NBSIR 88-3820

Collapse of Masonry Wall Under Construction in Pawtucket, RI, October 28, 1985

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National Bureau of Standards
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Prepared for:

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U.S. DEPARTMENT OF COMMERCE, C. William Verity, *Secretary*
NATIONAL BUREAU OF STANDARDS, Ernest Ambler, *Director*

ABSTRACT

Results from a study to determine the cause of the October 28, 1985 collapse of a masonry wall under construction in Pawtucket, RI are presented. The wall was a 60 ft-3 in. long, 23 ft-6in. high partially reinforced concrete masonry wall supported by wooden braces. The study included: inspection of construction plans and specifications; review of construction records and eyewitness accounts recorded immediately after the collapse, as well as testimony from OSHA inspectors and local building officials who visited the site a short time after the collapse; examination of photographs taken by OSHA inspectors and police investigators; analysis of meteorological data; and a stability analysis of the collapsed wall. It is concluded that the collapse was probably caused by a gust of wind which exerted lateral forces which exceeded the lateral-load capacity of the wall and its supporting wooden braces. Contributing factors were the lack of grout in the masonry cores which contained the steel reinforcement dowels and the inadequate anchoring of the dowels in the foundation.

Keywords: building; construction loads; construction safety; lateral bracing; masonry construction; stability; structural collapse; timber shoring; wind loads.

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1. INTRODUCTION

On October 28, 1985 at approximately 9:45 AM a partially reinforced concrete masonry wall which was under construction collapsed at the construction site of a Super Stop and Shop store located near the southeasterly corner of the intersection of Beverage Hill Avenue and Route 1A in Pawtucket, RI. The collapse occurred during a coffee break, when workers were sitting near the wall to get protection from wind. Three construction workers were killed and two were injured as a result of the collapse.

This report summarizes the findings of a National Bureau of Standards (NBS) study conducted at the request of the Occupational Safety and Health Administration (OSHA). The study was initiated in November 1986 and the findings were transmitted to OSHA in a memorandum report dated January 16, 1987. The purpose of the study was to analyze possible causes of the wall collapse and reasons why the bracing which supported the wall during construction did not prevent the collapse. Comparisons were also made between the strength of the bracing as installed and that required by applicable OSHA regulations and other applicable standards and criteria for bracing.

The wall extended from east to west, was 60 ft-3 in. long and 23 ft-6 in. high and had a 6x20 ft window opening 4 ft above the base of the wall and 25 ft-8 in. from its east end (see figure 1). The wall was supported by wooden braces. The collapse was apparently triggered by a wind gust from the northerly direction.

The masonry construction was carried out by a sub-contractor who was retained by the general contractor. Data on the construction details of the wall were obtained from plans and specifications prepared by the consulting architect, and from engineering inspection reports prepared by an inspection laboratory employed by the general contractor. Evidence from the collapse site in the

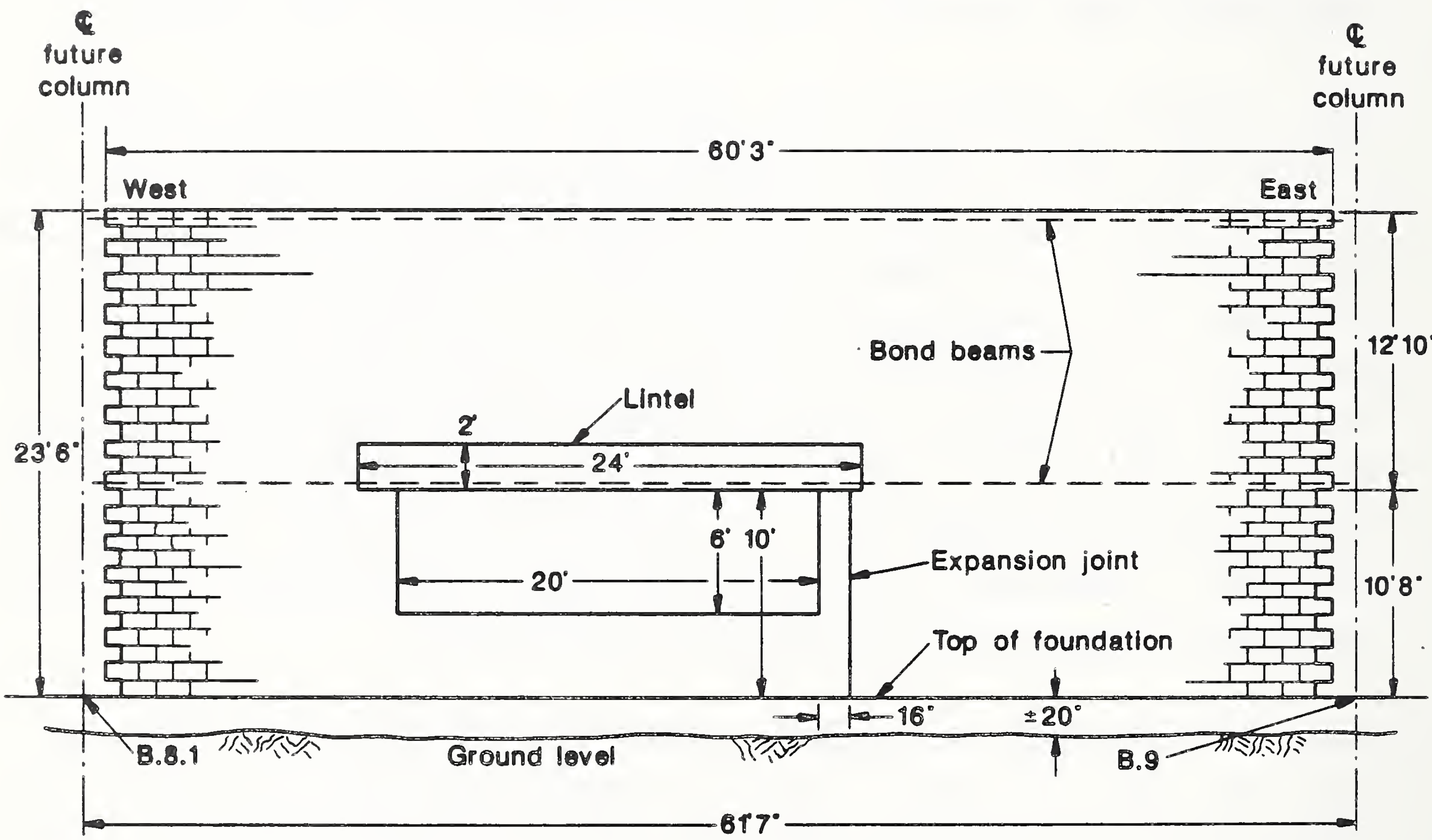


Figure 1: Elevation of Wall at the Time of Collapse, Looking from South.

form of Occupational Safety and Health Administration (OSHA) and police photographs, statements by OSHA inspectors who visited the site, and police records, including evidence by eye witnesses and photographs, was obtained from the Office of the Solicitor, Department of Labor, Boston, MA, the OSHA office in Providence RI, and the Building Commissioner, State of Rhode Island. NBS personnel visited the site in November 1986.

2. ESTIMATED WIND FORCES ACTING ON THE WALL AT THE TIME OF COLLAPSE

The following information was obtained from the National Weather Service (NWS) station anemometer located between runways of the Warwick, RI airport, about 12 miles South from the site of the accident, for October 28, 1985, 9:58 AM : Average wind speed 17 mph, gusting to 24 mph; wind from north-westerly direction; clear day with some high clouds. Wind speed was averaged over 1 minute and measured 20 ft above ground. The data were obtained by sampling the wind speed at 1-hour intervals for 1 minute (no attempt was made to pick out the "fastest" minute).

Other data for that day indicate the following:

Time (L.S.)	Average 1 min. wind speed	Max. gust	Direction (degrees)
8 AM	16 mph	--	350
9 AM	15 mph	--	320
11 AM	15 mph	22 mph	330
12 AM	16 mph	24 mph	340

Figure 2 shows a stripchart recording of wind speeds taken at the same NWS station. The wind speed in the chart is recorded in knots (1 knot = 1.151 mph). Note that the wind speed increased somewhat between 9:55 AM LST and 10:30 LST. At the accident site, this increase in wind speed probably occurred somewhat earlier, since the wind came from a northerly direction. From an

