Temporary Wind Bracing of Masonry Structures

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M.Hatzinikolas, Ph.D., P. Eng.
Executive Director
C.M.R.I.

R. Zmavc, M.Sc.
Research Associate
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S. C. Zervos, P. Eng.
Reasearch Engineer
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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 when the lateral loading occurs.

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

The bracing required to secure the walls from blowing over is dependent on five primary functions. These are:

(a) Type of wall as it relates to the self weight.

(b) Thickness of the wall as it relates to the resistance to the overturning moment induced by the wind.

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

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

(e) Geographical location of the building as it relates to the expected wind velocities (and the 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. The results of an experimental program, carried out by the Canadian Masonry Research Institute in Alberta, included tests on anchor evaluation and wind bracing evaluation are reported.

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 resulting in a change in pressure. Although stagnation pressure is produced near the centre of the wall, there is an increasingly steep pressure gradient towards the edges.

Behind the wall the streamlines of flow are unable to come together immediately, resulting in a reduction of pressure and a subsequent creation suction behind the wall. The result of the two forces acting on the wall gives rise to a moment which can be large enough to counteract the self weight of the wall resulting in 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 kilonewtons per square meter (kPa) 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" is the wind induced load in kPa and "h" is the height of the wall in meters. 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. It should be noted that wind induced failures are not restricted to masonry structures. In fact, partially completed steel structures have been known to fail during construction as a result of inadequate protection against blow overs.

Preventing collapse of masonry walls caused by wind is a relatively simple and inexpensive process. Temporary bracing placed at appropriate locations and

constructed 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 and that it is beyond the scope of this paper to examine failures in a probabilistic manner.
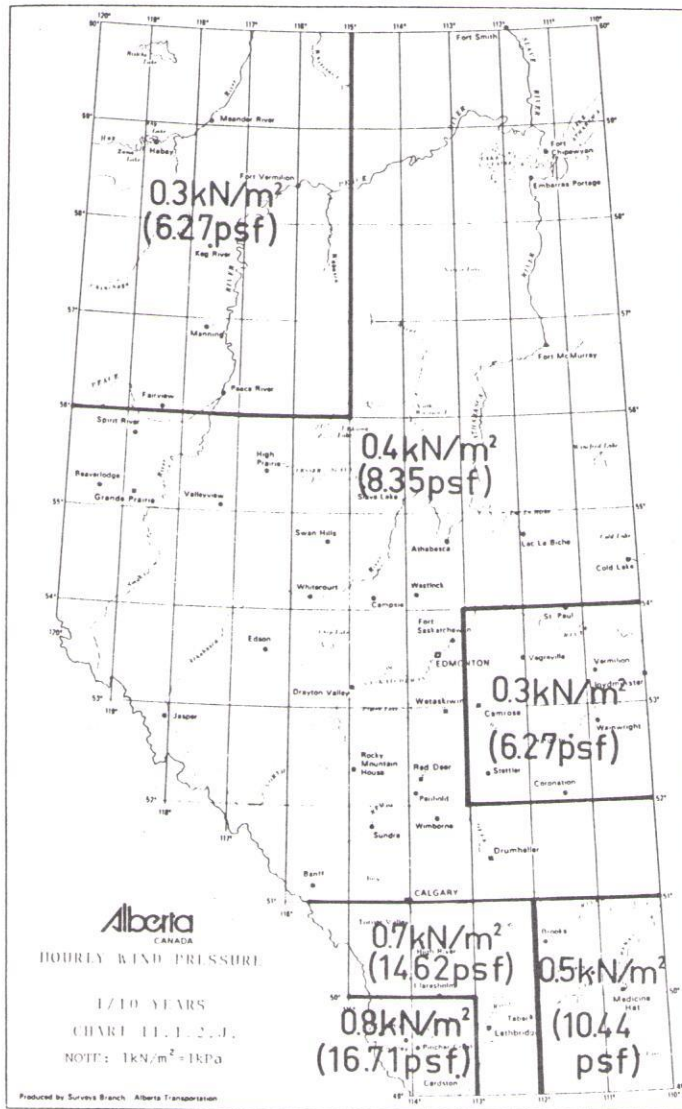

Figure 1. Wind Pressure Chart for the Province of Alberta

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 on 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 are free standing and in the same geographical area. The overturning moment per meter of wall is:

$$M_a = \frac{wh_a^2}{2} \qquad (1)$$

for wall A and :

$$M_b = \frac{wh_b^2}{2} \qquad (2)$$

for wall B.

If for example $h_a = 1.5h_b$ then $M_a = 2.25M_b$.

From this simple example it is obvious that the spacing of temporary bracing must decrease as the wall height is increased.
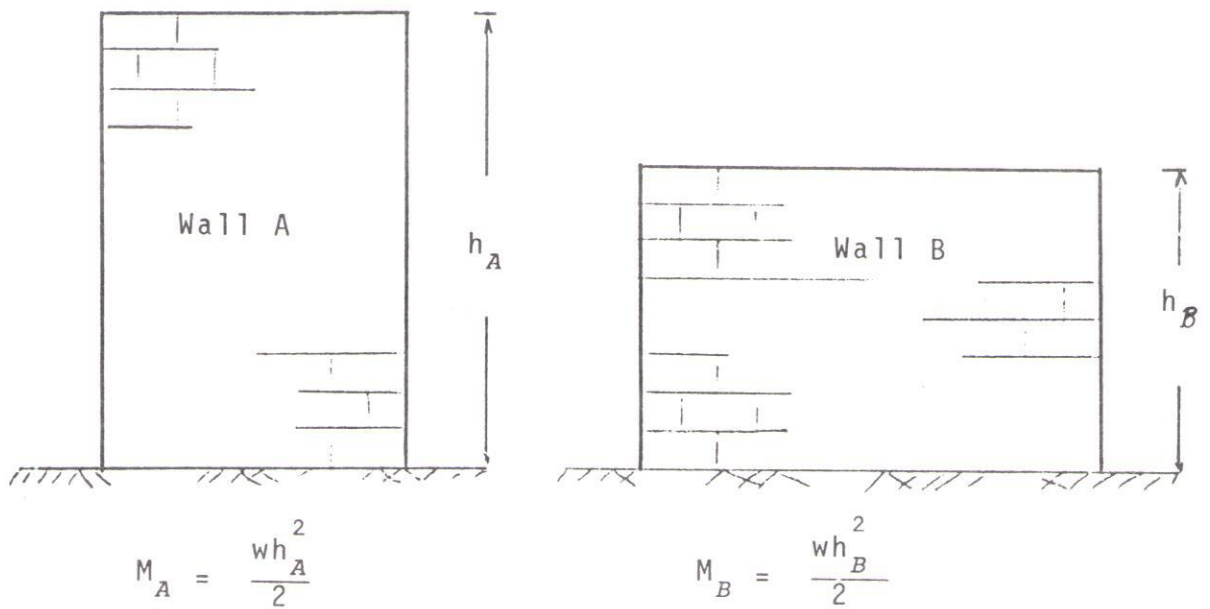
$$M_A = \frac{wh_A^2}{2}$$

$$M_B = \frac{wh_B^2}{2}$$

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.

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 kPa) on a cantilever wall can be determined with a factor of safety of one with the following formula:

$$w = 50x10^{-6} \cdot 1.3 \cdot Ce \cdot V^2 \tag{3}$$

The factor $1.3^{(1)}$ is the sum of the windward pressure (0.8) and the leeward suction (0.5). C_e is a factor for wind exposure and can be calculated by the one fifth power rule[2] as follows:

$$Ce = (\frac{h}{10})^{\frac{1}{5}} \geq 0.9 \tag{4}$$

The reference wind velocity V is entered in km/h.

Example:

Consider a free standing wall made of 200 mm concrete block (normal weight) weighing 250 kg/m². The wall is situated in an area where the maximum probable wind speed is 80 km/h.

Determine the maximum height the wall can be built without providing temporary bracing.

Using equation (3):

$$w = 50x10^{-6} \cdot 1.3 \cdot Ce \cdot 80^2 \tag{5}$$

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[1] Recommended by ACI, check NBC.
[2] Commentary B of The Supplement to the NBC, 1990.

