TEMPORARY WIND BRACING OF MASONRY STRUCTURES

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Introduction

The ability of masonry to carry vertical loads has been well established over the years. Masonry, when properly designed, can resist lateral loads of large magnitude provided that the structure is completed at the time when the lateral loading occurs.

Masonry structures, however, are very susceptible to lateral loads resulting from winds acting on the structure prior to installation of the roof system. Free standing masonry walls have been frequently blown over, resulting in the loss of materials, injury to workers and increase of construction costs.

The bracing required to secure walls from blowing over is a function of the following factors:

(a) type of wall as it relates to self weight

(b) thickness of the wall as it relates to (overturning) moment

(c) height of the wall as it relates to the overturning moment induced by the wind.

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(d) location of the wall in the structure as it relates to exposure to wind and to variation of pressure on the wall with height

(e) geographical location of the building as it relates to the expected wind velocities and to resulting pressures on the wall.

In this paper some aspects of wind-induced loads on free standing walls are examined and a procedure for designing temporary wind bracing is presented.

Wind Induced Load

When wind strikes a free standing wall, the wind flow perpendicular to the wall is forced to diverge and pass around the edges. The direction and magnitude of the original wind velocity is therefore altered by the encounter with the wall and causes changes in pressure. Stagnation pressure is produced near the centre of the wall, but there is an increasingly steep pressure gradient towards the edges.

Behind the wall the streamlines of flow are unable to come together immediately, thus reducing the pressure below the surrounding pressure. As a result of this reduction in pressure a suction is created behind the wall. The result of the two forces acting on the wall gives rise to a moment which can be large enough to cause a "blow over".

For walls with openings such as doors and windows or where other structures are near, the lateral load on the wall is more complex. An exact evaluation of the effect of wind on a structure requires extensive and expensive wind tunnel experiments supplemented by field data relating to location of the building, and records of wind patterns and velocities extending back over a large number of years.

For common structures it has been customary to base the design on information provided by authorities, relating to the maximum resulting lateral load induced by wind at the particular area where the building will be constructed. Figure 1 shows wind pressures in lb per square foot for the province of Alberta, Canada.

Once the wind induced load on a wall is established, designing for this load is not a difficult task. When the structure is completed the walls are braced at every floor level, thus the maximum moment induced by the wind on the wall is reduced from $wh^2/2$ to $wh^2/8$, where

w = wind induced load in lb/ft^2

h = height of the wall

The presence of vertical load also helps to increase the resisting moment.

As a result of support conditions partially completed structures are more susceptible to wind than completed ones. Wind induced failures are not a problem for masonry alone. Partially completed steel structures and precast panel concrete structures have been known to fail during construction as a result of wind loading.

Preventing collapse of masonry walls caused by wind is a relatively easy and inexpensive process. Temporary

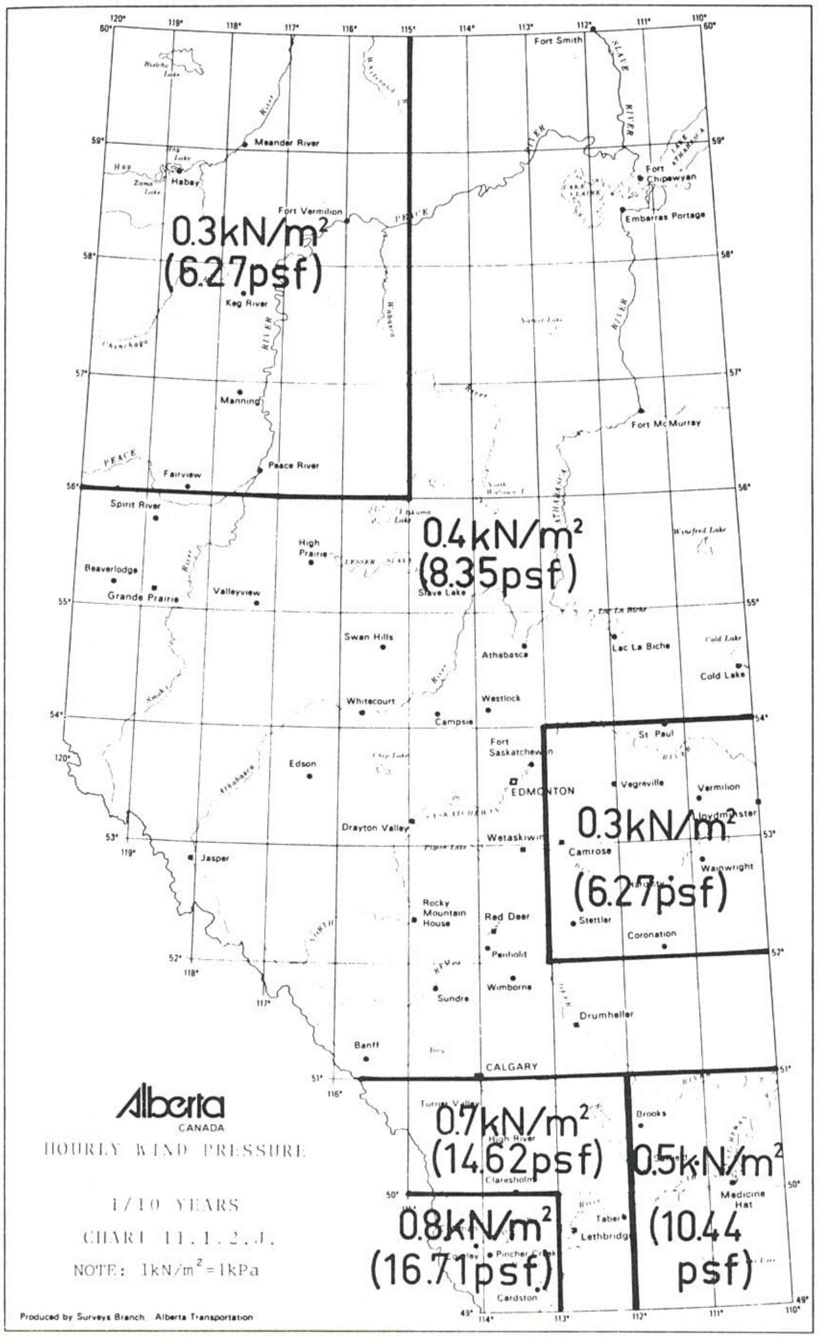

Figure 1 Wind Pressure Chart for the Province of Alberta

bracing placed at the appropriate locations and erected properly will provide adequate safeguard against the most probable wind. It should be pointed out that no structure is 100% safe for any kind of loading. However, it is beyond the scope of this paper to examine failures in a probabilistic manner.