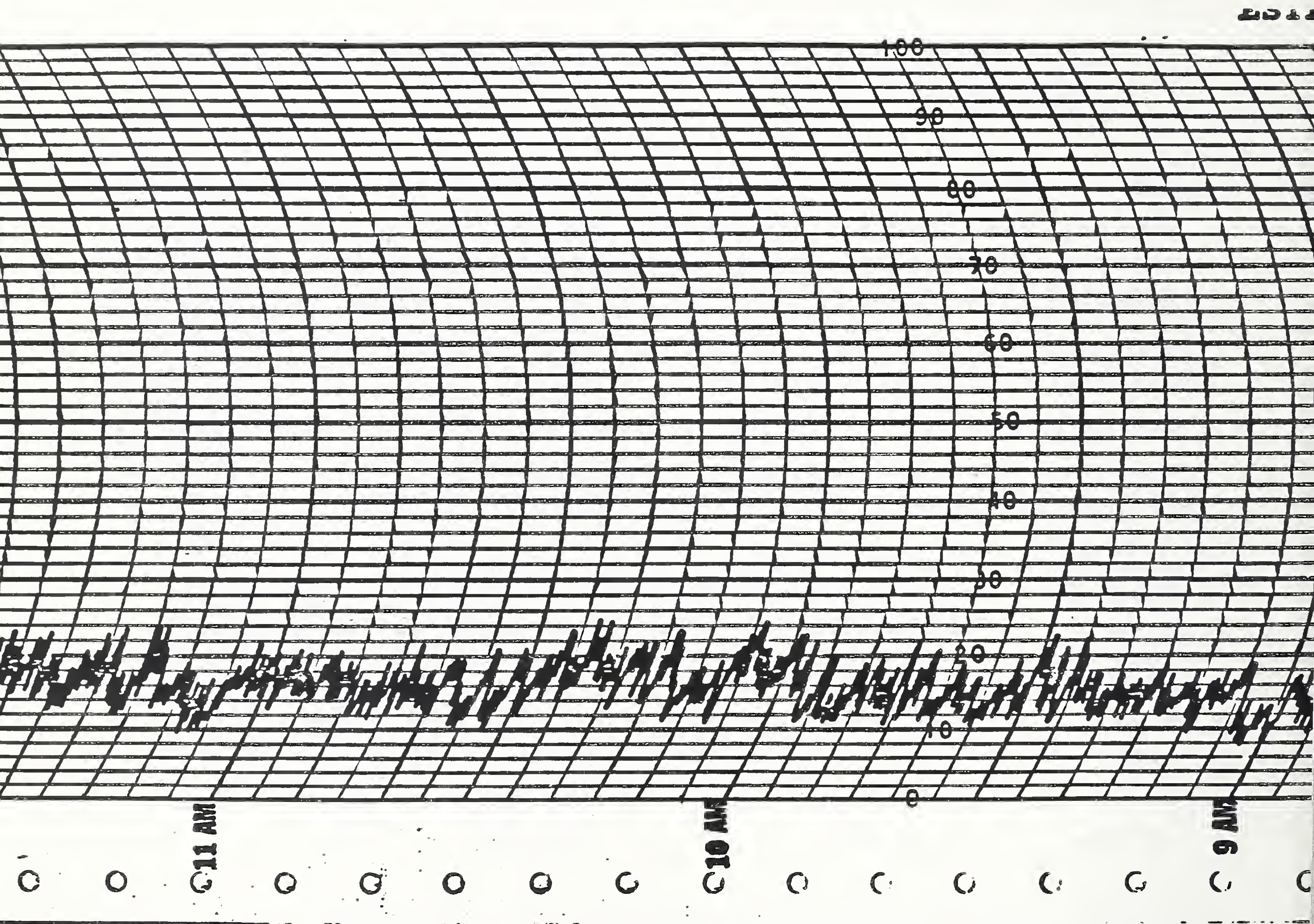


Figure 2. Stripchart Recording of Wind Speeds in Knots, Taken on August 28, 1985 at an Elevation of 20 ft. Above Ground at the NWS Station at the Warwick RI Airport.

examination of the recording, the fastest mile wind speed is estimated to have been 20 knots or approximately 23 mph.

Since this was a clear, windy day with no thunderstorms (which may be localized over a limited area) it is appropriate to assume that wind conditions at the location of the accident were similar to those at the weather station, differing only according to the characteristics of the terrain. It is assumed that the terrain at the weather station corresponds to "exposure category C" (open terrain with scattered obstructions having heights generally less than 30 ft)(refer to ref.1) and the terrain at the construction site is also in exposure category C, even though it is somewhat less exposed than the terrain at the weather station.

In accordance with reference 1, the total force exerted on a structure by wind can be calculated by the following equation:

$$F = q_z \, G_h \, C_f \, A_f \qquad \text{.........(Eq.1)}$$

where F is the force in lb, q_z is the velocity pressure evaluated at height z above ground, in psf, G_h is a gust response factor evaluated at a height z = H, where H is the height of the structure, C_f is a force (drag) coefficient, and A_f is the area of the structure projected in a plane normal to the wind.

The velocity pressure q_z in Eq.1 can be calculated as follows:

$$q_z = 0.00256 \, V^2 \qquad \text{..........(Eq.2)}$$

where V is the "fastest mile" wind speed, which is used as a reference speed in (1), measured at height z above ground.

The gust response factor for buildings in Ref.1 is given in Table 8, Ref.1. However in this instance, because of the size and aspect ratio of this wall, a gust response factor of 1 is more appropriate (refer to Fig. 3, Ref.1). The value of 1 is less, but considered more realistic than the more conservative gust response factors recommended in Table 8. The gust response factor $G_h = 1$ is used to estimate the windloads that acted on the wall.

The force (drag) coefficient C_f for rectangular flat plates which have aspect ratios similar to that of the wall under discussion and which have one edge in contact with the ground is approximately 1.2 for wind normal to the plate. Data from wind tunnel tests on flat plates which are not in contact with the ground indicate that as the direction of the wind changes from normal to 45°, C_d (calculated with respect to the full plate area, rather than the area projection normal to the wind direction) first slightly increases and then returns to approximately 1.2 for a wind direction which is inclined at about 45° with respect to the plate (Ref.2, pp156, 157, fig.4.6.3). The wind direction was apparently somewhere between normal to the wall and 45° inclination (the records indicate that the wind direction was somewhere between North and North-West). The 1.2 drag coefficient is in agreement with C_f stipulated in Ref.1 for "solid signs at ground level" (Table 13, Ref.1).

Wind speed in mph as a function of height above ground can be approximated by the equation:

$V = K z^{1/7}$ for "exposure C" (Ref.1 flat open terrain)....(Eq.3)

where K is a coefficient which depends on wind speed and z is the height above ground in ft.

The estimated wind load acting on the wall can be reasonably represented by a uniformly distributed pressure, q_e, acting normal to the wall surface from the windward direction (actually it consists of pressure on the windward side and suction on the leeward side). Pressure q_e can be derived from Eq.1, using a value of q_z calculated for a z value of z = H, where H is the height of the wall (23.5 ft.), a G_h value of 1, and a C_f value of 1.2. Thus, from Eqs. 1, 2, and 3:

$$q_e = 0.00256 x 1.2 x K^2 x H^{2/7} \quad ..... \text{(Eq.4)}$$

where q_e is the equivalent uniformly distributed pressure representing the wind load in psf, and K is the wind speed coefficient in Eq.3.

To obtain the estimated fastest mile wind speed at the construction site, it is necessary to account for the difference between the terrain at the weather station and that at the construction site. In this instance it is estimated that the fastest mile wind speed at the construction site was close to that recorded at the weather station, which was 23 mph.

In accordance with Eq.3 the K coefficient for the 23 mph wind speed is 14.99. The equivalent wind pressure on the wall, calculated by equation 4 is therefore:

$$q_e = 0.00256 x 1.2 x 14.99^2 x 23.5^{2/7} = 1.701 \text{ psf.}$$

The following wind loads are calculated for the wall as a whole, using the 23 mph fastest mile wind speed derived from from the stripchart recording:

Mw = 26,870 ft-lb; Vw = 2,204 lb

where Mw is the total overturning moment at the base of the wall caused by the wind load, and Vw is the total resultant shear force at the base of the wall.

High winds were also recorded on the three days prior to the accident. On Sunday, October 27, the recorded fastest mile wind was 25 mph from a direction of 300°, on Saturday October 26, the fastest mile wind was 18 mph from a direction of 320°, and on Friday October 25, the fastest mile wind was 21 mph from a direction of 310°. These wind conditions could have contributed to the the wall failure by rupturing the mortar bond or otherwise weakening the resisting moment of the wall prior to the accident.

3. ESTIMATED LOAD RESISTANCE PROVIDED BY THE BRACES

3.1 Probable Bracing Configuration

Figure 3 shows braces supporting walls on the same construction site that did not collapse in the accident. The photographs were taken on the day of the accident. The braces shown in figure 3(a) consisted of two 16 ft long 2x10 planks. The upright plank had a 2x4 cleat nailed to it which resisted the upward thrust of the diagonal strut. The diagonal strut rested on the soil with no noticeable embedment and no footing to resist downward thrust, and no stakes were driven into the soil to resist horizontal forces and displacements. The diagonal struts are restrained by one concrete masonry block placed at their lower end and weighted down by two concrete masonry blocks, apparently to hold them in place. A closeup photograph of the lower end of the diagonal strut is shown in figure 3(b). Figure 3(c) shows a strut which was inclined at a very steep angle, apparently to brace a higher wall. The upper portion of that latter brace is shown in figure 3(d). The vertical bracing members examined in the field were not nailed or otherwise attached to the masonry wall and were held in place by the horizontal thrust exerted by the diagonal member (aided by the weight of the two masonry blocks resting on the diagonal member).