Assuming the height is less than 6 m, then $C_e = 0.9$ and:

$$w = 0.3744 \ kPa \tag{6}$$

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

$$M = \frac{w \cdot h^2}{2} = 0.1872 \cdot h^2 \tag{7}$$

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:

$$\frac{W \cdot t}{2} = \frac{250 \cdot 9.81}{1000} \cdot h \cdot \frac{200}{2000} = 0.24525 \cdot h \tag{8}$$

where:
- W = total self weight (kN/m)
- t = Wall thickness (mm)
- h = Wall height (m)

Equating the two moments and solving for h, the maximum unsupported height can be obtained

$$0.1872 \cdot h^2 = 0.24525 \cdot h \tag{9}$$

$$h = 1.31 \ m$$
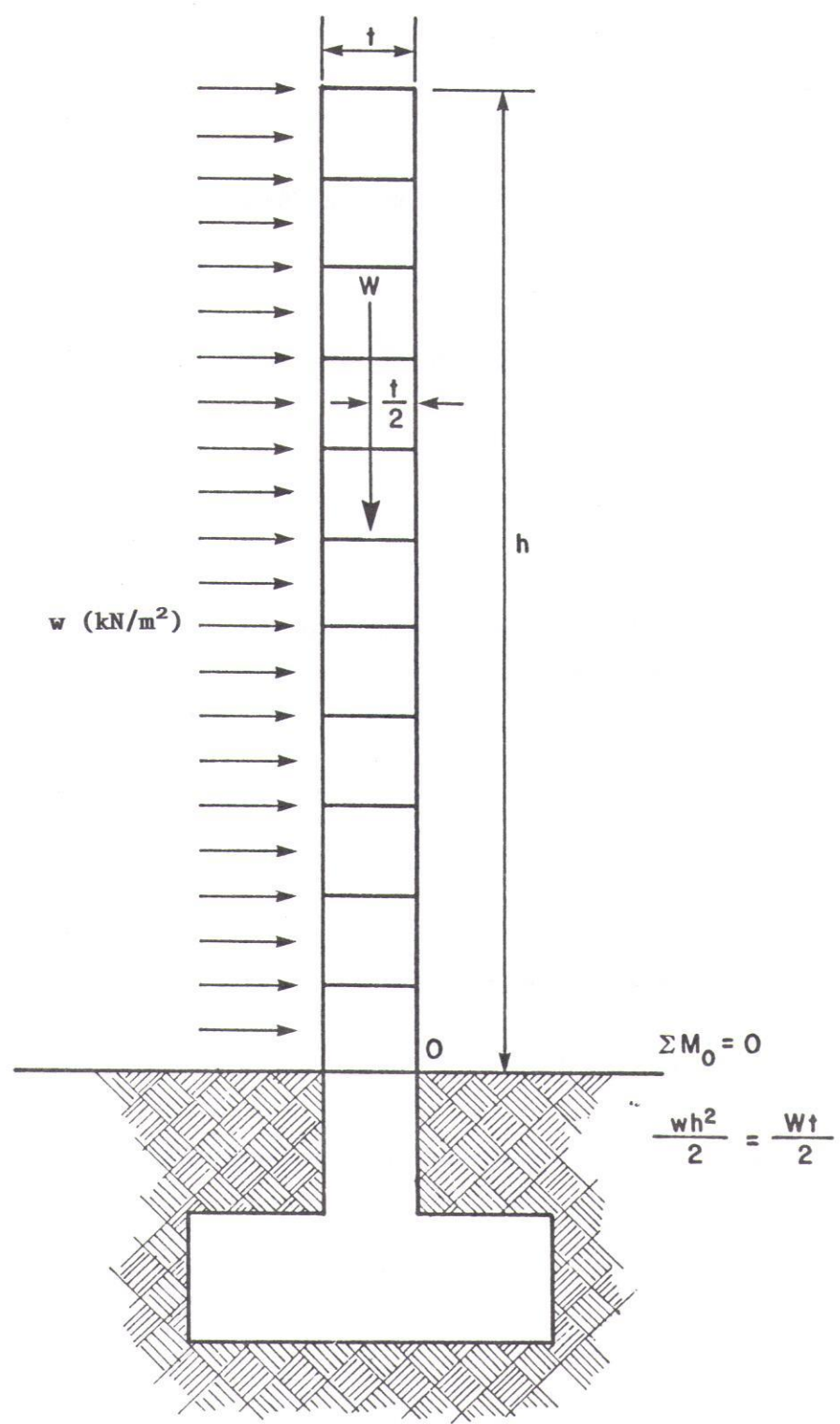
Figure 3. Equilibrium of the Wall.

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

The heights shown in Figure 4 are the heights a wall may safely stand above the top of the bracing. For walls higher than the allowable unbraced height as obtained from Figure 4, bracing must be provided.
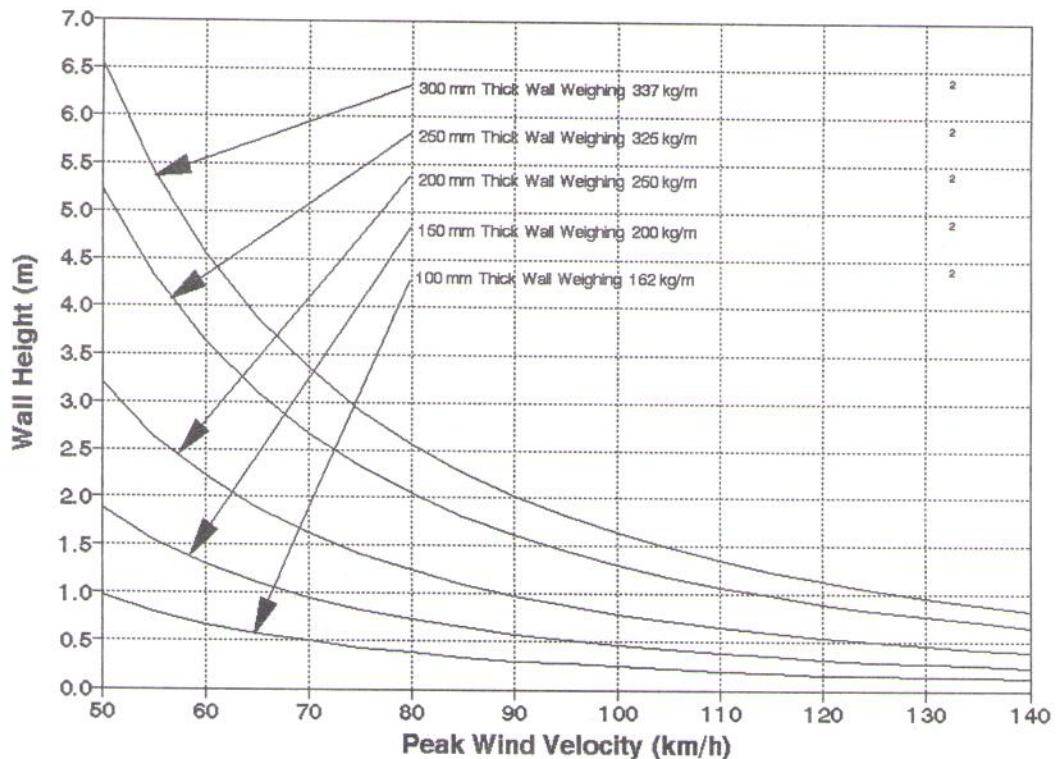
Figure 4. Maximum Unsupported Height of Masonry Walls during Construction for Normal Weight Blocks.

DESING 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" mm thick, weight "w" kg/m^2 and is "h" m in height. The bracing material to be used is 38 x 235 (2x10), #1 O/B Douglas Fir. Allowable stress and the Modulus of Elasticity for the material is 12 Mpa and 12400 MPa respectively (assumed values).
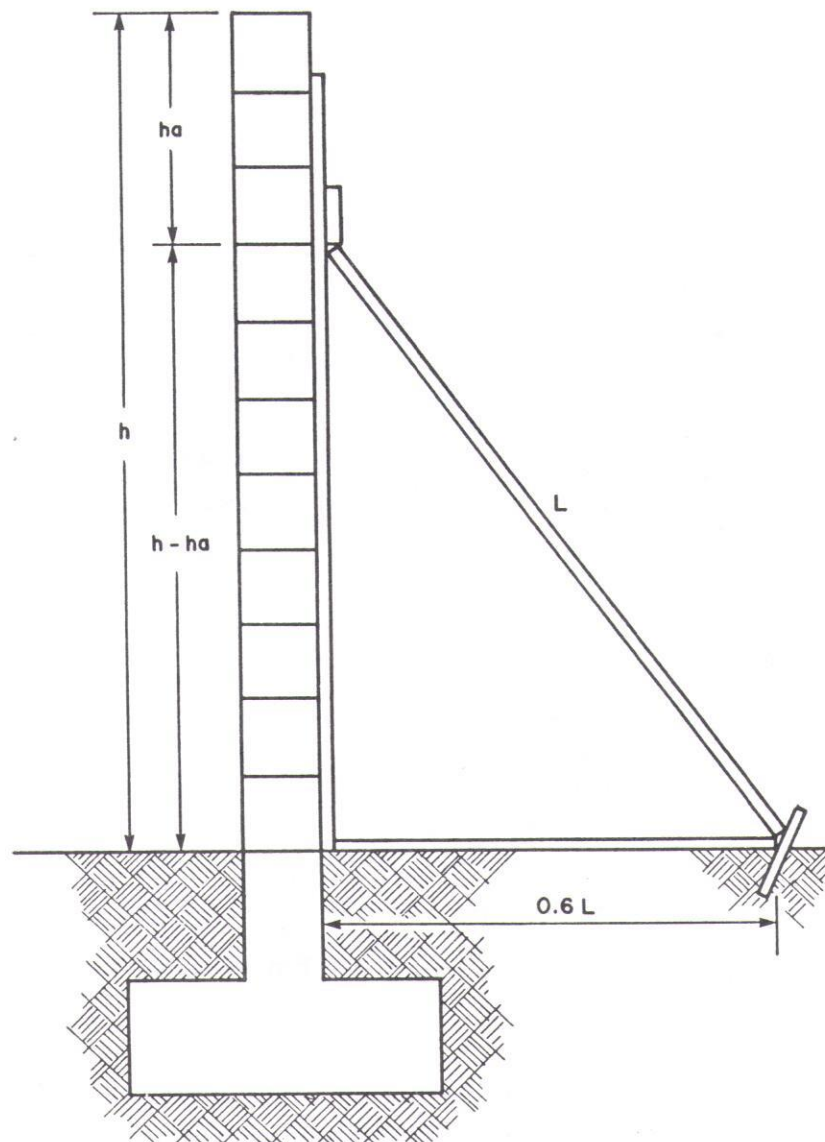