General Concepts

The total wind load acting on a wall is a function of the exposed area. The overturning moment resulting from this force is, in addition, dependent in the height of the wall.

Walls with the same exposed area but different heights have the same force acting upon them, but the overturning moment is much larger on the higher wall.

Consider for example, the walls shown in Figure 2. Both walls have the same area exposed to the wind. Both walls are free standing and in the same geographical area.

The overturning moment per foot of wall is

$$M_A = \frac{wh^2_A}{2}$$

for wall A and

$$M_B = \frac{wh^2_B}{2}$$

for wall B.

If $h_A = 1.5h_B$ then $M_A = 2.25M_B$

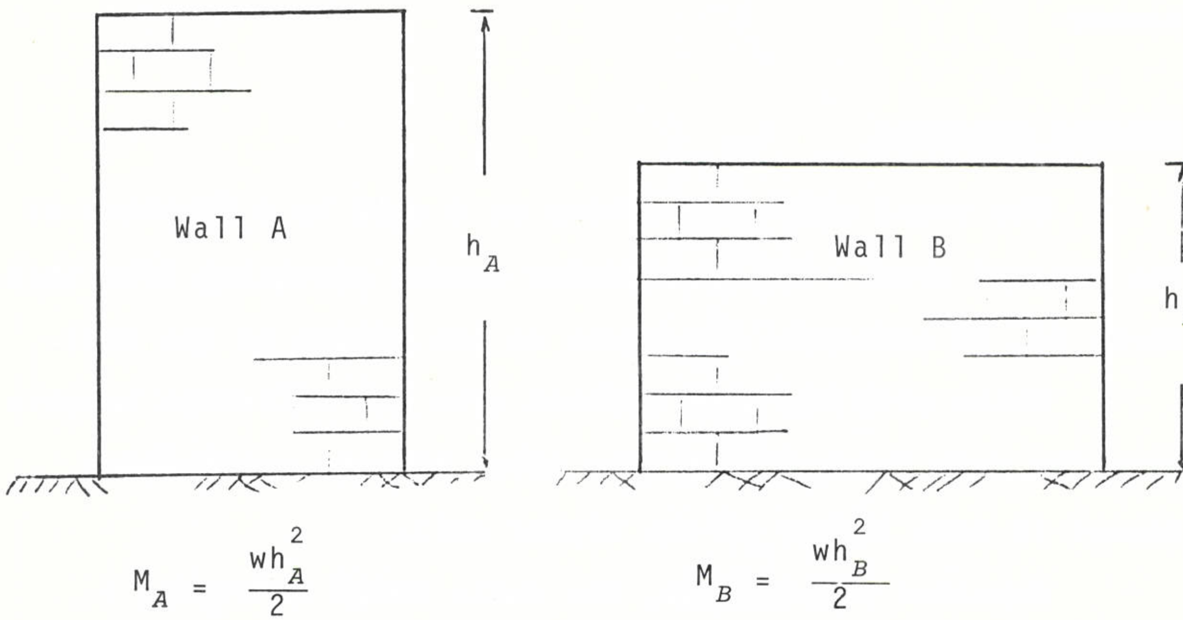

Figure 2 Wind Induced Overturning Moment for the Two Walls A and B with the Same Total Wind Load Relates to the Ratio of the Square of their Height

From this simple example it is obvious that the spacing of temporary bracing must decrease as the wall height is increased.

Maximum Unbraced Height

The maximum height of a free standing masonry wall for which the wind induced moment will not cause overturning is a function of the self-weight of the wall, the thickness of the wall and the wind velocity.

The wind pressure W (in lb per square foot) on a cantilever wall can be determined with a factor of safety of one by the following formula:

$$W = 0.00256 \times 1.3(F.V.)^2 \quad \ldots\ldots\ldots\ldots \quad (1)$$

in metric units the wind pressure W(in kg/cm^2) is given by:

$$W = 0.00482 \times 1.3(F.V.)^2 \quad \ldots\ldots\ldots\ldots \quad (2)$$

The factor 1.3 is the sum of the windward pressure (0.8) and leeward suction (0.5). F is a factor for wind velocity correction with height. It is 0.94 for walls up to 20 ft. high. For higher walls a factor F of 1.00 should be assumed for the zone 20 to 30 ft. above the ground and 1.06 for the 30 to 40 ft. zone. V is the peak (gust) wind velocity in miles per hour (equation 1) or km per hour (equation 2).

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*Note: The factor of 1.3 is recommended by A.C.I. for factors applicable in a particular area, check the N.B.C.

Consider a free standing wall made of 8 inch concrete blocks* weighing 54 psf. The wall is situated in an area where the maximum probable wind speed is 50 mph. The maximum height the wall can be built without providing temporary wind bracing can be calculated as follows:

$$w = 0.00256 \times 1.3 \times (0.94 \times 50)^2$$
$$= 7.35 \text{ psf}$$

The overturning moment due to this load at the base of the wall is:

$$M = \frac{wh^2}{2} = 7.35 \times \frac{h^2}{2} \text{ ft-lb}$$

This moment is resisted by the self-weight of the wall. When overturning is imminent the moment caused by the wind pressure is equal to the resisting moment. With reference to Figure 3, the restraining moment (bond is neglected) is:

$$\frac{Wt}{2} = (54 \times h) \times (\frac{t}{2}) = \frac{54 \times 7.625}{12 \times 2} \times h = 17.15h \text{ ft-lb}$$

where

W = total self weight per linear foot

h = wall height

t = wall thickness

Equating the two moments and solving for h the maximum unsupported height is obtained.

