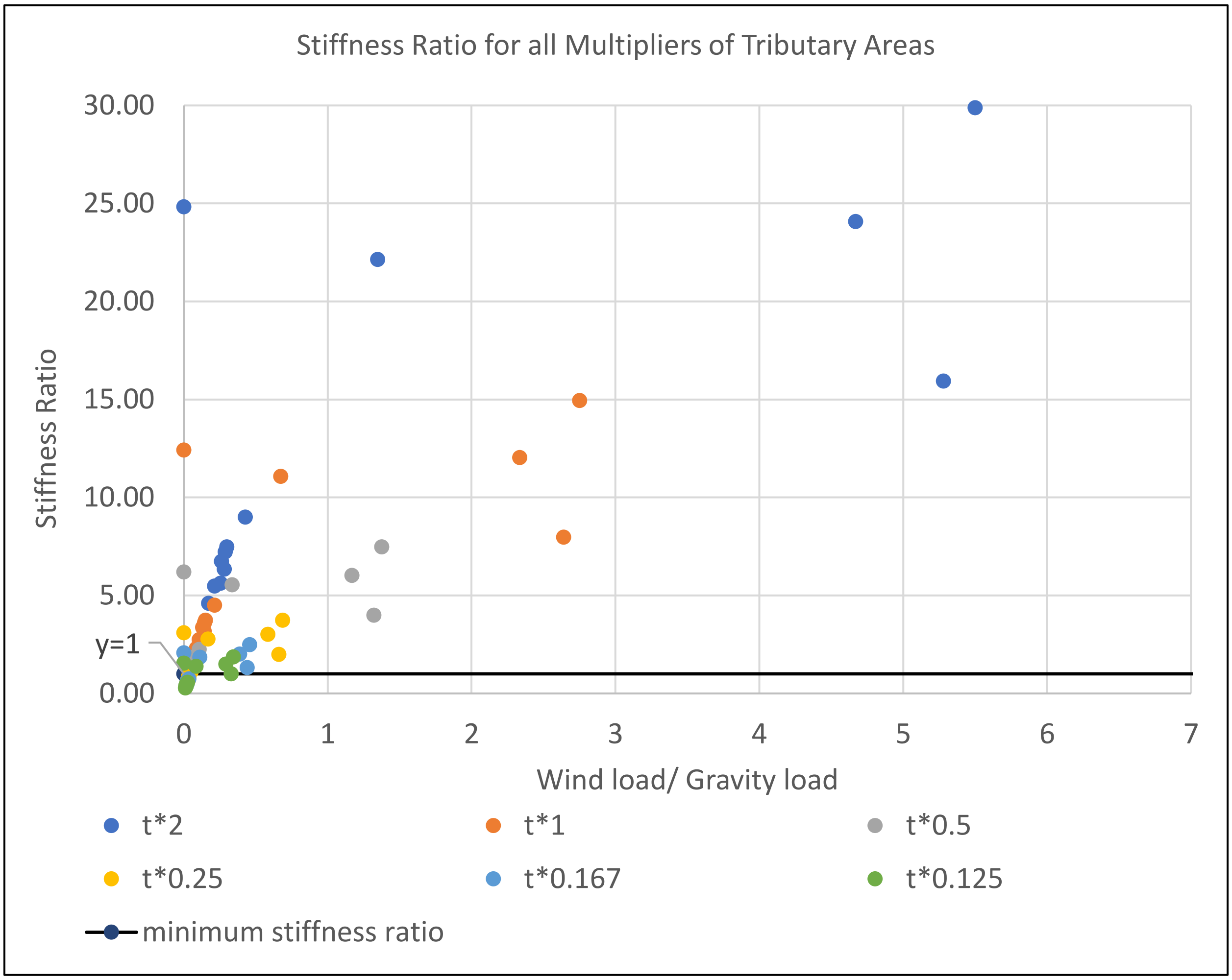


6.2 Summary of Results

By conducting this analytical test, it was observed that for a building of dimension 40 feet by 60 feet, the stiffness provided by the light timber shear walls designed according to the provisions of ASCE 7-16 and IBC 2022 was more than the stiffness required for bracing the steel frame of 25 feet width and 15 feet height embedded inside the same building. This test was conducted for thirteen cases, each of which relates to various locations of United States with varying wind speed and snow load while the gravity loads like dead load and live load of the building were assumed to be constant. This verifies that the light timber shear walls which are designed with reference to the IBC 2022 sections for given wind loading will provide required lateral stiffness to an interior steel frame which are replaced for a load bearing or partition wall in renovation of the building. Further investigation was done to see the effect of the tributary area of snow load on the stiffness of shear walls. For this purpose, a ratio was developed and termed as stiffness ratio, which is the ratio of the stiffness provided by the shear walls to the stiffness required for bracing the steel frame. It was found out that the stiffness ratio was more than one for the building which was half and double the standard tributary area of snow load(20 feet by 25 feet). For the case, which was four times the standard tributary area of snow load, the stiffness ratio was more than one for 6 cases and less than one for 7 cases. For the case which was six times the standard tributary area of snow load, the stiffness ratio was more than one for 5 cases and less than one for 8 cases. For the building which was eight times the standard tributary area of snow load, stiffness ratio was more than one for 4 cases and less than one for 9 cases. Figure 21 shows the summary of the results in a single graph.

Figure 21

Stiffness Ratio for all Parametric Cases



On general observation it can be seen from the graph that most of the points are above the black horizontal line which represents the minimum stiffness ratio of 1. This means that light timber shear walls provide stiffness requirement for bracing steel frame of 25 feet width and 15 feet height, which is placed in the interior of the building.

6.3 Conclusions

The following conclusions were drawn from this research.

1. For a single-story building of standard dimension 40 feet by 60 feet and 15 feet height, the stiffness provided by the shear walls which is most important part of the lateral load resisting system, designed according to the provisions of ASCE 7-16 and IBC 2021 has enough

stiffness to provide lateral stiffness to a steel frame which is typically placed inside the building during renovation of the structure.