Figure 5. Bracing Example

Design Procedure

1. From Figure 4, for the particular wall thickness and weight per square meter, 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) and calculate the required length for the diagonal bracing:

$$l = \sqrt{(0.6 \cdot l)^2 + (h - h_a)^2} - \frac{h - h_a}{0.8} \tag{10}$$

3. Calculate the required spacing for the bracing by considering the wall as a simply supported beam loaded with wn, where n is the spacing of the bracing and w is the wind load in kPa.

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

$$R_T = \frac{w \cdot n \cdot h^2}{2(h - h_a)} \tag{11}$$

    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 (11) the reaction which will give rise to a load P in the member is:

$$\frac{3}{5}P = \frac{w \cdot n \cdot h^2}{2(h - h_a)} \tag{12}$$

and solving for n, the spacing is obtained:

$$n = \frac{6}{5} \cdot \frac{P(h-h_a)}{w \cdot h^2} \tag{13}$$

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 must also be carried, either by a vertical member or by providing positive anchorage on the wall for the diagonal member. The secondary checks to be carried out are given in the numerical example which follows.
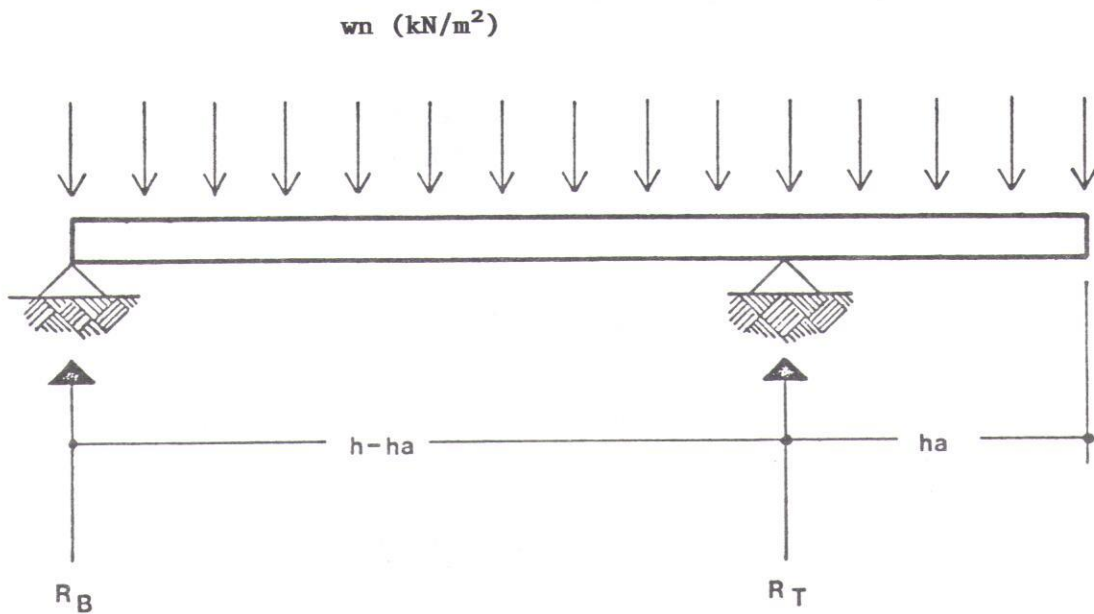

wn (kN/m^2)

$h-ha$ ha

R_B R_T

Figure 6. Idealized Wind Loading

Numerical Example of Wind Bracing Design.

A 250 mm thick, 6.5 m high concrete masonry wall is to be braced for maximum wind speed of 100 km/h. The wall weighs 325 kg/m², and 38x235 (2x10) Douglas Fir is to be used for bracing material.

Solution:

From Figure 4 for a 250 mm wall the maximum unbraced height is 1.5 m. The wind pressure from equation (3) is:

$$w = 50x10^{-6} \cdot 1.3 \cdot (\frac{6.5}{10})^{\frac{1}{5}} \cdot 100^2 = 0.596 \ kPa$$

The height of the brace insert above the floor ($h-h_a$) is 6.5 - 1.5 = 5 m. The length of the diagonal brace required is (equation (10):

$$l = \frac{h-h_a}{0.8} = \frac{5}{0.8} = 6.25m$$

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

$$P_{cr} = \frac{\pi^2 EI}{(kl)^2} = \frac{\pi^2 \cdot 12400 \cdot 1.0745x10^6}{(0.7 \cdot 6250)^2} = 6.87 \ \ kN$$

Introducing a factor of safety of 1.1, the critical load is reduced to 6.25 kN. The spacing of the braces is calculated from equation (13):

$$n = \frac{6}{5} \frac{6.25(6.5-1.5)}{0.596 \cdot 6.5^2} = 1.49 \ m$$

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 5.5 kN 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 (ground). 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 ground level by adequate means. The details of the bracing are examined in the following section.

To avoid wind induced failures in the vertical direction resulting from the moment between supports, the maximum spacing of the temporary bracing should not be more than 4.5 m.
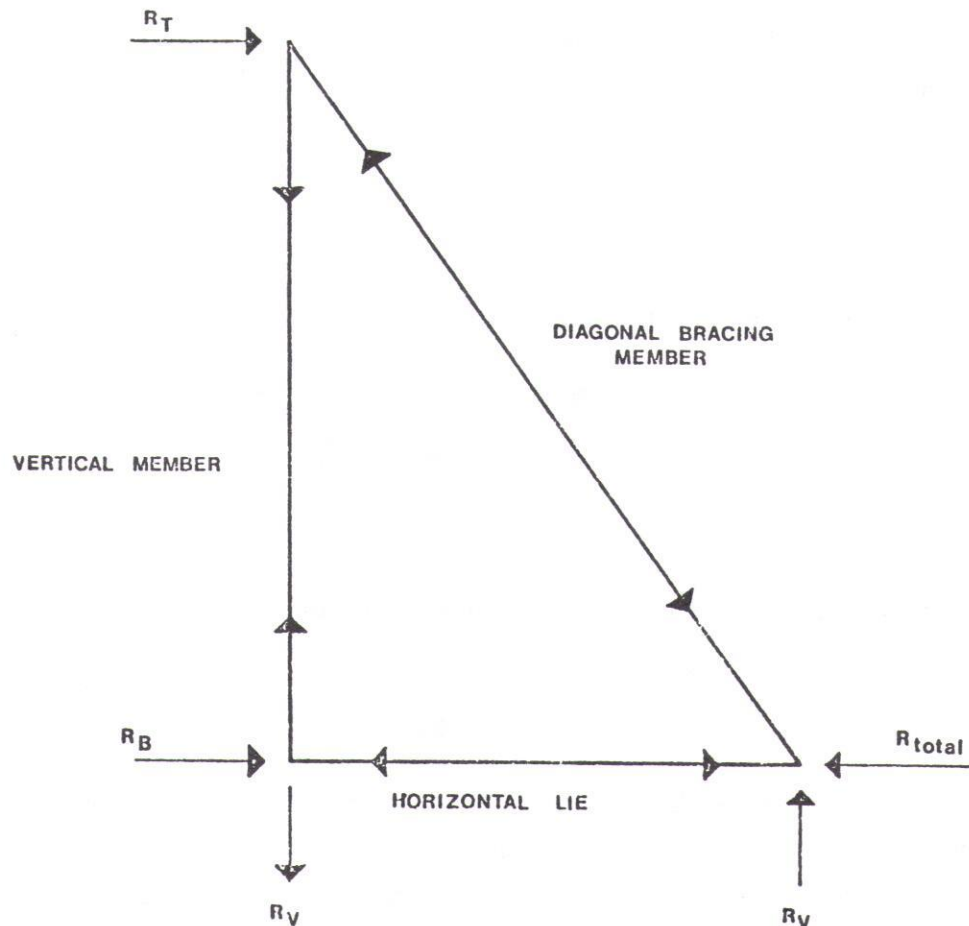
Figure 7. Idealized Force Conditions on Bracing.