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*Normal weight

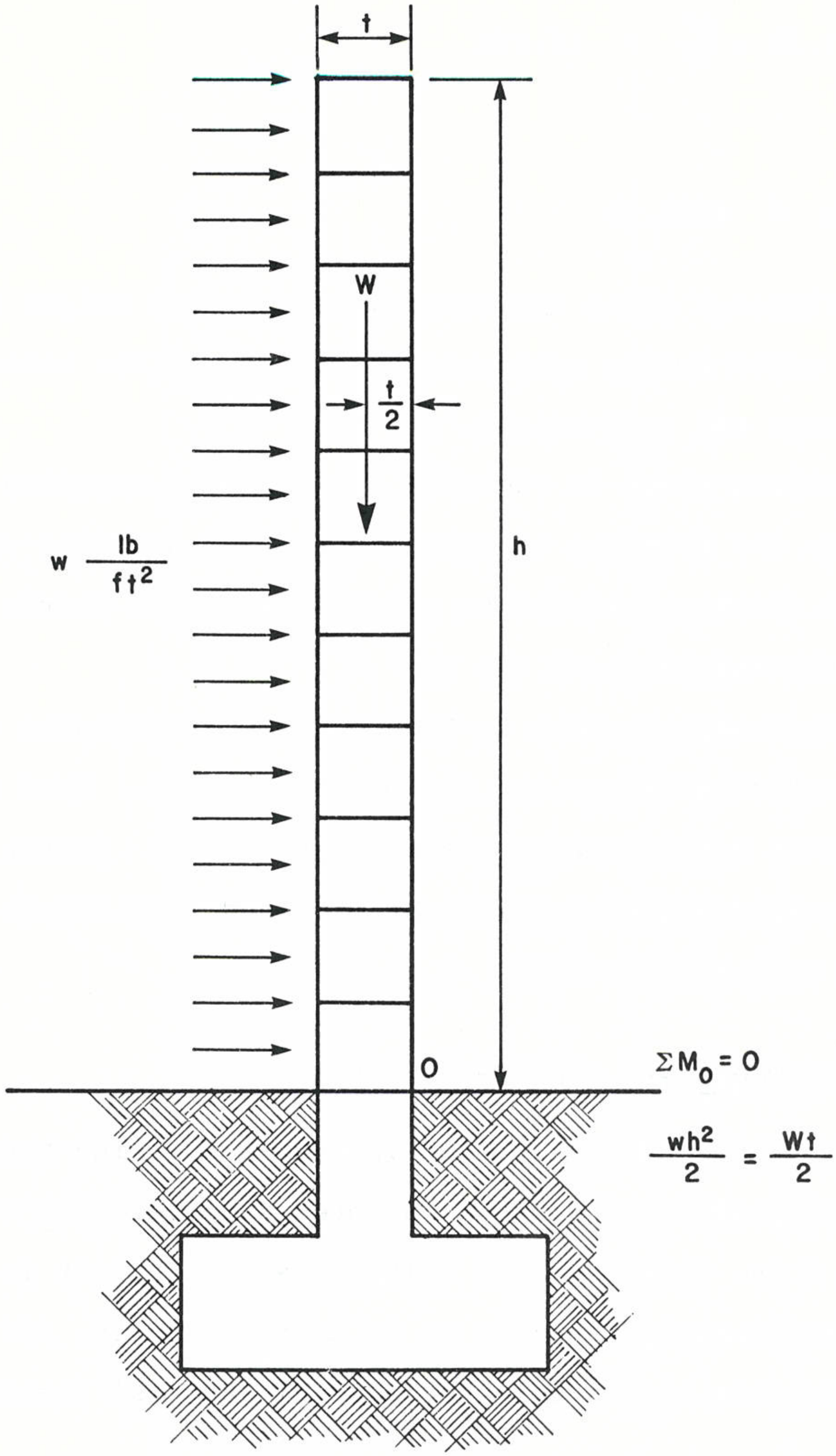


Figure 3. Equilibrium of Wall

$$7.35 \times \frac{h^2}{2} = 17.15h$$

or

$$h = 4.66 \text{ ft}$$

Figure 4 gives the maximum unsupported height for masonry walls during construction calculated for various thicknesses and self-weight.

Values given in Figure 4 are for free standing walls with free air movement on both sides. For a cavity wall, the wall thickness should be assumed to be two thirds the sum of the thickness of the two

The heights shown in Figure 4 are the heights a wall may safely stand above the top of the bracing. Free air movement on both sides is assumed. For walls higher than the allowable unbraced height as obtained from Figure 4, bracing must be provided.

Design of Temporary Wind Bracing System

Consider the wall shown in Figure 5. Wind bracing is to be designed for a maximum expected wind velocity V. The wall is t in. thick, weight w lb/ft^2 and is h feet in height.

The bracing material to be used is 2 x 10, 24f Douglas Fir, allowable stress for the material 1800 psi (assumed value).