The evidence shown in figure 3 indicates for the braces shown that: (1) the two 16 ft long planks were used even when the wall was very high (in the latter case the angle of inclination of the

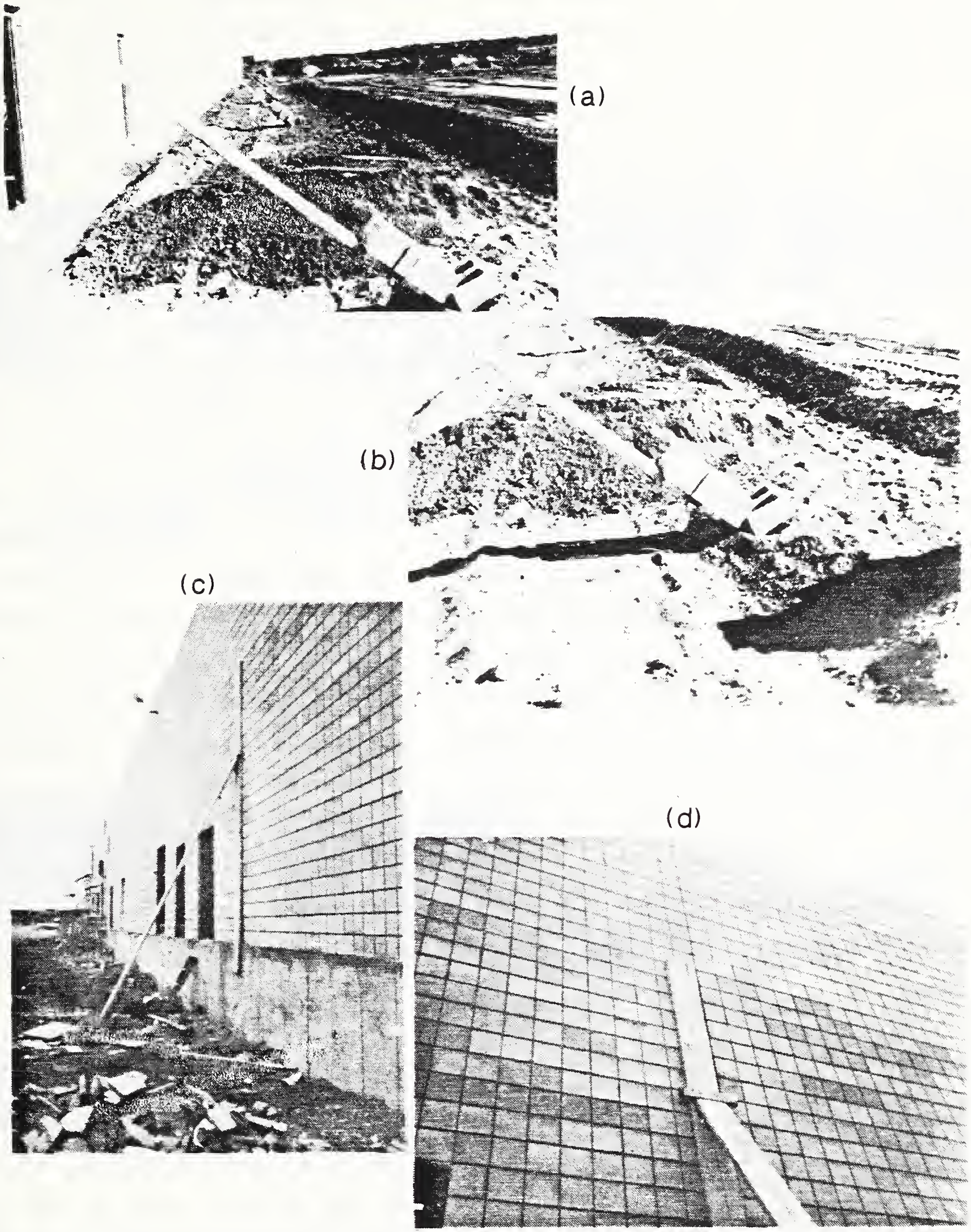
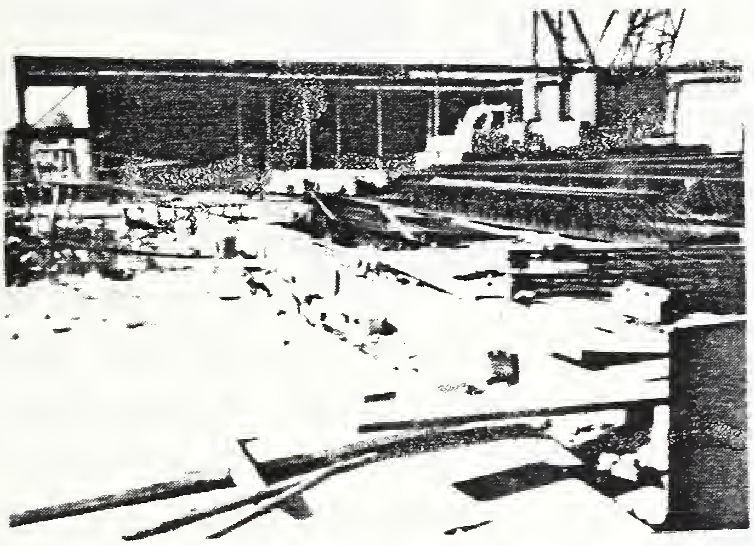


Figure 3: Braces Supporting Other Walls on Construction Site
(photograph taken on day of collapse).
(a) Typical braces.
(b) Strut support at ground level.
(c) Brace with steeply sloped strut.
(d) Cleat for brace with steeply sloped strut.

diagonal strut to the horizontal was increased in order to support the wall at a higher level); (2) no intermediate support was provided to reduce the unbraced length of the diagonal strut; (3) no attempt was made to anchor the lower end of the diagonal strut in the soil or otherwise prevent it from moving horizontally (except for the restraint provided by the weight of one concrete block) or to increase the soil bearing area in order to prevent excessive settlement of the strut.

Figure 4 shows the evidence of bracing found on the windward (north) side of the collapsed wall. Figure 4(a) shows a view from east to west. It shows collapsed braces at the east end of the wall (column B9 in plan drawing A 1.1, dated July 15, 1985) and near the center of the wall (to the east of the window opening). It can be seen that, like the braces shown in figure 3, each brace consisted of two 16-ft long 2x10 planks, one of which had a cleat nailed to it. Figure 4(b) shows an end view of the collapsed brace in the center of the wall together with the three concrete blocks which were used to restrain the diagonal strut. Figure 4(c) shows the brace at the west end of the wall (col B8.1) and figure 4(d) shows the restraining concrete blocks for the brace in figure 4(a). The western diagonal was restrained by a pipe (not shown). The distance from the lower end of the vertical plank to the underside of the cleat was 9 ft 9 in for the brace at the east and west end of the wall and 10 ft -4in. for the brace at the center.

It is concluded from figure 4 that: (1) the braces used on the windward side of the collapsed wall were similar to those shown in figure 3; (2) the wall was supported by three braces on the windward side; (3) the height at which the diagonal struts supported the wall equaled or exceeded the distance from the bottom of the vertical plank to the underside of the cleat, but could not be less than that distance (for instance in figure 3(c) the bottom of the vertical plank is higher than the ground, but



(a)

(b)

(c)

(d)

Figure 4: Collapsed Braces on the Windward Side of the Wall.

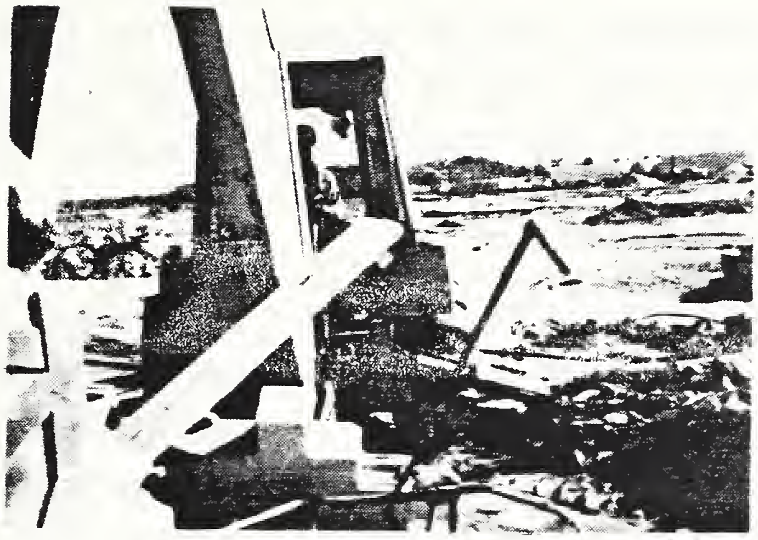
(a) View from East to West showing collapsed braces near East end and near center of wall.
(b) End view of collapsed brace near center of wall.
(c) Collapsed brace near West end of wall.
(d) Restraining concrete masonry units for brace near East end of wall.

it is unlikely that the vertical plank was pushed into the ground).