BRACING DETAILS

Bracing members are usually slender and thus the orientation of the 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.
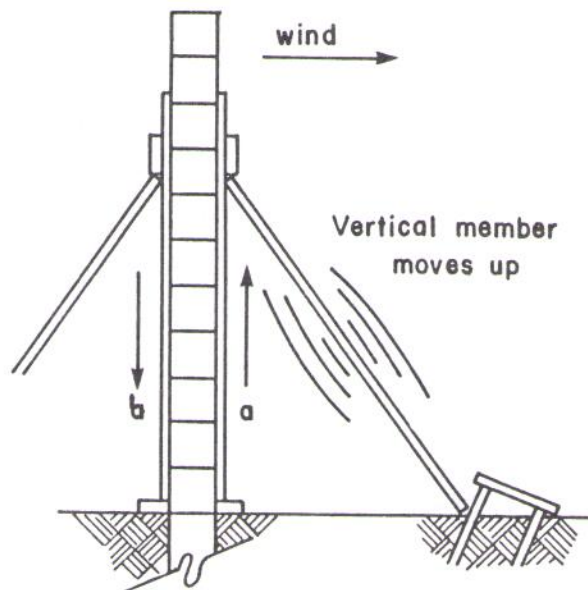
An unsupported brace can loosen nails and pegs through vibration. It is weak, and can move up (a) taking the vertical member with it, or move down (b), depending on the wind direction.
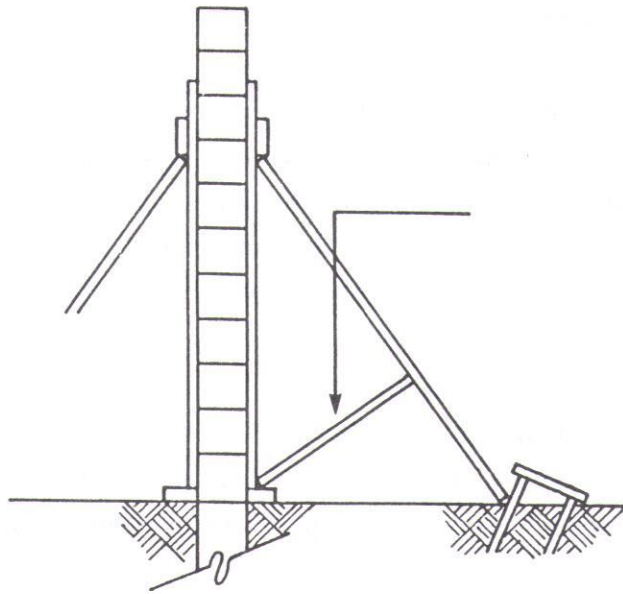
A brace support strengthens the beam effect buckling several times, stops vibration, and allows no vertical movement, up or down.
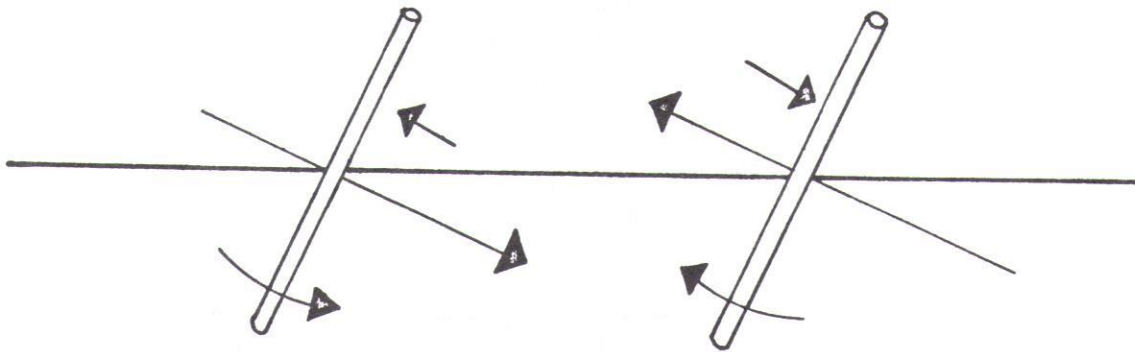
Where pegs or pins driven into the ground provide adequate support when used with a compression brace, pegs and pins often fail when used to anchor guy wires in tension.

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.

17

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 tightened to avoid pulling the wall out of plumb?
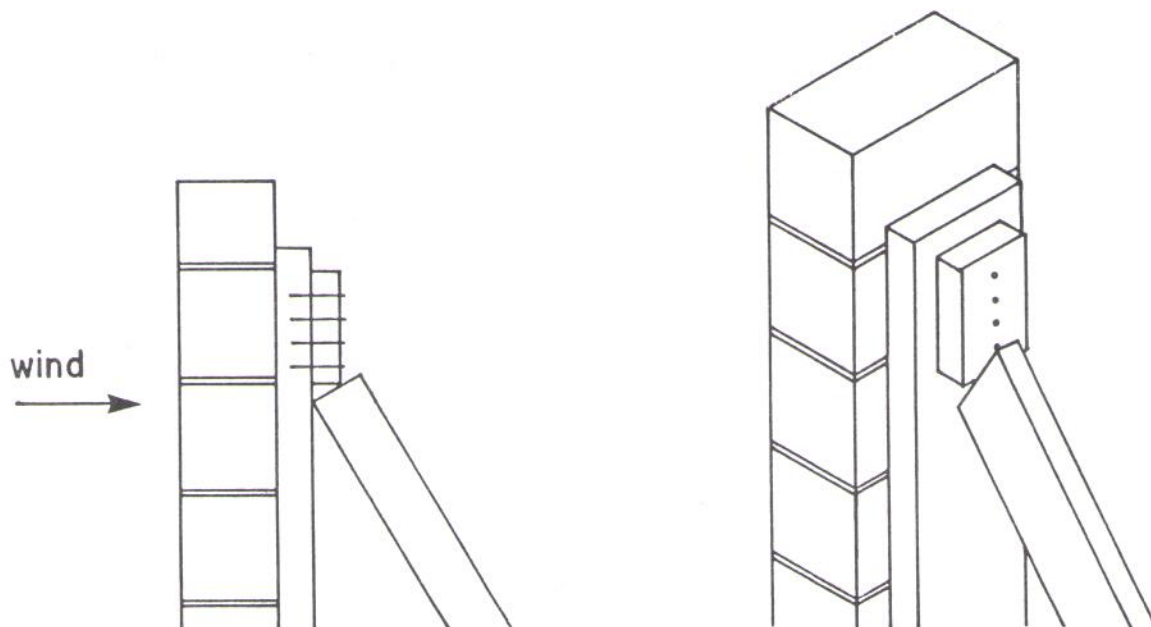
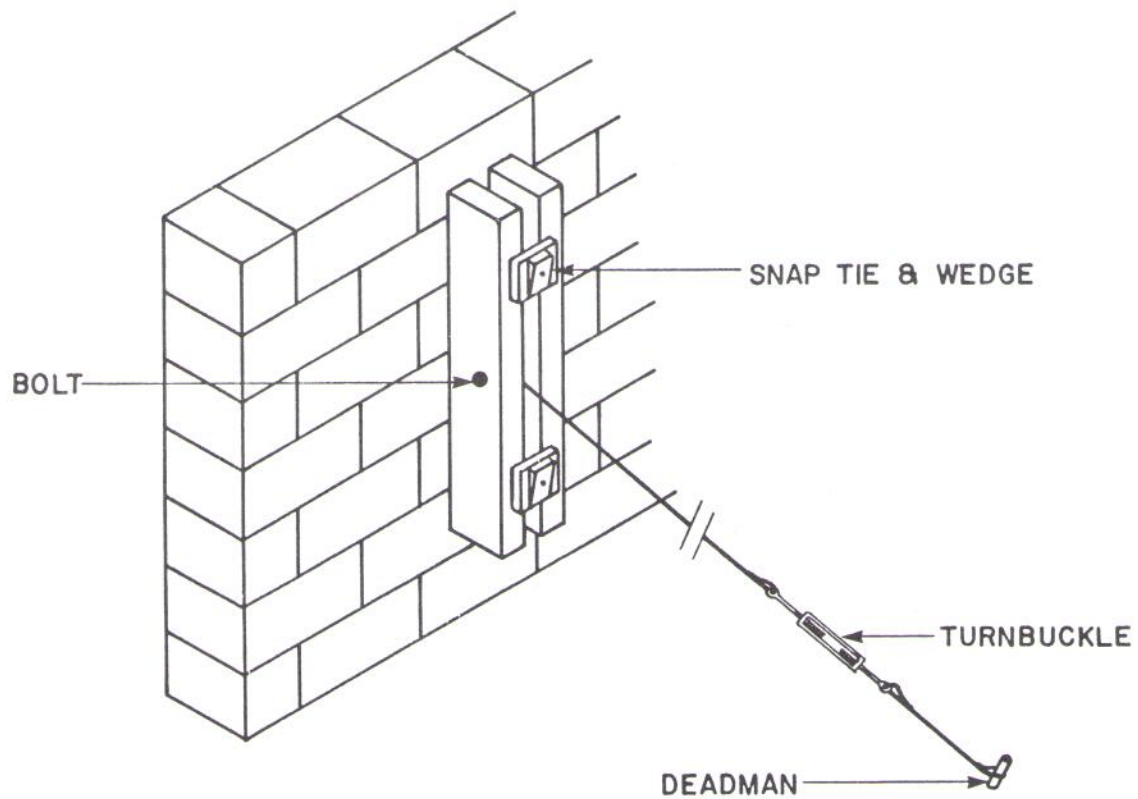To restrain the upward movement of the brace under wind pressure, a well nailed cleat is recommended.
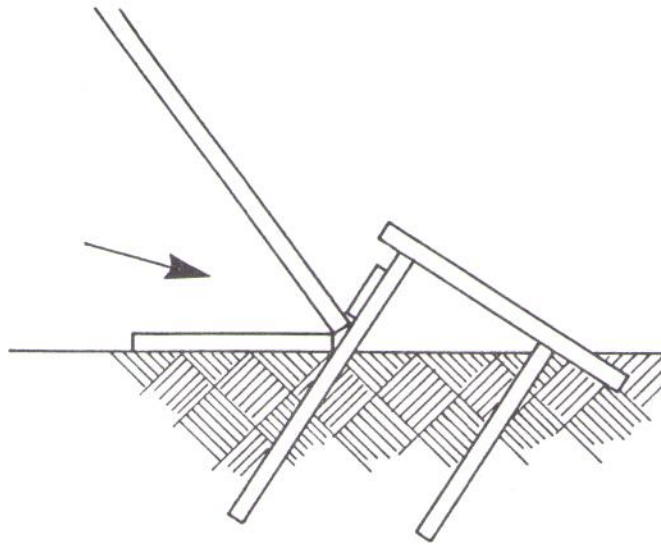
Many walls have toppled because of soil loosening due to moisture. By taking advantage of the leverage provided by two connecting pegs, the thrust resistance is increased considerably.