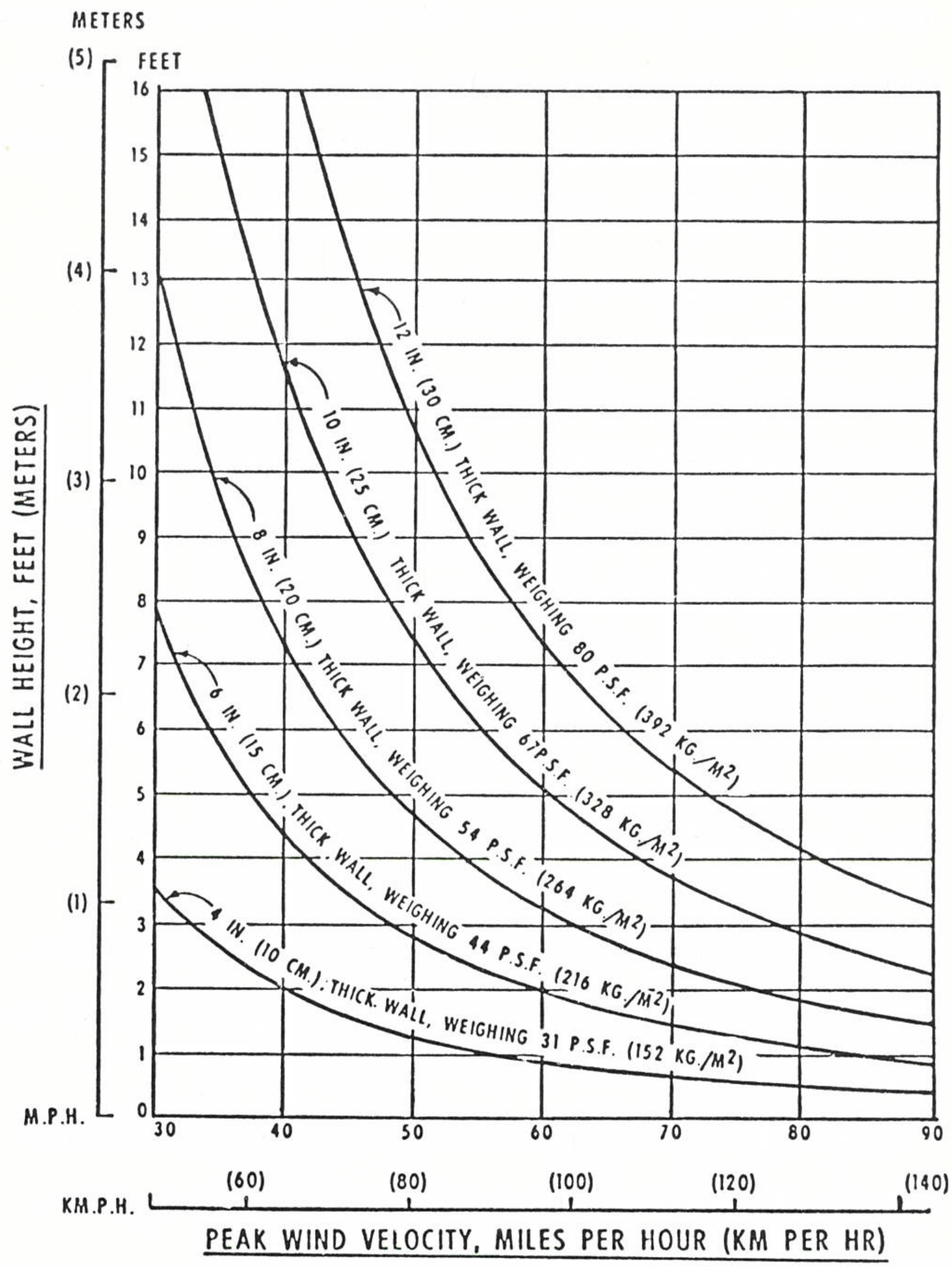


Figure 4 Maximum Unsupported Height of Masonry Walls during Construction for Normal Weight Blocks.

Design Procedure

1. From Figure 4 for the particular wall thickness and weight per square foot find h_a, the unbraced height, by entering this graph with the appropriate wind velocity.
2. Calculate the height where the bracing must reach $(h - h_a)$, (Figure 5). Calculate the required length for the diagonal bracing:

$$\ell = \sqrt{(0.6\ell)^2 + (h - h_a)^2}$$

3. Calculate the required spacing for the bracing by considering the wall as a simple supported beam loaded with $w\eta$, where η is the spacing of the bracing and w the wind load in lb/ft^2.

Referring to Figure 6 the reaction at the top of the support is found from statics.

$$R_T = \frac{w\eta h^2}{2(h - h_a)} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (3)$$

This reaction is the horizontal component of the force in the diagonal member. The force in the member itself is $5/3(R_T)$.

Substituting in equation (3) the reaction which will give rise to a load P in the member and solving for η, the spacing is obtained:

$$\frac{3}{5} P = \frac{w\eta h^2}{2(h - h_a)} \qquad \text{or}$$

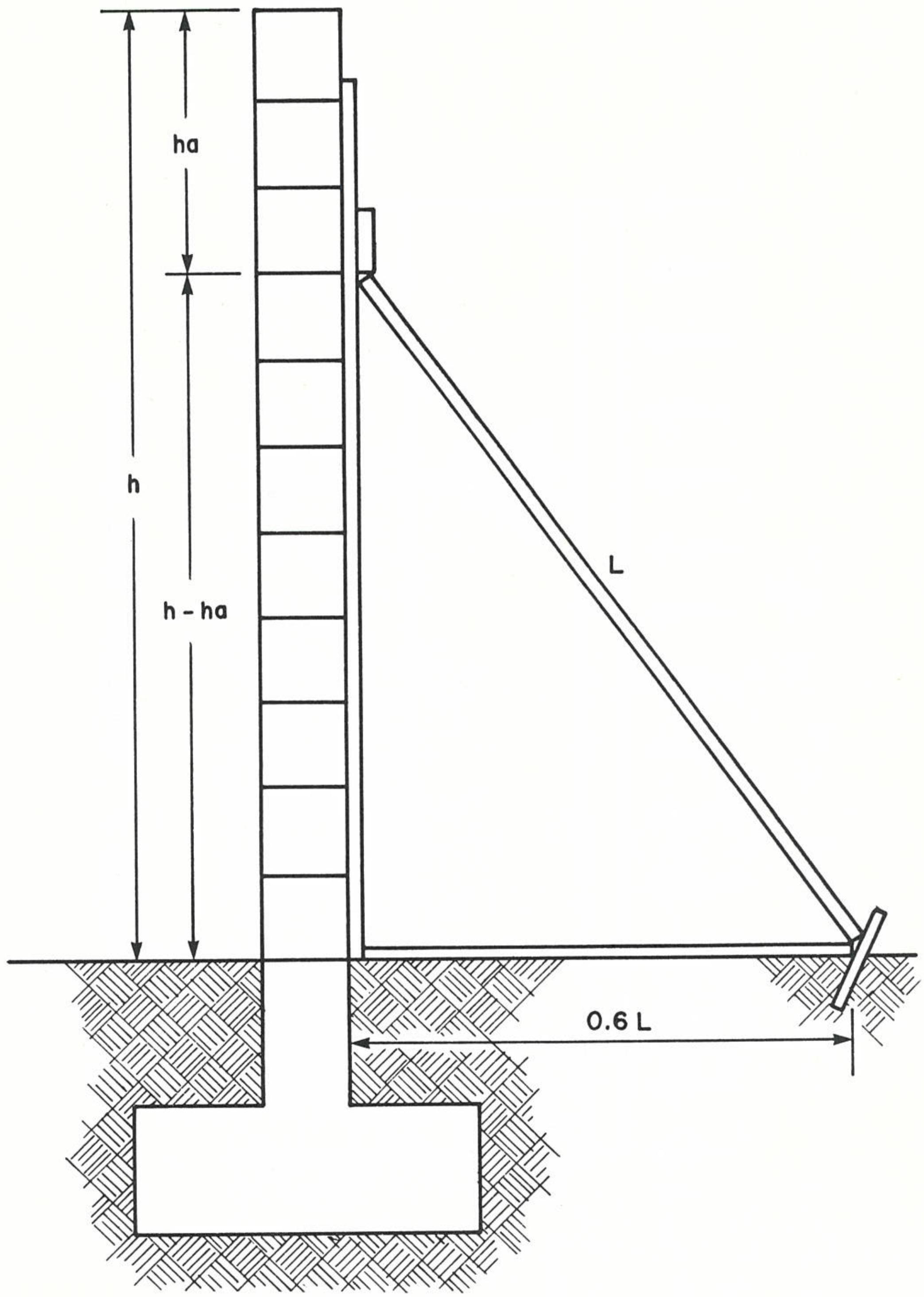


Figure 5 Bracing Example

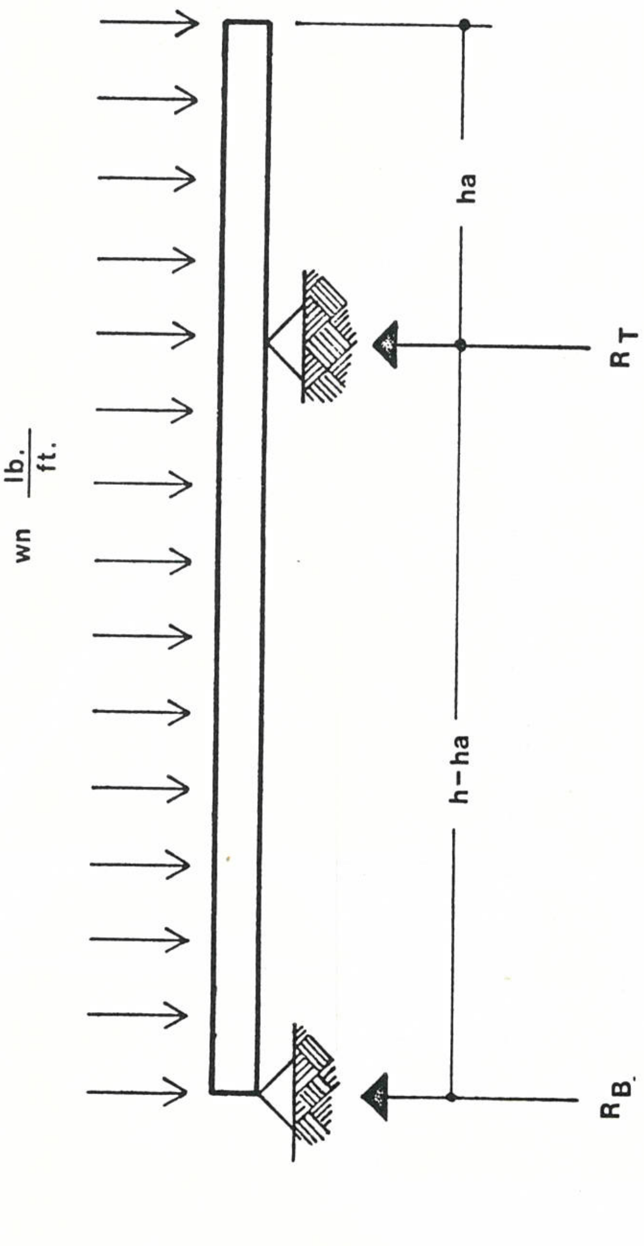


Figure 6   Idealized Wind Loading

$$\eta = \frac{5}{6}\,\frac{P(h - h_a)}{wh^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

The ability of the diagonal member to carry the load is influenced by the length of this member as it relates to the buckling load. The vertical load also must be carried, either by vertical member or by positive anchoring on the wall of the diagonal member. The secondary checks to be carried out are given in the numerial example which follows:

Numerical Example of Wind Bracing Design

A 10 in. thick, 22 ft. high concrete masonry wall is to be braced for maximum wind speed of 60 mph. The wall weighs 67 psf, and 2 x 10 in. Douglas Fir is to be used for bracing material.

Solution:

From Figure 4 for a 10 in. wall the maximum unbraced height for wind velocity of 60 mph is 5 ft.

The wind pressure from equation (1) is:

$$w = 0.00256 \times 1.3 \times (0.94 \times 60)^2 = 10.5 \text{ lb/ft}^2$$

The height of the brace insert above the floor $(h - h_a)$ is 22 - 5 = 17 ft.

The length of the diagonal brace required is:

$$\ell = \sqrt{0.6\ell^2 + (h - h_a)^2} = 21.25 \text{ ft.}$$

The buckling load of this member in the weak axis is:

$$P_{cr} = \frac{EI\ell}{(k\ell)^2} \; \frac{\pi^2 \times 6.667 \times 1.8 \times 10^6}{(0.7 \times 21.25 \times 12)^2} \; 3716 \text{ lb.}$$

Introducing a safety factor of 1.10, the critical load is reduced to 3380.

Brace spacing (from Equation 4):

$$\eta = \frac{5 \times 3380\ (22 - 5)}{6 \times 10.5\ [22^2]} = \frac{246500}{30492} \simeq 9.5 \text{ ft.}$$

Note that in the above calculations the critical or buckling load governed the maximum load on the diagonal bracing. It was also assumed that one end of the bracing was fixed (k = 0.7). If lateral supports are provided to the bracing the buckling load will be increased, however the labour and material cost will also increase. The position and orientation of the bracing material influences the performance of the bracing. In this particular example a vertical force of 2346 lb. must also be carried by the bracing system. A vertical member, as shown in Figure 7, will provide for tying the diagonal brace and the vertical load.

The load from the bracing has to be transmitted to the supports of the system in the floor. The importance of good support for the bracing is obvious: all previous calculations are based on the assumption that the loads will be resisted at the floor level by adequate means. In the following section details of bracing are examined.