Figure 5 shows some of the evidence of bracing found on the leeward (south) side of the collapsed wall. In this instance, bracing members were damaged either by the wind load prior to the collapse or by the falling debris from the wall. Figure 5(a) shows a vertically split 2x10 plank at the east end of the wall (col.B9). This was a vertical bracing member and the cleat that has been nailed to it near the upper end can be seen in the picture. Figure 5(b) shows a shattered 2x10 plank on the west end of the wall. Figure 5(c) shows a broken 2x10 plank, covered by debris, near the west end of the wall, looking south. Examination of the field evidence which started within about 60 minutes after the collapse and continued through the removal of the debris turned up components of two braces similar to those shown in figure 3. There is no evidence that there was a third brace near the center of the wall. The location of the cleats in the two braces found was 16 inch from the top of the vertical plank to the underside of the cleat for the brace at the east end of the wall, and 24 inches from the top of the plank to the underside of the cleat for the brace at the west end of the wall.

It is also important to note that the ground elevation at the leeward side of the wall was below the base of the wall (part of the foundation was exposed-see figure 1). At the east end of the wall the ground elevation was approximately 20 inches below the base of the wall and at the west end the ground elevation was approximately 10 inches below the base of the wall.

It is deduced from figure 5 and observations reported by OSHA inspectors that, on the leeward side, the wall was probably supported by two braces located near the two ends of the wall, which were similar to the braces shown in figure 3, and that near the east end of the wall (B9 in plan drawing A 1.1) the point of



(a)

(b)

(c)

Figure 5: Bracing Members Found on Leeward Side of Wall.

(a) 2x10 plank with cleat near East end of wall.
(b) Shattered 2x10 plank near West end of wall.
(c) Broken 2x10 plank near West end of wall, looking South.

support provided by the diagonal brace was 13 ft or more above the base of the masonry wall (the top of the footing) [16 ft - 16 inches (top of plank to bottom of cleat) - 20 inches (base of wall to ground)].

Near the west end of the wall (B8.1) the point of support provided by the diagonal brace was 13 ft-2 inches or more above the base of the masonry wall (16 ft - 24 inches - 10 inches). This implies that, even if the vertical members rested on the ground, the inclination of the diagonal member was steep, similar to that of the brace shown in figure 3(c). On the east end the inclination was 14.7 vertical to 6.4 horizontal or approximately 67^{o} to the horizontal. On the west end the inclination was 14 vertical to 7.8 horizontal or approximately 61^{o} to the horizontal.

Figure 6 shows a plan view of the wall and the probable location of the braces on both sides of the wall, based on measurements and observations by OSHA inspectors who visited the site after the collapse. The cross section of the collapsed wall and the probable geometry of the braces on the leeward side which was deduced from the available evidence is shown in figure 7. There was 8 in. block to an elevation 10 ft-8 in. above the base of the wall and 12 inch block above that elevation. It is therefore logical, that the support was provided above the 10 ft-8 in. elevation from the base of the wall. Evidence from field observations supplied by OSHA inspectors indicates that during construction the overhanging portion of the 12 in wall was supported by vertical 2x4 shores to prevent uneven settlement of the mortar bed. No traces of these 2x4 shores were found in the debris of the collapsed wall, indicating that these support members were probably removed before the collapse. Figure 8 shows the photograph of another wall on the construction site from which the 2x4 shores were removed, which was taken one day after the collapse.