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

A steel pin 25 mm 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.

Wire loop or tie restricts vertical movement.

Vertical member to base of wall not req'd.

Brace support for low wall not req'd.

DOUBLE ACTING BRACE

If brace is fastened directly to floor member, a friction load is not req'd.

BOLT

SNAP TIE & WEDGE

TURNBUCKLE

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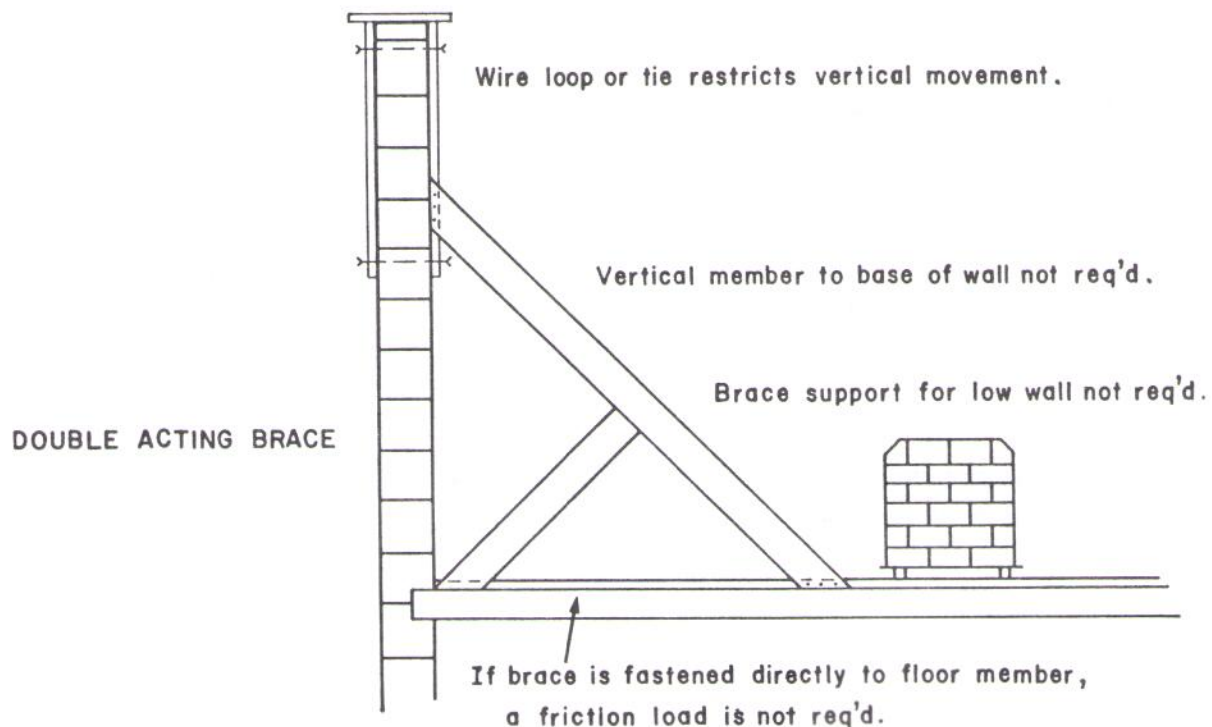
Where the height of the wall makes compression braces impractical, guy wires in tension are a proven method.

RED OR YELLOW RIBBONS SHOULD BE TIED TO THE CABLE.

EXPERIMENTAL WORK.

An experimental program was carried out by R.M. HARDY & ASSOCIATES LTD in Calgary, Alberta, on behalf of the Canadian Masonry Research Institute. This program consisted of two series of test; anchor evaluation tests and wind bracing evaluation. A brief description of these tests is given in this chapter.

A. Anchor evaluation tests.

These tests were designed to determine the force required to pull an anchor from unfrozen soil, the intent being to predict the maximum safe load an anchor could be expected to resist. Knowing this load and the wind velocity data for an area, the minimum safe number of anchors for a system can be determined.

The ground anchors tested (Photos 1 and 2) were placed in the ground at roughly 15 degrees from vertical and the guy wire angles were of 42 and 53 degrees from the horizontal (Figures 8 and 9). The anchor embedment lengths used are included in Table 1 and are considered minimum. (Increased anchor length and embedment associated with proper anchor strength significaly increase pull-out resistance).

The loads were applied by an Enerpac hydraulic jack, and recorded through a load cell and Budd strain indicator.

The soil in which these tests were conducted was a fill consisting of sandy silt with organic layers and debris, with a bearing capacity of 35.9 kN/m^2 (750 Lb/ft^2).