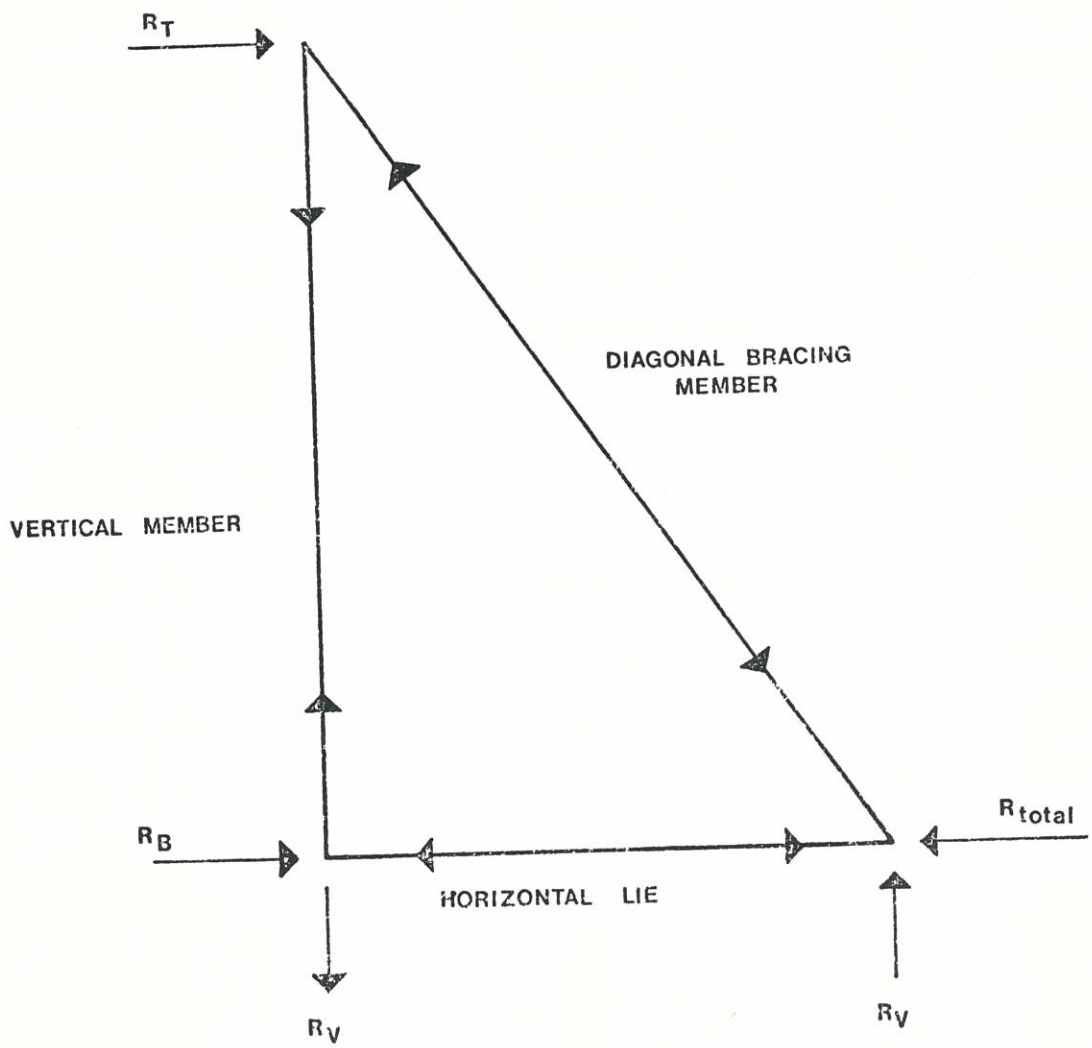


Figure 7 Idealized Force Conditions on Bracing

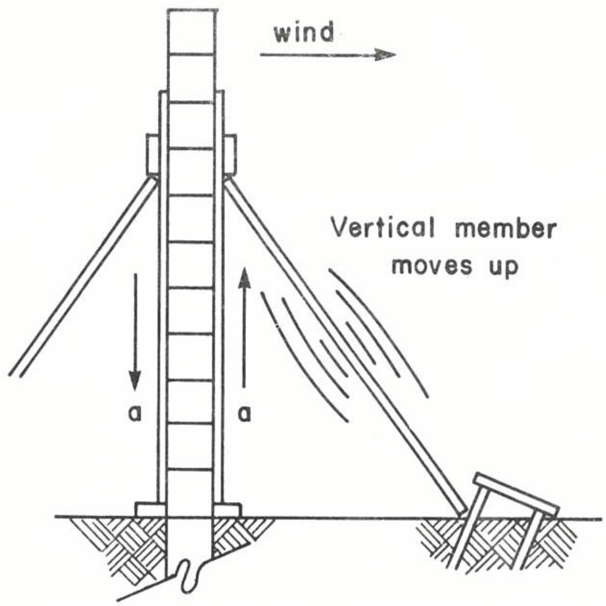
To avoid wind induced failures in the vertical direction resulting from the moment between supports the maximum spacing of temporary bracing should not be more than 15 feet.

Bracing Details

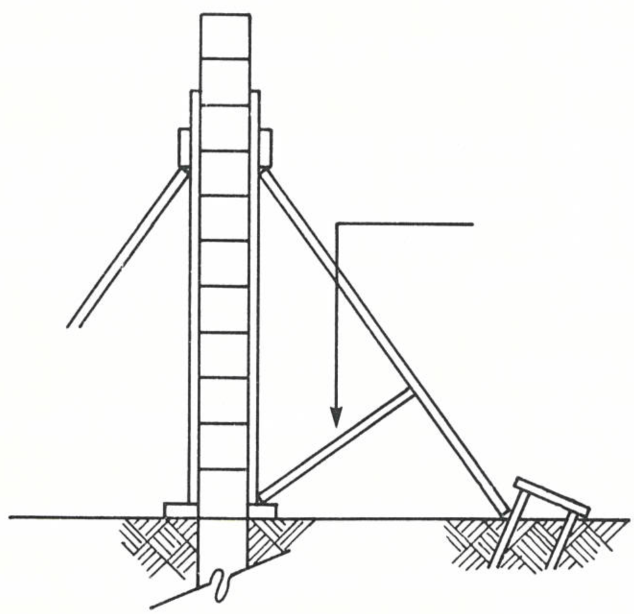
Bracing members are usually slender. As a result orientation of rectangular wood planks is very important. The members should be positioned in such a way as to provide maximum resistance to bending in the loaded plane. Diagonal braces should be provided with cross members in order to reduce their effective length and also to provide an alternative path for the loads.

The most important aspect of bracing however, is the fastening of the brace to the floor.