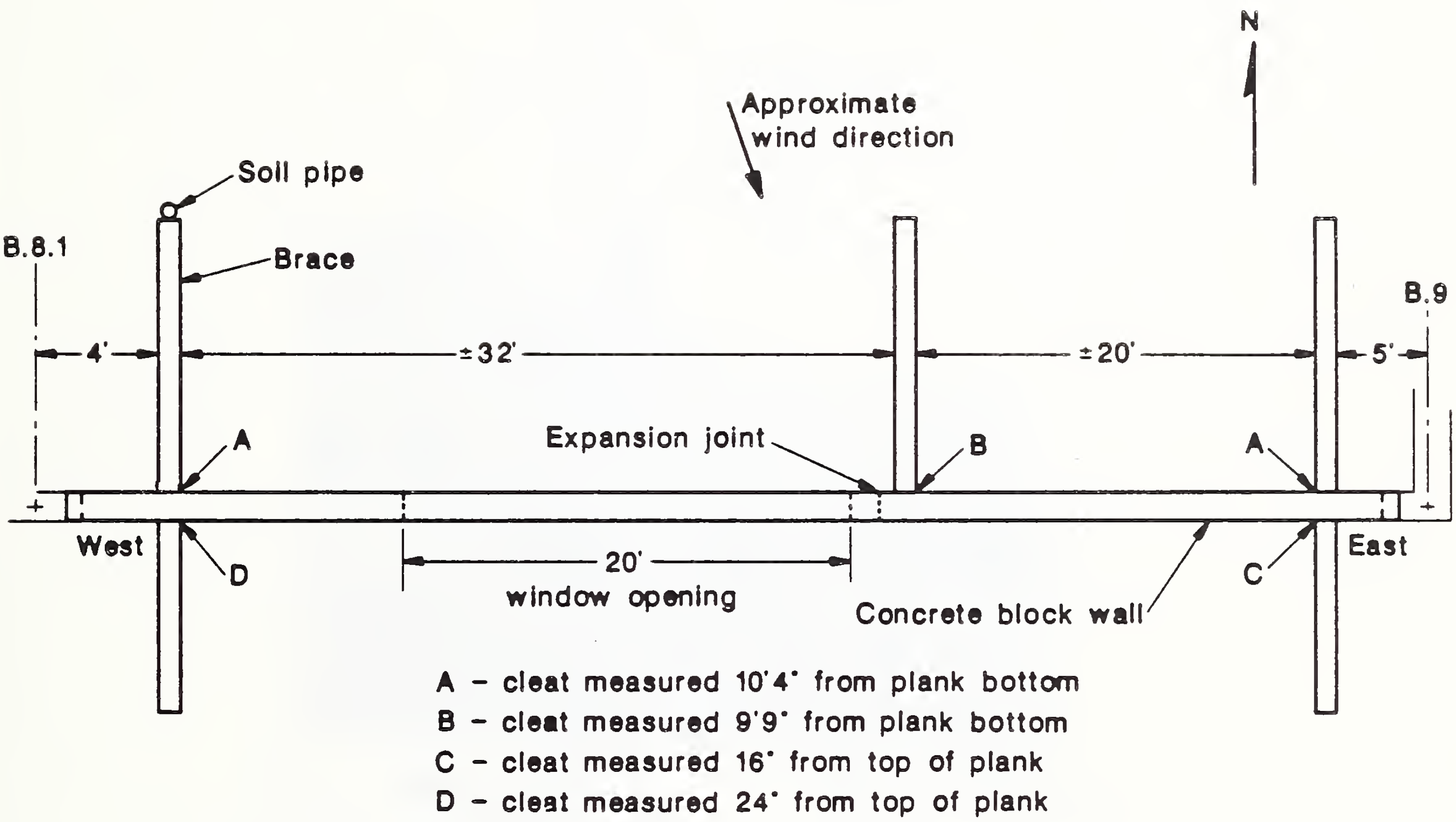


Figure 6: Top View of the Collapsed Wall Showing Probable Bracing Location

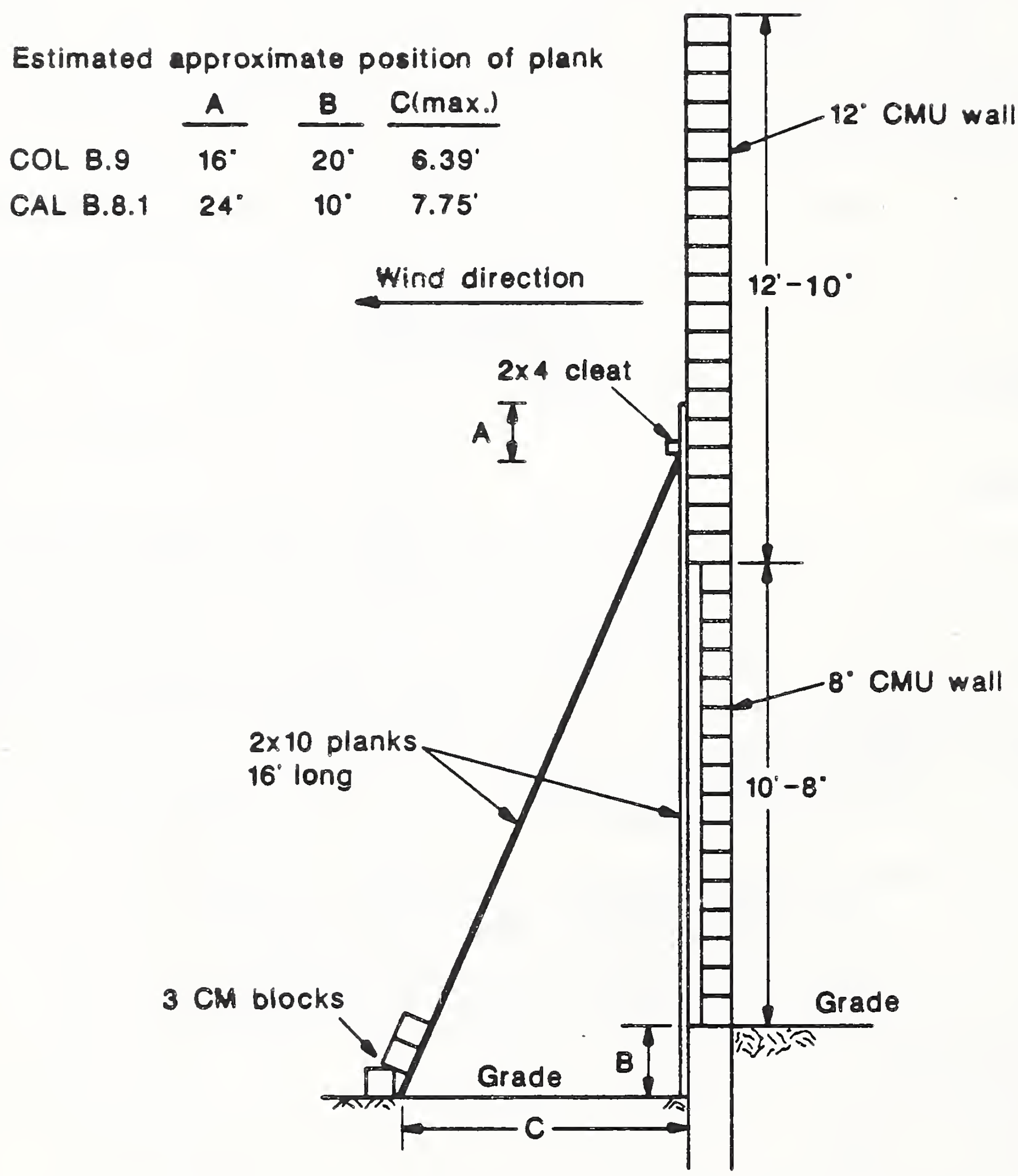


Figure 7: Probable Bracing Configuration on Leeward Side of Collapsed Wall.

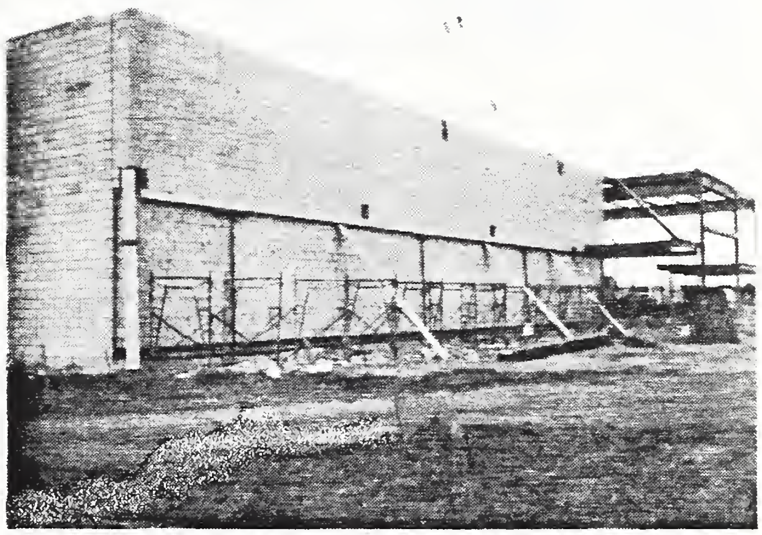
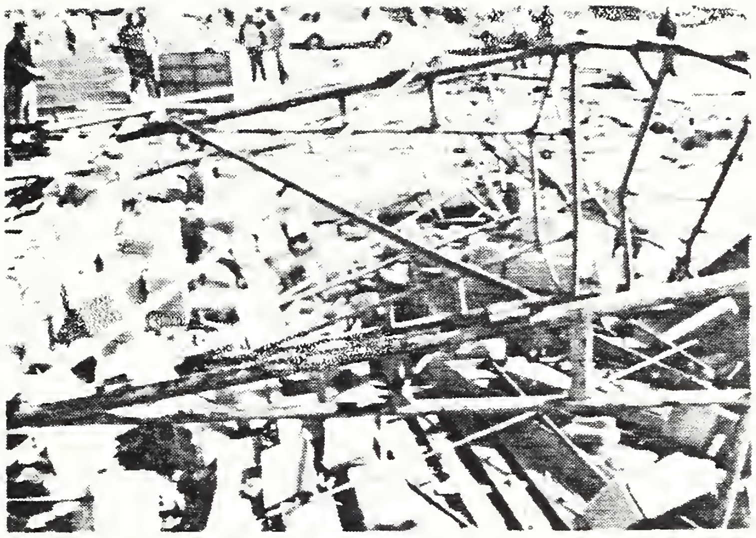


Figure 8: Wall on the Same Construction Site from which the 2x4 Planks Supporting the Overhanging Wall Were Removed.

Evidence from photographs and reports by OSHA inspectors from the collapse site indicates that in addition to the braces described above there were scaffolds adjacent to the wall. These consisted of scaffolds under the window opening and scaffolds on the leeward side at the two ends of the wall. The scaffolds under the window opening were originally used to provide support to the concrete forms for the lintel. The scaffolds were adjusted by screw jacks at the bottom. After the lintel was poured, the screw jacks were relaxed to remove the forms. Evidence provided by the OSHA inspectors indicates that after removal of the forms these screw jacks were tightened lightly, but no upward thrust was exerted against the wall. The scaffolds at the two ends of the wall were left in place for future use to finish the connection of the ends of the wall to the columns which were to be erected at both ends of the wall. These scaffolds did not bear against the wall. The remains of the scaffold that supported the lintel form are shown in figure 9(a). Figure 9(b) shows a scaffold recovered from the wall (on side) and an undamaged scaffold. The probable position and configuration of the scaffolds prior to the collapse is shown in figures 10 and 11. There is no indication that these scaffolds could provide significant lateral support to the wall.

3.2 Estimated Load Capacity of the Bracing

Several possible failure mechanisms could have led to the collapse of the braces shown in figure 7. Possible failure mechanisms considered are: (1) structural failure of diagonal plank; (2) upward sliding of vertical plank; (3) kickout or settlement of diagonal plank at ground level; (4) shortening of diagonal plank as a result of sag. These failure mechanisms are discussed below.



(a)

(b)

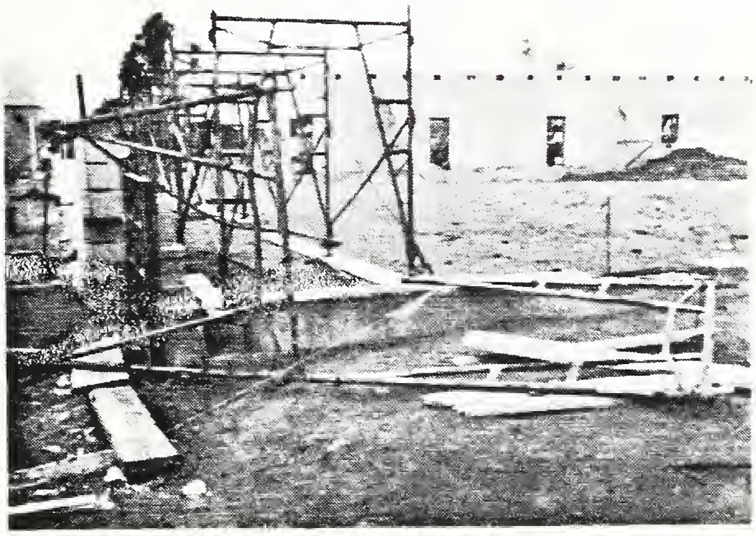


Figure 9: Scaffolding that was Used Along the Collapsed Wall.

(a) Remain of Scaffold that Supported Lintel Forms.
(b) Scaffold Recovered from Collapsed Wall (on side).