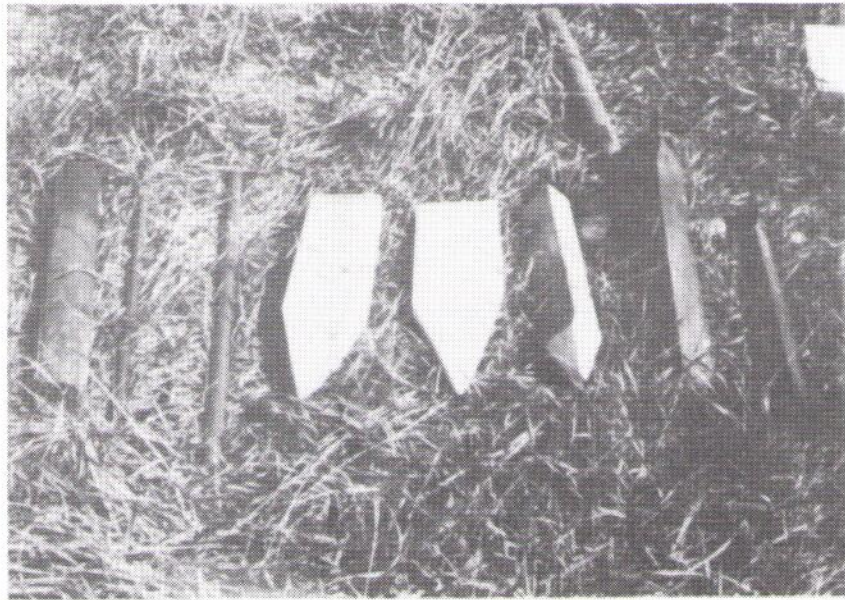Table 1 presents the results of these tests.
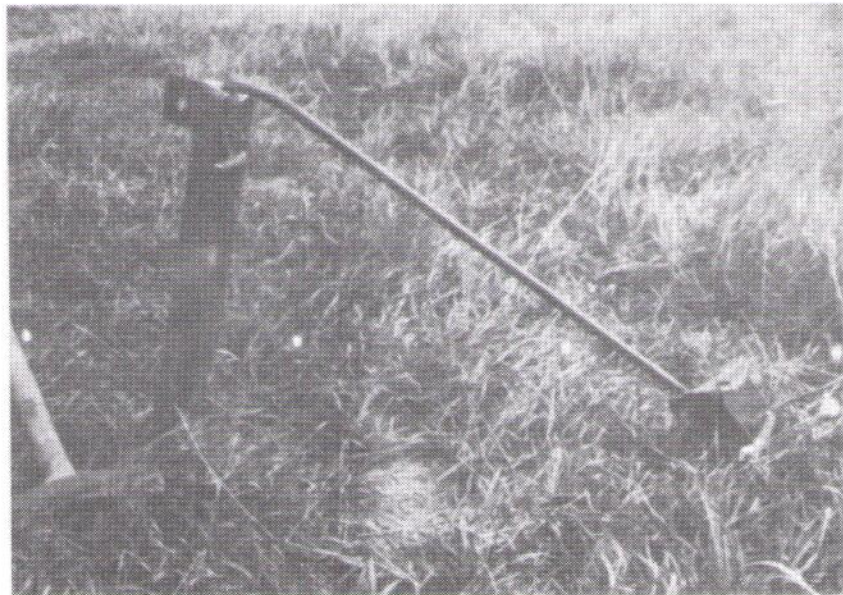
Photo 1. Ground Anchor Types.
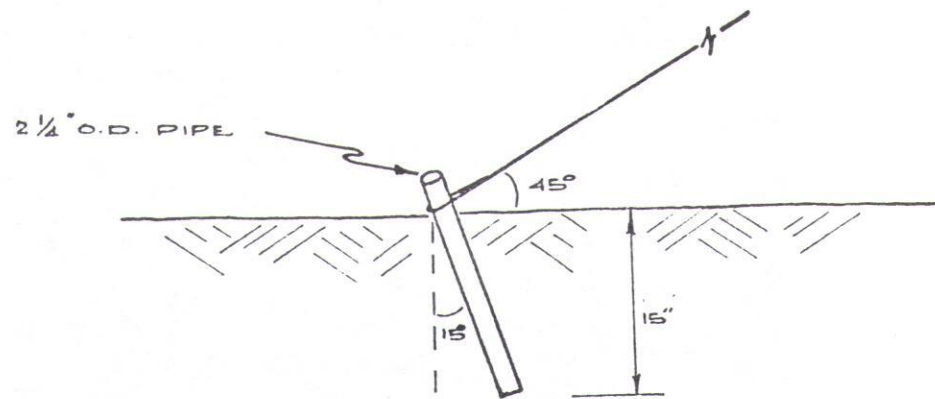

Photo 2. Double Anchor Configuration.

Figure 8. Pipe Anchor.
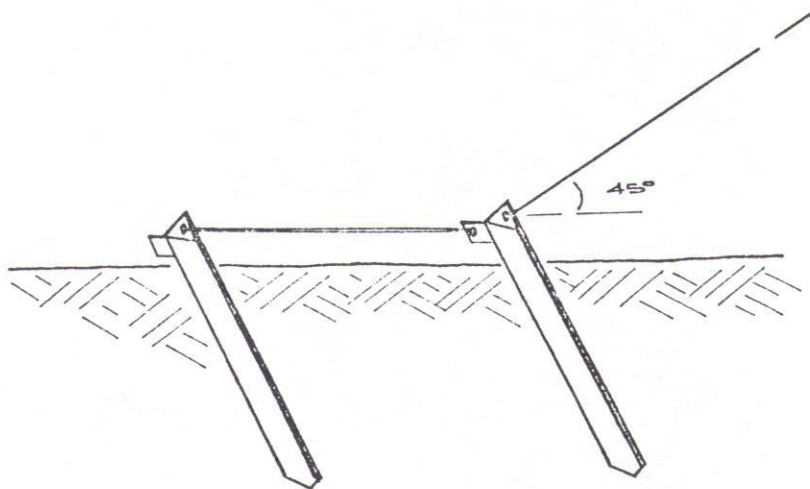
Figure 9. Tandem 1$^{1/2\text{"}}$ Angle-Iron Anchor.

Test No	Speciment	Embedded Length (inch)	(mm)	GUY WIRE ANGLE 42 degrees Load (Lbs)	(kN)	Failure Type (*)	53 degrees Load (Lbs)	(kN)	Failure Type (*)
1	1/2" rod	20"	508 mm	200	0.89	1	--	--	--
2	3/4" rod	20"	508 mm	500	2.22	1	--	--	--
3	1-1/2" angle	15"	381 mm	600	2.67	2	380	1.69	2
4	1-1/2" angle in tandem	15" each	381 mm	950	4.23	2	500	2.22	3
5	2-1/2" angle	15"	381 mm	600	2.67	2	160	0.71	2
6	2-1/2" angle in tandem	15" each	381 mm	500	2.22	3	950	4.23	3
7	1x4 board	12"	305 mm	--	--	--	220	0.98	2
8	2x4 board	12"	305 mm	--	--	--	200	0.89	2
9	2x2 board	13"	330 mm	--	--	--	220	0.98	2
10	2-1/4" O.D. pipe	15"	381 mm	--	--	--	550	2.48	2
11	1/2" U rod	15"	381 mm	--	--	--	450	2.00	2
12	Screw anchor	12"	305 mm	--	--	--	600	2.67	2

(*) Failure Type:
1. Anchor failure
2. Anchor pulled out of soil
3. First of tandem angles-vertical pull out

Table 1. Ground Anchor Pull-out Tests.

B. Wind Bracing Evaluation.

This test was designed to evaluate the ability of typically braced walls to resist wind forces.

Two full scale walls, 5030 mm (16'-6") high and 8530 mm (28 feet) long, were constructed using 200x200x400 mm (8x8x16 inch), 12 Kg (26 pound) light-weight concrete blocks.

Two types of mortar were used for wall construction, the west half of each wall using Type N mortar, and the east half of each wall using Type S mortar (one part normal Portland cement to three parts of aggregate).

Both wall sections were oriented such that they would receive maximum forces from the prevalent N-NW wind peak gusts (Figure 10).

Both sides of the walls were braced (Photo 3).

The south side of the walls was braced with 38x235 (2x10) Douglas Fir planks as shown in Photo 4.

The vertical members of the south side bracing of the west wall were nailed to the walls and the ground anchors were 50 mm (two inches) diameter pipe driven at least 380 mm (15 inches) into the soil (Photo 5). The upper ends of the 2 x 10 braces were nailed to the vertical sections and were butted against 2 x 6 pieces, also nailed to the vertical sections (Photo 6).

The south side braces of the east wall were placed against the wall and were not fastened in any way (Photo 7). The lower end of the braces was restrained only by the soil and the vertical forces of the concrete cylinders (Photo 8). Photo 9 shows the upwind, north side bracing of the two walls.