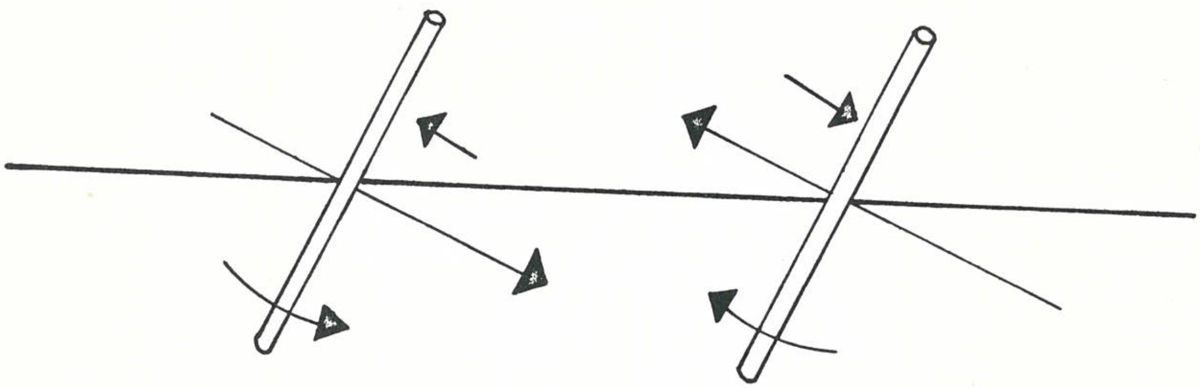
The most common type of support is by means of pegs (wood or steel rods) driven into the ground. Many walls have been lost because of soft soil or loosening due to rain. Some details of bracing procedures are given in the following diagrams adopted from a booklet published by the Workers Compensation Board of Alberta.



An unsupported brace can loosen nails and pegs through vibration. It is weak, and can move up (a) taking the vertical member with it, or move down (a), depending on wind direction.



A brace support strengthens the beam effect several times, stops vibration, and allows no vertical movement, up or down.



Where pegs or pins driven into the ground provide adequate support when used with a compression brace, pegs and pins often fail when used to anchor guy wires in tension.

Where a compression brace thrusts down and tightens, a guy wire lifts up and loosens.

For guy wire bracing, pins in unfrozen ground must be sturdy enough to resist bending and be driven to a reliable depth.

A safer method is to anchor to dead men consisting of concrete filled post holes.

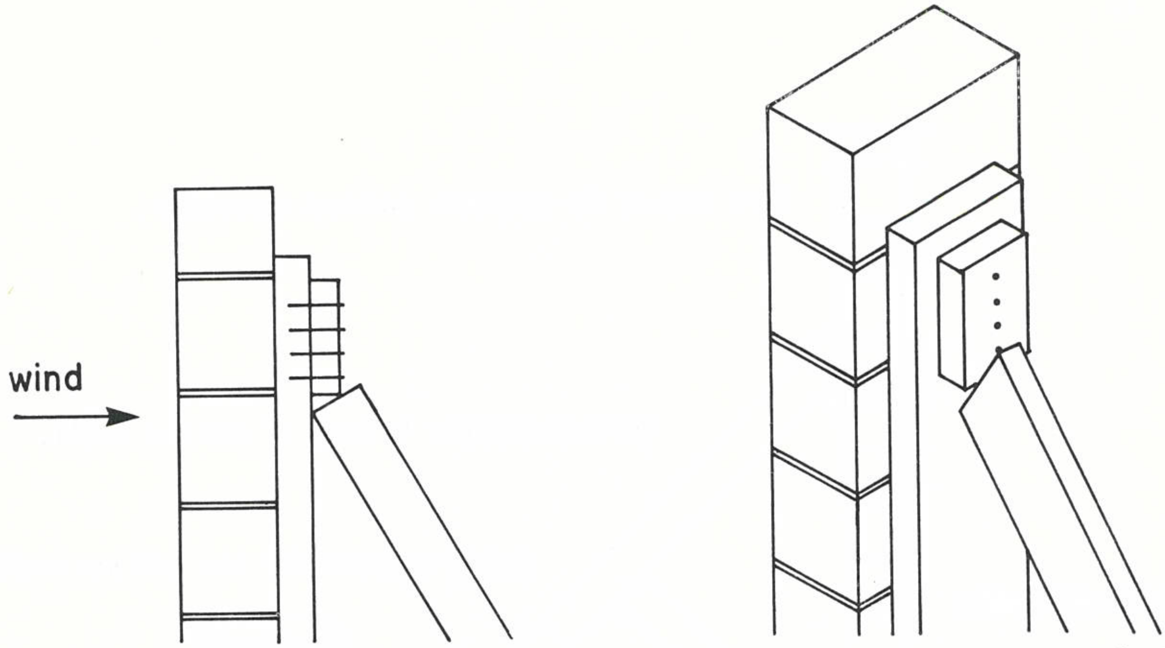
The guy wire should be wire rope. Mild steel wire, even when several strands are twisted together, often fails, as the tension applied by twisting often approaches the ultimate strength of the wire. Nicks and other imperfections in one of the strands can lead to progressive failure of the remaining strands.

Other points to check:

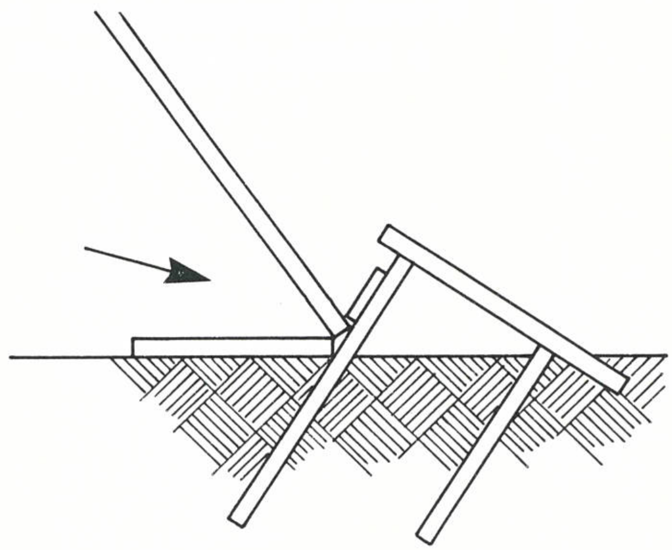
Are the snap ties or other devices through the wall strong enough to resist the load?

Are the cable clamps the right size and properly fastened?

Finally, are the turnbuckles evenly tight to avoid pulling the wall out of plumb?



To restrain the upward movement of the brace under wind pressure, a well nailed cleat is recommended.