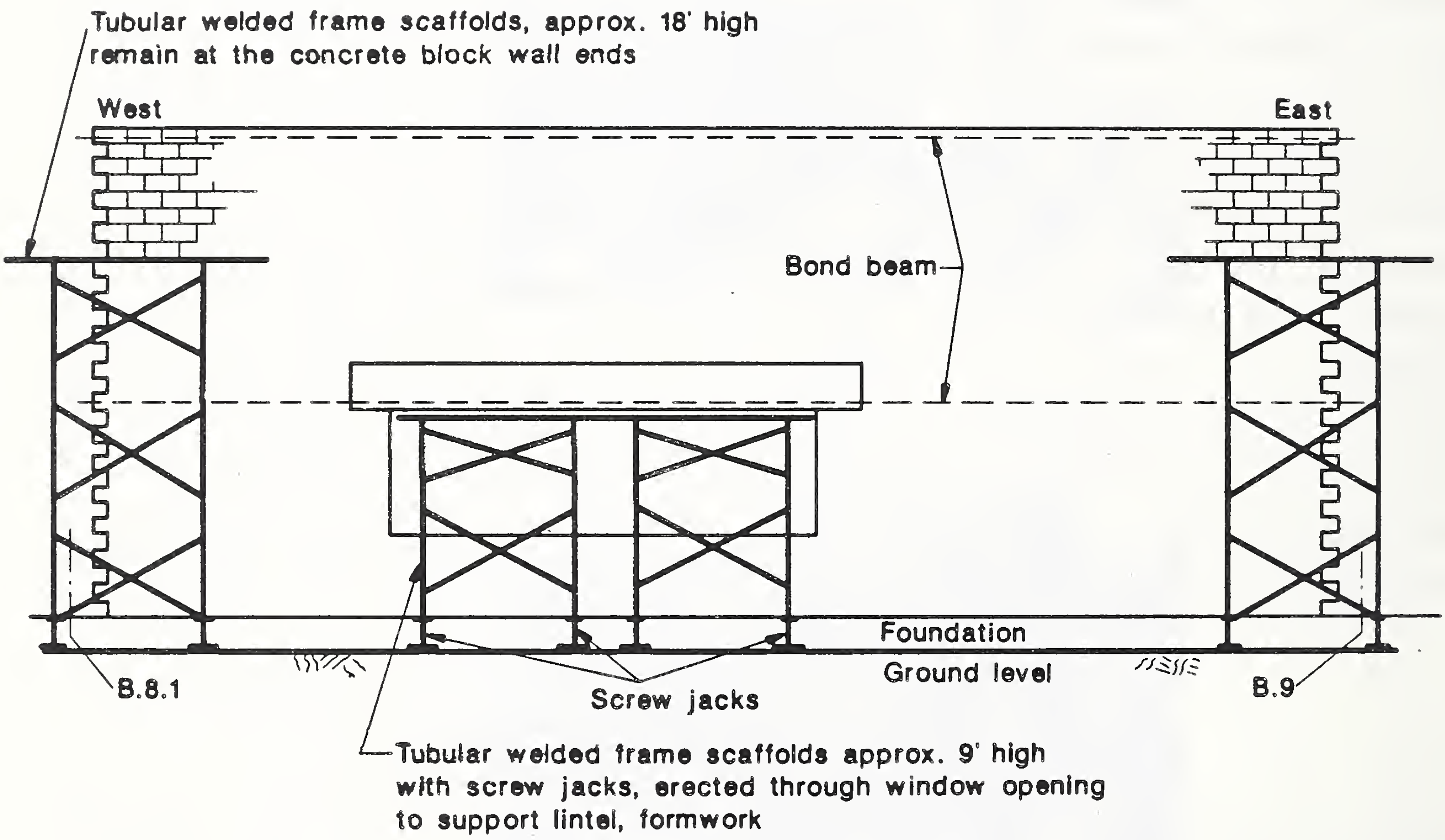


Figure 10: Front View of Scaffolds Looking From South.

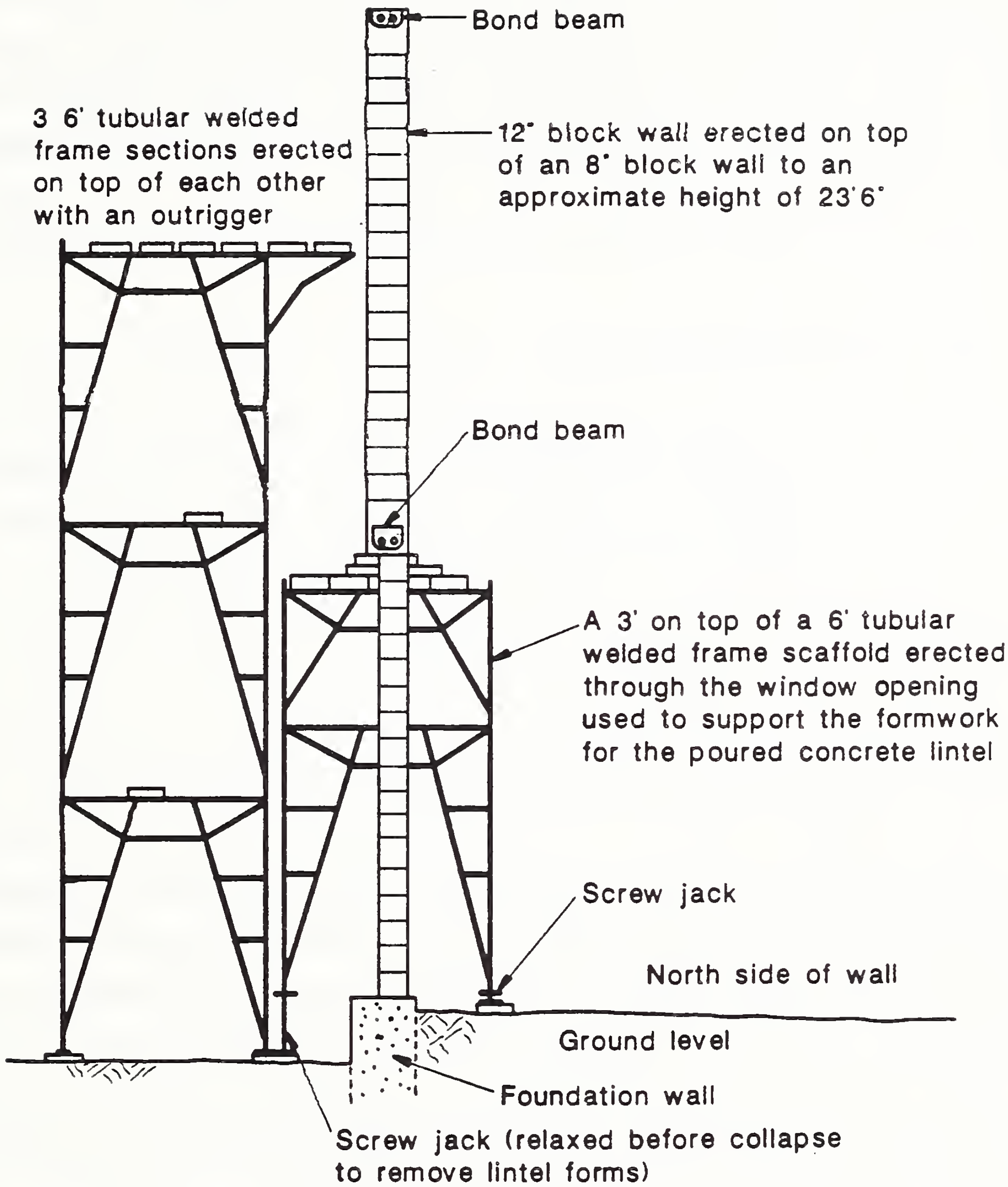


Figure 11. Side View of Position of Scaffolds

(1) Structural failure of diagonal strut:

The actual dimensions of the 2x10 strut are 1.5 in. x 9.25 in. The 1.5 in. narrow dimension, combined with the 16 ft length make this strut a very slender member. Neither of the ends of the strut has any significant restraint against rotation, so that the strut will act like a column which is pin ended at both ends. Its propensity for buckling is enhanced by the initial downward sag, caused by its own weight. The upper bound for the ultimate buckling load can be calculated by the "Euler equation":

$$P = (3.1416^2 \ EI)/(L)^2 \quad \ldots\ldots\ldots\ldots \quad \text{(Eq.7)}$$

where: E is Young's Modulus for the wood, estimated at 10^6 psi
I is the moment of inertia of the cross section
L is the length of the plank = 16x12 inches.

$$I = (9.25 \times 1.5^3)/12 = 2.6010 \text{ in}^4$$

$$P = (3.1416^2 \times 10^6 \times 2.6016)/(16 \times 12)^2 = 697 \text{ lb.}$$

The actual ultimate load capacity of the strut, assuming that flexural failure would occur prior to buckling as a result of the amplification of the initial downward sag by the axial load, at an approximate ultimate bending stress of 2,500 psi, would be 660 lb. Only the horizontal component of this load can effectively resist the wind pressure. The following resisting moments would therefore be exerted by the struts on the base of the wall at the instant of strut failure:

At B9 (east side): Mr = 660x13x6.39/16 = 3,427 ft-lb
At B8.1 (west side): Mr = 660x13.17x7.75/16 = 4,210 ft-lb

Since the window opening is nearer to the west end, the east end brace which has a lower load capacity was probably more heavily loaded. But even if both struts failed simultaneously, the maximum overturning moment over the base of the wall that could be resisted by the braces was 7,600 ft-lb. This compares with an estimated wind induced overturning moment of 26,900 ft-lb. Thus if the braces alone had to resist the wind load, their failure under the prevailing wind condition was a probable event.

(2) Upward Sliding of Vertical Plank

The vertical component of the axial strut load must be resisted by frictional forces between the vertical plank and the masonry wall. Since the horizontal component of the axial strut force is less than 50% of the vertical component (refer to figure 6), a friction coefficient of more than 2 is required to develop the required frictional resistance. Upward sliding of the vertical plank would cause the brace to fail. No reliable data on the friction coefficient between an aged timber plank and a concrete masonry wall could be located and no laboratory experiments were conducted to obtain information. However the required friction coefficient is high when compared with accepted engineering practice. For instance ref.(3) recommends a range from 0.5 to 0.6 for the static coefficient of friction of masonry on wood for the purpose of engineering design (Table 3.1). Sliding failures could have been prevented by nailing the vertical board to the wall or by a cleat under the 12 in. wall at the transition point between the 8 in. and 12 in. wall. No evidence was found to indicate that either of these precautionary measures was taken.