The deflection of the walls was measured by a linear variable displacement transformer placed in the center and at 1370 mm (4'-6") from the top of each wall. The wind velocity and direction were recorded by field recording wind set designed to average wind gusts over a 15 second period.
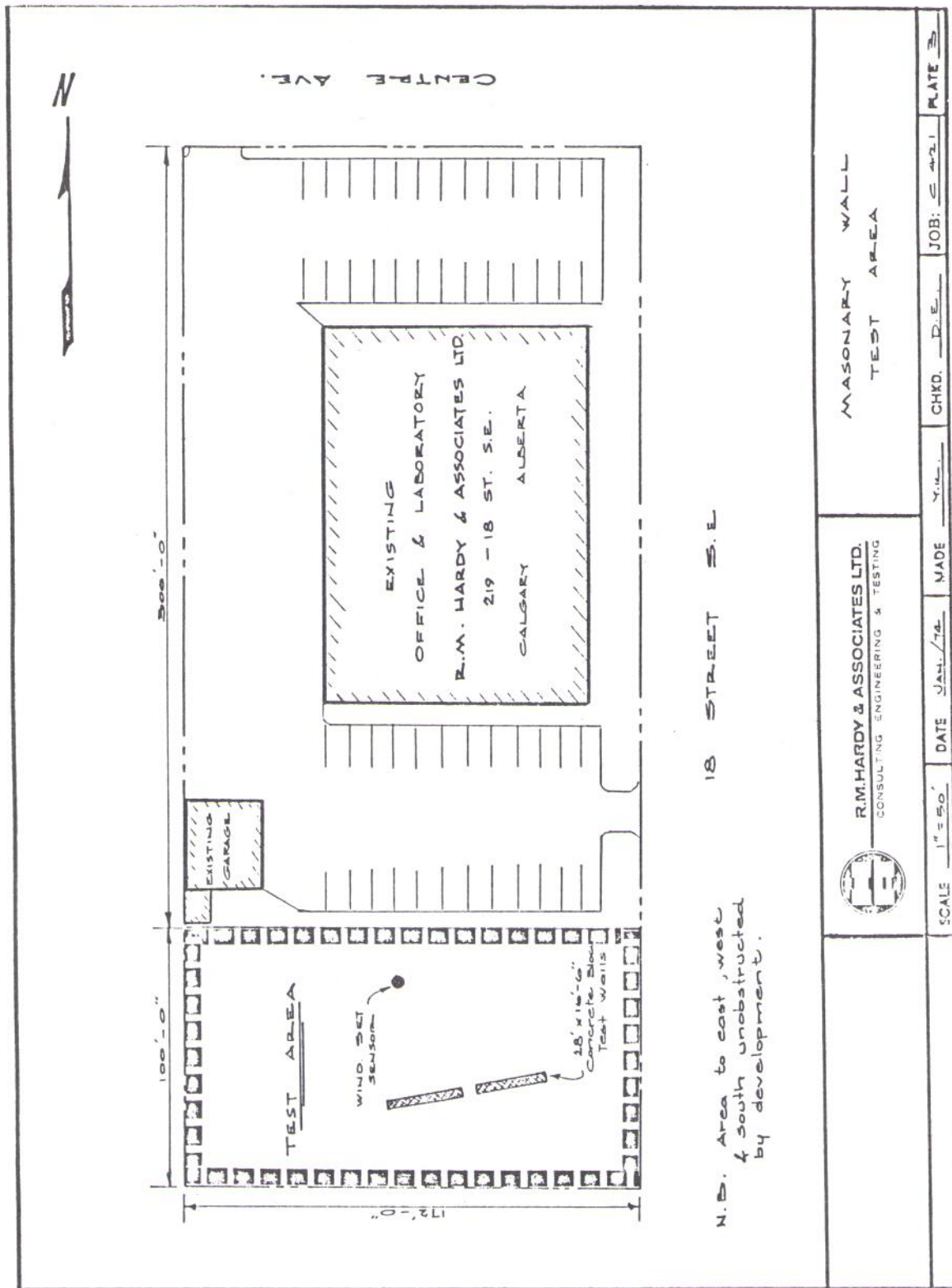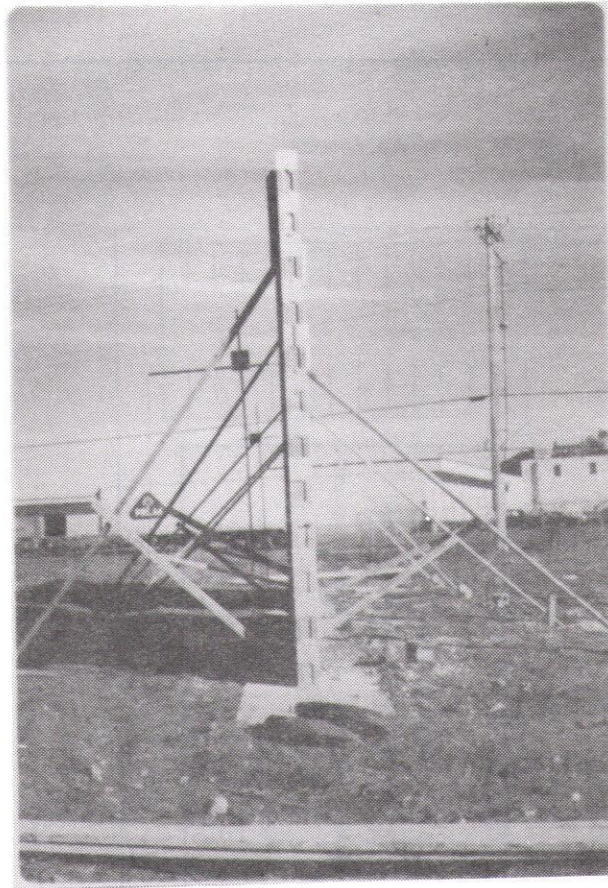
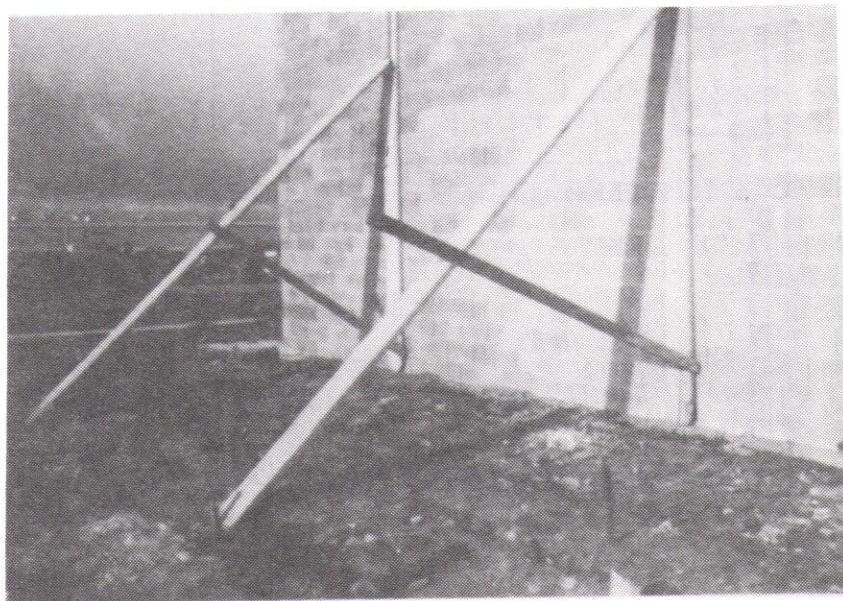**Figure 10.** Layout of Testing Area.

Photo 3. Wall Bracing Profile.

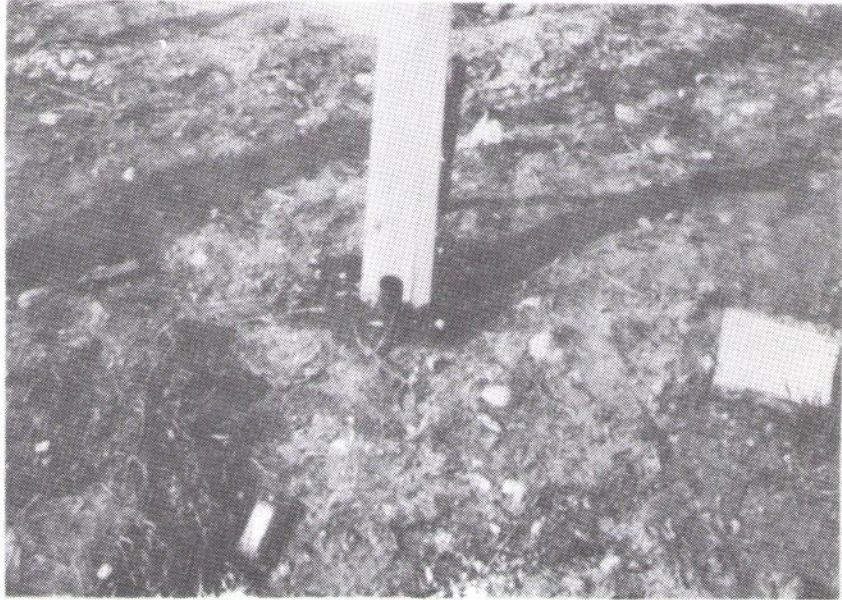**Photo 4**. West Wall South Side Bracing.

Photo 5. West Wall South Side Ground Anchor.
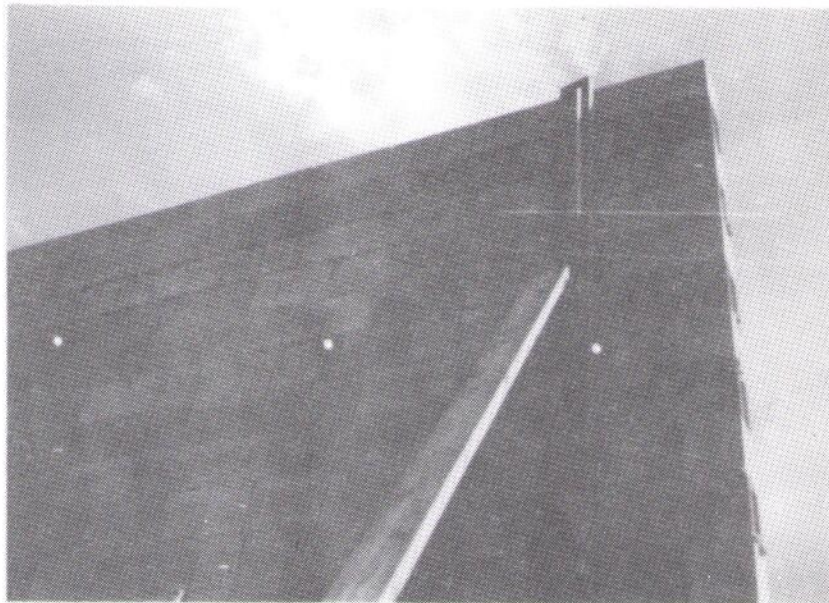