2. On further analyzing the effect of the tributary area of snow load on the stiffness of the shear walls, it was found that the same shear walls would be stiff enough to provide bracing for the steel frames, up to double the standard tributary area. The standard tributary area was considered as 20 feet by 25 feet. This further ensures structural safety while performing renovation work in the building where there is modification or removal of interior load bearing wall with steel frames. In this analysis, the only gravity load that fluctuated was snow load while the other gravity loads like Dead loads and Live loads were kept constant. We consider internal pressure as suction or negative pressure, if internal pressure is considered as positive then there will be uplift pressure on the roof. The uplift on roof might reduce the gravity loads and hence reduce the stiffness demand of steel frames and resulting in increased stiffness ratio. Seismic loading was not considered as a part of the lateral loading for this research. It might increase the lateral load and hence a shear wall of higher stiffness would be designed and hence increasing the stiffness ratio.
3. The cases where the tributary area ratio was multiplied by 0.25, 0.167 and 0.125, which had stiffness ratio less than one, the ratio of wind load to snow load was less than 0.05. This conclusion helps to identify if the Stiffness provided by the shear walls will be enough for the steel frames just by finding the ratio of Wind load to Snow load.

6.4 Design Recommendations

The shear wall designed based on chapter 27, ASCE 7-16 and chapter 23, IBC 2021 provisions, were stiff enough for fulfilling bracing requirements of interior steel frame of a single-story building (40 feet width by 60 feet length and 15 feet height), this research paper recommends no further bracing is required for the interior steel frame.

Additionally, if internal pressure is considered as positive then the uplift on the roof will reduce required stiffness by reducing the gravity load. The inclusion of seismic load as lateral load will drive up the stiffness ratio. All these factors concluded that the shear walls designed using these IBC and ASCE 7-16 provisions have enough stiffness for the steel frames placed at the interior of buildings, located at any regions of The United States. However, for some cases where the stiffness provided was not enough and these were the cases where tributary areas were multiplied by 0.25, 0.167 and 0.125. These tributary areas are impractical as the steel frames were 100ft, 150ft, 200ft for timber buildings of dimension 40ft by 100ft, 40ft by 150ft and 40ft by 200ft. This implied we don’t need to worry about the results of cases where tributary areas were multiplied by 0.25, 0.167 and 0.125.

7.0 Conclusion

7.1 Summary of Results

Based on the numerical tests conducted in chapter six, it was concluded that the stiffness provided by light timber shear walls designed according to the provision IBC 2022 was more than the stiffness required for bracing of the steel frame embedded inside the building. This test was conducted for a building of dimension 40 feet by 60 feet and height of 15 feet. The steel frame had a width of 25 feet and a height of 15 feet and was embedded parallel to the longest side of the building. This test was true for 13 cases, each of them representing different locations of the United States with varying wind speeds and snow loads. Meanwhile, the gravity load of the building like dead load and live load was assumed to be constant.