(3) Kickout or Settlement of Diagonal Plank

It is deduced from figures 2 and 3 that the diagonal plank simply rested on the ground, was restrained by one concrete block and weighted down by two blocks. In accordance with field density tests performed on the construction site the soil on which the planks rested was compacted fill, which is considered competent soil for support of proper footings. However the arrangement as installed entirely relied on the surface condition of the soil (the upper 2 to 5 inches), which is not predictable. Failure by horizontal displacement is not considered likely in this case, because of the steep inclination of the diagonal strut. However there was little resistance to settlement (pushing of the strut into the ground), at least for the initial 1 to 2 inches until the soil under the strut is compressed by the downward thrust. It is important to note that each 1 in. of downward displacement of the strut in the inclined direction of its axis would cause a horizontal displacement of 2 to 2.5 inches at the point where the strut supports the wall. Good practice would require, as a minimum, to restrain the diagonal struts by stakes, driven into the ground (ref.4), or preferably to provide an adequate footing to prevent displacement at the ground level (ref.5).

(4) Shortening of Diagonal Plank

The diagonal strut is very slender and flexible. A large lateral displacement at the cleat is required to develop the load capacity of the strut. By the time the diagonal strut is loaded to 630 lb it experiences a deflection (sag) of approximately 8 in. in its center which in turn will cause the distance between the cleat and the ground support of the strut to shorten by more than 1 inch (measured along the initial undeflected axis of the diagonal strut). This shortening is associated with a horizontal displacement of the wall at the cleat of more than 2.5 inches. At this displacement the resisting moment provided by the gravity load of the wall itself would be zero. The displacement caused by the sag of the plank could be further increased by settlement of the strut at its ground support. This "softness" of the strut will cause an initial failure at the base of the wall and a tilt in the wall before the brace resistance is effectively mobilized. Had the struts been adequately braced against buckling, their shortening at the 630-lb load level would have been about 0.01 inches.

4. LOAD RESISTANCE PROVIDED BY THE WALL

4.1 General Discussion

In order to overturn, a wall must rotate over the leeward edge of its base. In addition to the shoring, the wall itself, if properly constructed, provides resistance to overturning, which consists of the resisting moment provided by the weight of the wall and the resisting moment provided by the tensile strength of the mortar beds and the tensile resistance of the grouted dowels connecting the wall reinforcement to the foundation, which is small immediately after construction but increases rapidly with time. Since it took time to construct the reinforced masonry wall to its 23 ft-6 in. height, the grouted dowels and the mortar, had they been constructed in accordance with plans and specifications, would have provided significant resistance to the wind load. It is therefore important to establish the probable conditions of the mortar and the grouted dowels at the time of the collapse.

4.2 Condition of Wall

In accordance with the plans and specifications, this was to be a partially reinforced, grouted masonry wall, whose lower 10 ft-8 in. part is of composite construction having an 8-in. concrete block wythe and a 4-in. clay brick wythe, connected by metal ties, and whose upper 12 ft-10 in. part consisted of 12-in. concrete block. The wall was to be reinforced vertically with #5 bars placed 2 ft O.C. in grouted cores and with 2 #5 bars each placed in horizontal bond beams at its mid-height and at its top. The connection between the wall and the strip footing which supported it was to be accomplished by 5 ft long #5 dowels, spaced 2 ft O.C. and embedded 2 ft-6 in. in the foundation. These dowels extended from both ends of the wall to a distance of 4 ft-8 in. from either side of the 6x20 ft. window opening. From these points to the edges of the window opening the plans specified #6 dowels spaced 8 in. O.C. The plans show flashing on the south side of the wall, to be installed between the brick and the block wythes and to extend from the underside of the bottom brick course to the top of the second 8-in. masonry course. Masonry block were to be units with a 1500 psi gross-area compressive strength, which are specially fabricated for reinforced masonry and shaped so that the cores would line up vertically for easier grouting. Specified grout was to be ASTM C476 (2,500 psi) and specified mortar ASTM C270 Type S mortar. Grout with coarse aggregate was to be used to grout large cores with reinforcement and grout with sand only to grout narrow cores. Grouting was to be low lift, with a lift (pour) height of 4 ft.

In accordance with the specifications masonry work was to be continually inspected in the field under the supervision of a professional engineer.

Actual field evidence indicates that the wall that collapsed was not constructed in accordance with the plans and specifications summarized above. The following deviations are noted:

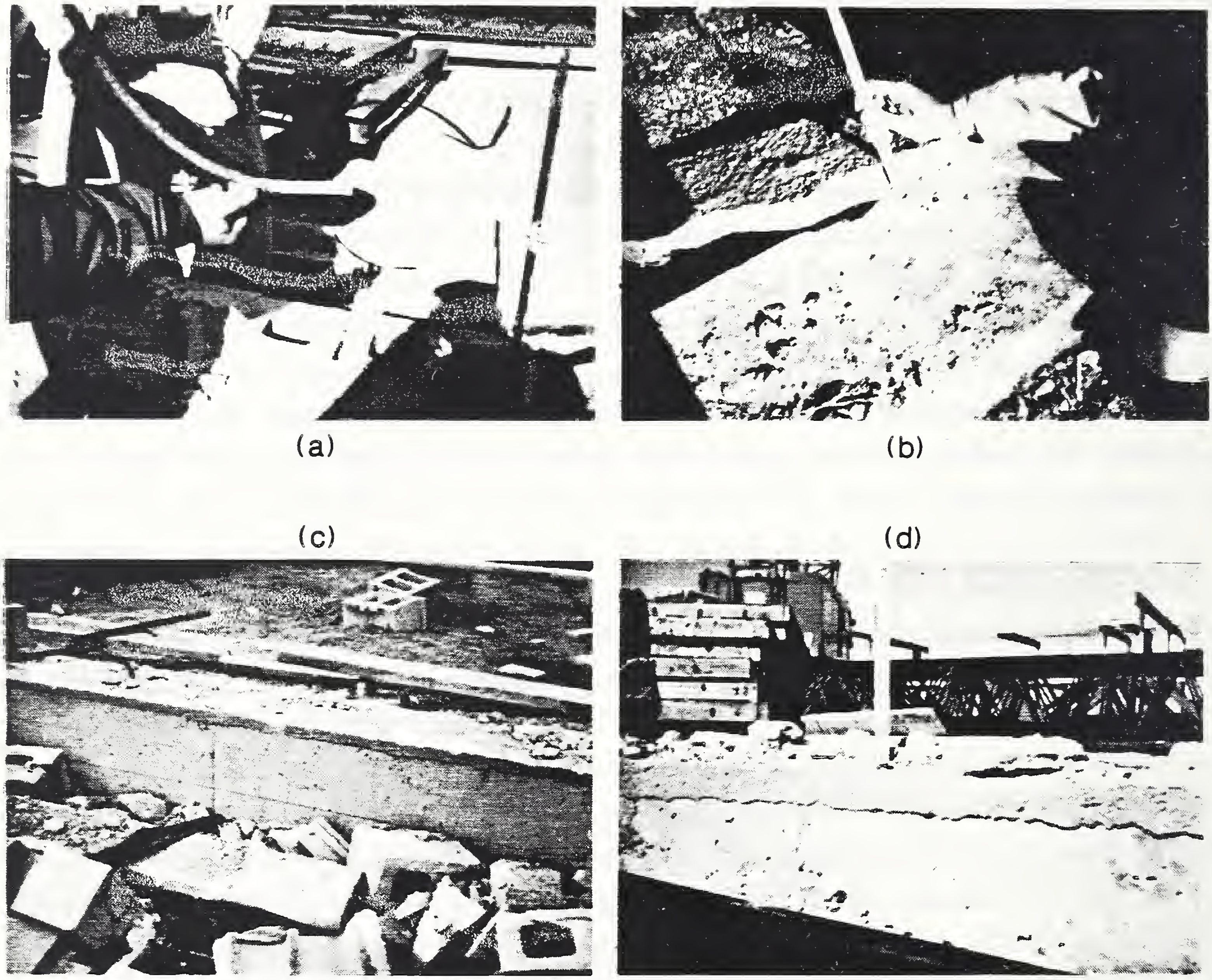
(1) The dowels were not engaged by the foundation

Available records indicate that the dowels were omitted when the original footing was poured. To remedy this situation, the contractor drilled holes into the foundation and grouted the #5 dowels into these holes. No #6 dowels were found at the collapse site even though they were called for in the plans. The depth to which the holes were drilled was 5 inches, and sometimes somewhat less. Thus the dowels were embedded in the foundation a maximum of 5 inches (the plans called for 2 ft-6 in. embedment). However even the 5 in. embedment was not fully effective. Apparently the top of the foundation, as originally poured, was about 2 in. below its intended elevation. To remedy this situation, a 2 in. concrete cap was poured. As a result, the dowels were embedded 3 inches in the foundation and 2 inches in the cap, which was not bonded to the foundation.