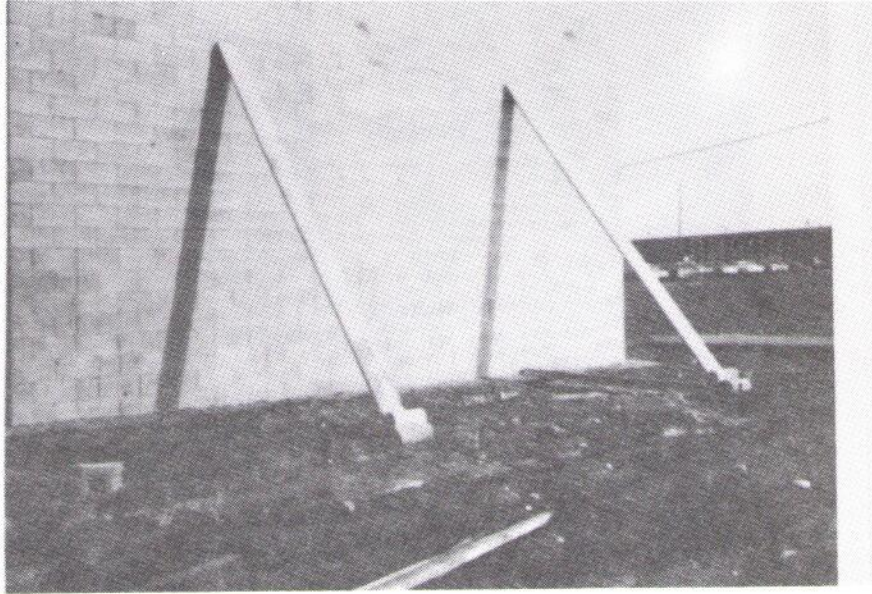**Photo 6.** West Wall Upper Load Transfer Point.
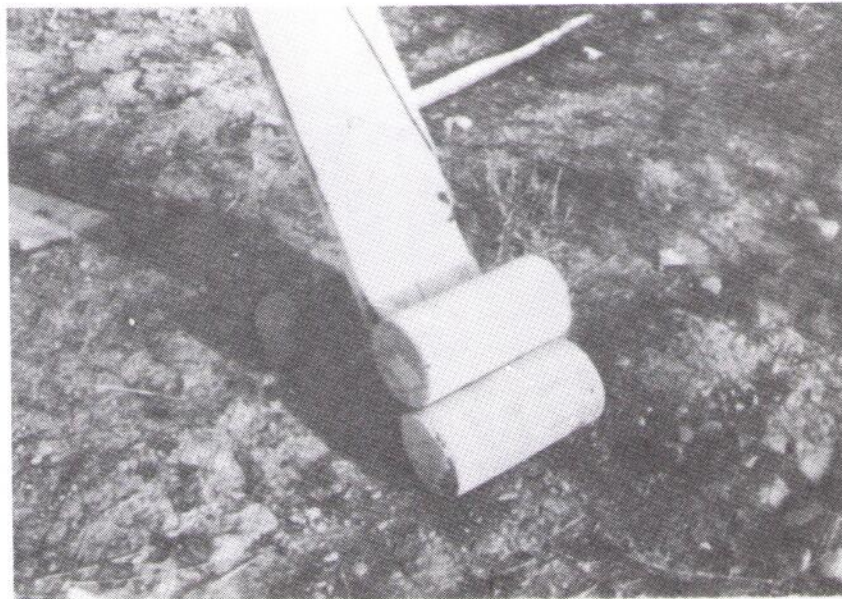
Photo 7. East Wall South Side Bracing.

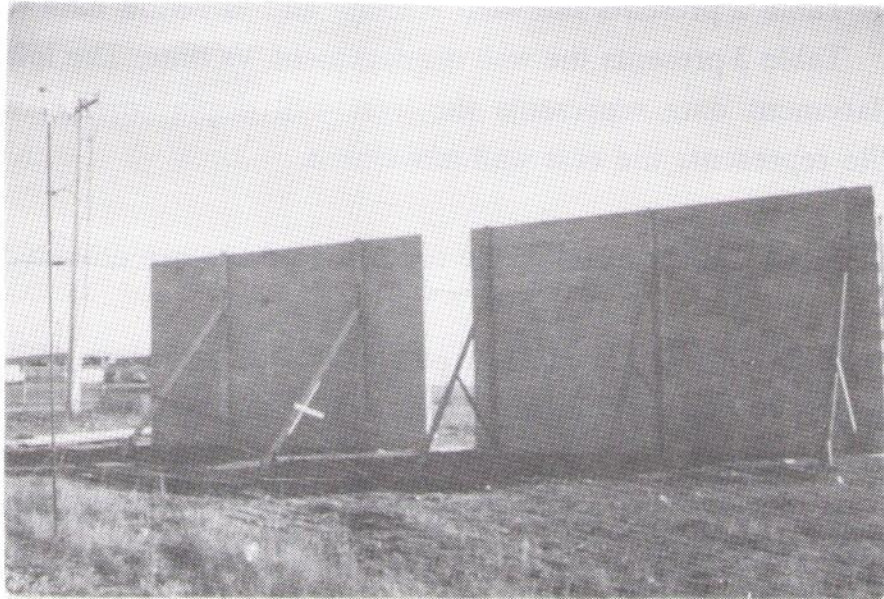**Photo 8.** East Wall South Side Grond Anchor System.

Photo 9. North Facing Wall Bracing.

Results:

Table 2 presents the wind velocity and direction data recorded.

Table 3 presents the wall displacement Vs time. The left profile of this displacement data represents the west wall displacements, while the right profile represents the east wall movement.

From Tables 2 and 3, the following results are obtained:

A 33.8 Km/h (21 mph) wind caused displacements of 3.20 mm (0.126 inches) at the top of the east wall and 1.27 mm (0.050 inches) at the top of the west wall.

A wind of 38.6 Km/h (24 mph) caused the following displacements at the top of the walls: 27.9 mm (1.1 inches) for the east wall and 2.79 mm (0.11 inches) for the west wall.

At 43.4 Km/h (27 mph), the top of the west wall recorded a deflection of 7.11 mm (0.28 inches). The east wall collapsed at a wind velocity between 38.6 and 43.4 Km/h (24 and 27 mph) as shown in Photo 10.

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.

Table 2. Wind Velocity and Direction Data.

7:00

8:00

0.178" Displacement

9:00

10:00
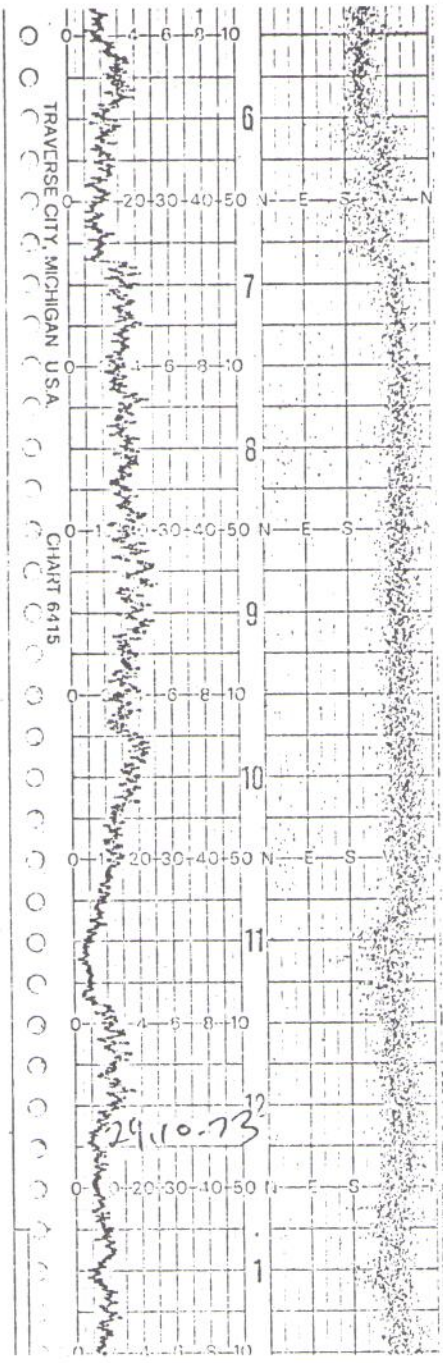
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CONSULTING ENGINEERING & TESTING

TABLE IV
WALL DISPLACEMENT
VS
TIME FOR OCTOBER 28, 1973

SCALE	DATE JAN. 74	MADE Y.K.	CHKD. D.E.	JOB: C421	PLATE

Table 3. Wall Displacement vs Time.

Photo 10. East Wall Blow Down.