Further analysis was done to check if this satisfies all the locations. For this purpose, a ratio of stiffness provided by the shear walls to the stiffness required for bracing the steel frame was created and termed as stiffness ratio. The stiffness ratio criteria satisfied for cases which was half and up to double the standard tributary area of snow or standard snow load. For the building, which had four times the standard tributary area of snow, the stiffness ratio was more than one for 6 cases and less than one for 7 cases. For the building, which had four times the standard tributary area of snow, the stiffness ratio was more than one for 5 cases and less than one for 8 cases. For the building, which had four times the standard tributary area of snow, stiffness ratio was more than one for 4 cases and less than one for 9 cases.

7.2 Conclusions

Buildings and structures in high wind areas are always at elevated risk of being overwhelmed and might cause damage to life and property. Shear walls are considered as the backbone of Lateral Load Resisting System. It’s very common in renovation projects to replace the partition walls with steel frames, but no research has been done to check if the shear walls have enough stiffness to brace these steel frames. Hence, in this research we took a single-story building with the dimensions 40 feet by 60 feet and 15 feet height as a standard size and designed light timber shear walls for it according to the provisions of ASCE 7-16 and IBC 2021. It was found out that the light timber shear walls have enough stiffness to provide bracing requirements of the steel frames placed in the interior of the building. This was found true for most of the extreme to moderate regions of the United States in terms of wind speed and snow load. Other gravity loads like live load and dead loads were assumed to be constant. Additionally, we considered the internal pressure as suction or negative pressure, if internal pressure is considered as positive then there will be uplift pressure on the roof. The uplift on the roof might reduce stiffness demand by reducing the gravity load. Seismic load has not been considered while calculating the lateral load, but it might increase the stiffness demand resulting in selection of a shear wall of high stiffness hence increasing the stiffness ratio.

The conclusion of this research is that the chances of severe damage to the structural components of the building undergoing structural renovation is incredibly low, as these light timber shear walls have enough stiffness to encounter the lateral load i.e., wind load on the building. Further analysis was done to determine the extent of the snow load or tributary area of snow load, to check if the same shear walls would meet the demand for interior steel frames. It was found that the same shear walls would be stiff enough for the interior steel frames up to double the snow load or tributary area of snow (standard tributary area dimension being 20 feet by 25 feet). For some of the cases the stiffness ratio was below 1 when the tributary areas were multiplied by 0.25, 0.167 and 0.125. But these tributary areas are impractical as the width of the frame would be 100ft, 150ft and 200ft for timber buildings 40ft by 100ft, 40ft by 150ft and 40ft by 200ft. It was also found out that the cases where the tributary area ratio was multiplied by 0.25, 0.167 and 0.125, which had stiffness ratio less than one, the ratio of wind load to snow load was less than

0.05. This conclusion helps to identify if the Stiffness provided by the shear walls will be enough for the steel frames, by finding the ratio of Wind load to Snow load.

This conclusion saves the cost of renovation projects by reducing the time, material, and labor cost for bracing of steel frames. Additionally, a lot of extra floor space is provided to the area where the steel frames are placed. Saving space brings added functionality and enhanced essence to the floor space.

7.3 Design recommendations

The shear walls designed based on chapter 27, ASCE 7-16 and chapter 23, IBC 2021 provisions, were stiff enough for fulfilling bracing requirements of interior steel frame placed inside a single-story building (40 feet width by 60 feet length and 15 feet height). This research paper recommends no further bracing is required for the interior steel frame. The shear walls designed were stiff enough for the interior steel frames from extreme to moderate wind speed and snow fall regions of the United States. Also, the shear walls selected by using these provisions have enough stiffness for steel frames for up to double the standard tributary area of snow or standard snow load (standard tributary area dimension being 20 feet by 25 feet). Hence, interior steel frames which are replaced for interior partition walls are recommended not to be braced necessarily. The interior positive pressure inside the building creates an uplift on the roof and might reduce stiffness demand by reducing the gravity load. At the same time, the inclusion of seismic load as lateral load will drive up the stiffness ratio. All these factors concluded that the shear walls designed using the IBC and ASCE 7-16 provisions have enough stiffness for the steel frames placed at the interior of buildings, located at any regions of The United States. We found out some cases where the stiffness provided was not enough and these were the cases where tributary areas were multiplied by 0.25, 0.167 and 0.125. But these tributary areas are impractical as the width of the frame would be 100ft, 150ft and 200ft for timber buildings 40ft by 100ft, 40ft by 150ft and 40ft by 200ft.

7.4 Future Work

The effect of the varying gravity loads like (dead loads and live loads) to stiffness ratio would be useful information. Also, the study of the effect of positive internal pressure resulting in uplift on the roof, in relation to the stiffness ratio would be valuable.

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