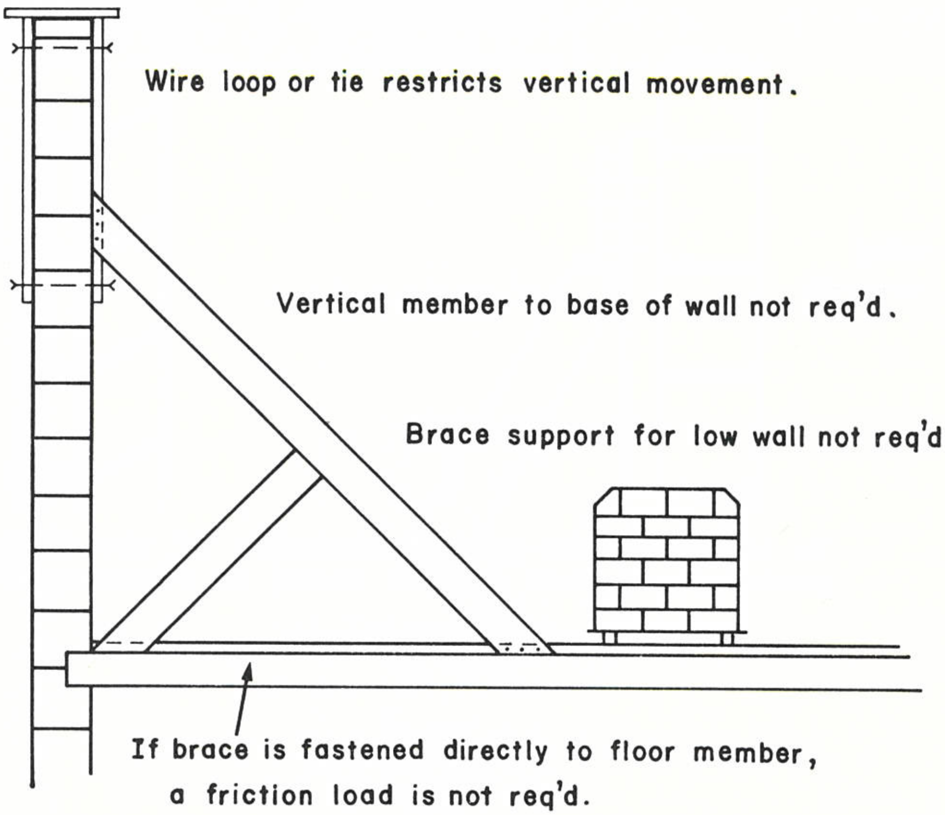


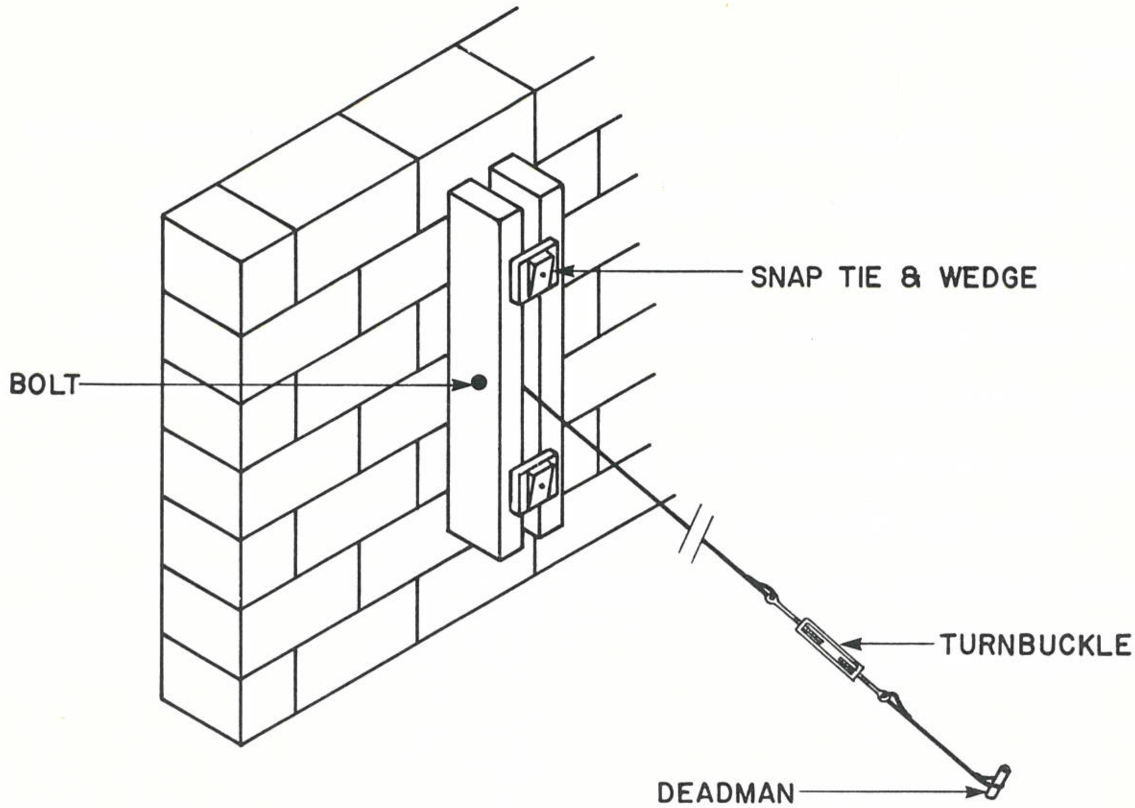
Many walls have toppled because of soil loosening due to moisture. By taking advantage of the leverage provided by 2 connecting pegs, the thrust resistance is increased considerably.

As a further precaution both the brace and vertical member against the wall should be supported on mud sills.

A steel pin 1" or more in diameter is an adequate peg in frozen ground. Its usefulness is, however, severely reduced by thawing ground. Insulate against rising temperatures by driving the pin through a piece of scrap plywood.

DOUBLE ACTING BRACE





Where the height of the wall makes compression braces impractical guy wires in tension are a proven method.

RED OR YELLOW DANGER RIBBONS SHOULD BE TIED TO THE CABLE.

Summary

Wind induced loads can cause failures of masonry walls during construction. An approach for determining these forces and providing resistance is presented. The method can be used with minor alterations for the design of cable bracing where the resistance is provided by tension. Details of bracing construction are shown in the form of diagrams.

From the analysis presented it is clear that wind induced failures can be prevented with relatively small additional cost.

ACKNOWLEDGEMENTS

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