The above-discussed situation is illustrated in figure 12. Figure 12(a) shows a dowel which was pulled out of the foundation by an OSHA inspector. The part of the dowel that was embedded in the foundation can be seen by its white coloration (the rest of the dowel was not embedded in concrete at all). Figure 12(b) shows the 2-in. concrete cap, figure 12(c) shows the foundation with the cap and a solitary dowel, and figure 12(d) shows the depth of the hole from which a dowel was pulled out. In accordance with oral statements to the author by the OSHA inspectors who visited the site one man could extract the dowels from the foundation with ease, by pushing them to the side in two opposite directions and then pulling up. Minimum embedment for #5 dowels that would permit development of their tensile strength would have to be 24 dowel diameters or 15 inches. If the 2 inch cap is added to this, the dowel would have to be embedded at least 17 inches to be fully effective. It should also be noted, that even with the 3 inch actual embedment it would not have been as easy to extract the dowels, had they been effectively grouted (it would take about 1,100 lb to pull out the bars if the grout achieved 1/2 its 28-day strength).

(2) The grouting of the dowels and the other vertical reinforcement into the masonry units was not sufficient to attain the required structural performance.

The dowel shown in figure 12(a) was not grouted into the masonry cores. This is further illustrated in figure 13(a) which shows



(a) (b)

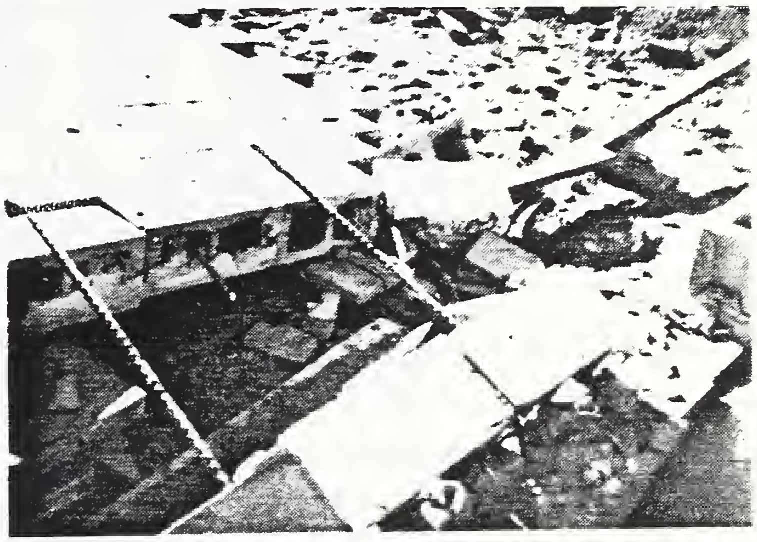
(c) (d)

Figure 12: Pulled Out Dowel and Unbonded 2" Concrete Cap on Footing of Collapsed Wall.

(a) Dowel pulled out by inspector.
(b) 2" unbonded concrete cap.
(c) Footing with concrete cap and remaining dowel.
(d) Depth of hole from which dowel was Pulled.

(a)

(b)



(c)

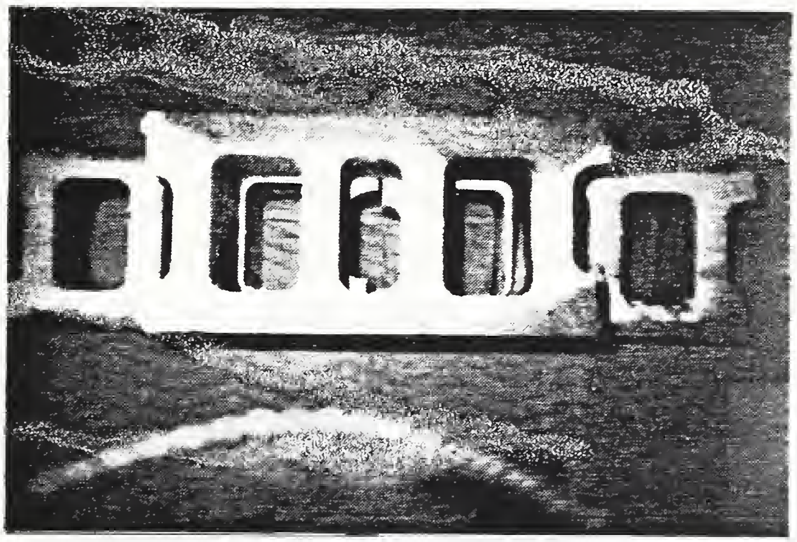
Figure 13: Grouting of Dowels and Other Masonry Reinforcement into Masonry Cores.

(a) Failure surface with protruding ungrouted dowel.
(b) End view of collapsed wall with ungrouted reinforcement bars.
(c) Holes opened in wall that did not collapse. Cores were found to be ungrouted, and in some instances lacked specified reinforcing bars.

dowels and other reinforcement protruding from masonry units. The bars were rust colored, an indication that they were not embedded in grout. This is further illustrated in figure 13(b). Figure 13(c) shows holes which were opened to inspect other masonry walls on the same project. In many cores which were intended to be reinforced no grout was found, and in some cores the reinforcing bars were also missing.

The masonry units actually used in the construction were standard units rather than the special units specified in the plans, which are specifically designed so that the cores of the units line up. The resulting problem is illustrated in figure 14. Figure 14(a) shows stacked special units where the cores line up. Figure 14(b) shows stacked standard units, where part of the core is blocked by successive courses of masonry. This results in cores which are more difficult to fill with grout in a satisfactory manner. While present standards (6,7) permit the use of such units, they do not recommend it. There is also no evidence that cleanout windows were used at the bottom of the grouted cores to permit removal of debris from the cores before they were grouted.

The flashing, which consisted of a vinyl strip, was put over the top of the first course of masonry rather than the second course. This resulted in a situation where a substantial portion of the top of the first-course unit was covered by flashing, the bottoms of the cores to be grouted were partially blocked by flashing, and one mortar bed was laid on top of flashing thus interrupting the tensile bond. A slot was cut in the flashing where there were dowels. This permitted placement of the flashing around the dowel, but blocked the rest of the core. Thus even if some of the grout reached this level (much of it did not), the flashing prevented it from reaching the top of the foundation. Figure 15 shows the flashing strip with the slot to accomodate the dowel, pealed away from the top of the first course of masonry.



(a)

(b)

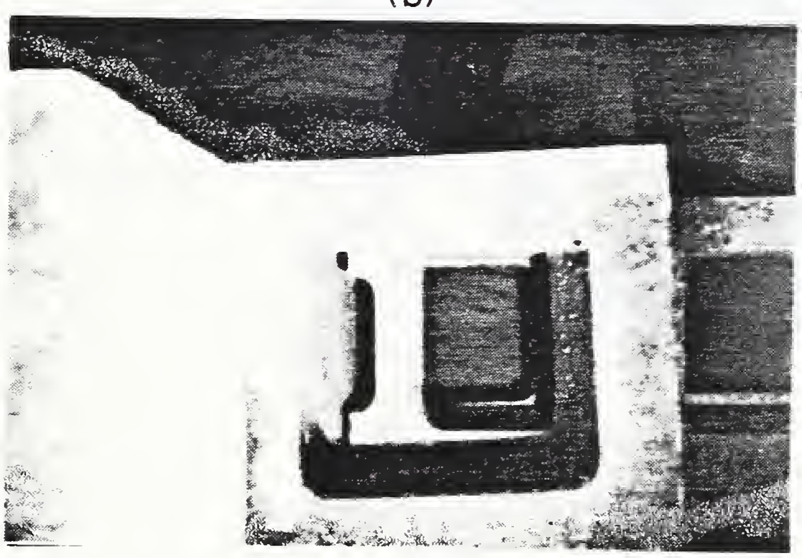


Figure 14: Core Alignment in Stacked Masonry Units

(a) Stacked Ivony Type block (cores aligned).
(b) Stacked standard CMU units (cores not aligned).

Figure 15: Flashing Strip Peeled Away from Top of First Course of Masonry.

(3) The mortar bond was partially interrupted on top of the first course of masonry and the effectiveness of the mortar bond at the foundation level was impaired by the presence of an unbonded 2-in. concrete cap.

As previously noted, the flashing, which was placed at the leeward side of the wall covered the top surface of the first masonry course. This is well illustrated in figure 15. Note also that the top side of the first course units shows no sign of any mortar bonding. The geometry of the flashing was such that it interrupted the mortar bond on the leeward side of the wall. Different parts of the failure surface are shown in figures 16(a) to (d). It appears that the combination of the effects of the flashing and the 2 in. unbonded concrete cap on top of the foundation was to lower the tensile resistance of the mortar bond at the base of the wall below that normally expected even from an unreinforced masonry wall.

4.3 Estimated Resisting Moment at the Base of the Wall

The resisting moment of the wall to overturning over the leeward edge of the base of the masonry wall is attributable to two effects: the effect of the gravity load and the combined effect of the mortar bond and grouted dowels.

The effect of gravity loads is evaluated, assuming that the 2x4 vertical shores supporting the overhanging 12 in. wall were removed prior to the collapse. Since the outer edge of the mortar courses is somewhat inward from the face of the units and the wall cannot rotate without some crushing failure of the edge over which it rotates, it is assumed that the center of rotation is 1/4 in. inward from the outer edge of the 8-in. masonry units. The weight of the wall attributable to the masonry units alone was 38 psf for the 8 in. wall and 55 psf for the 12 in. wall. With an allowance for the grout the weight is estimated

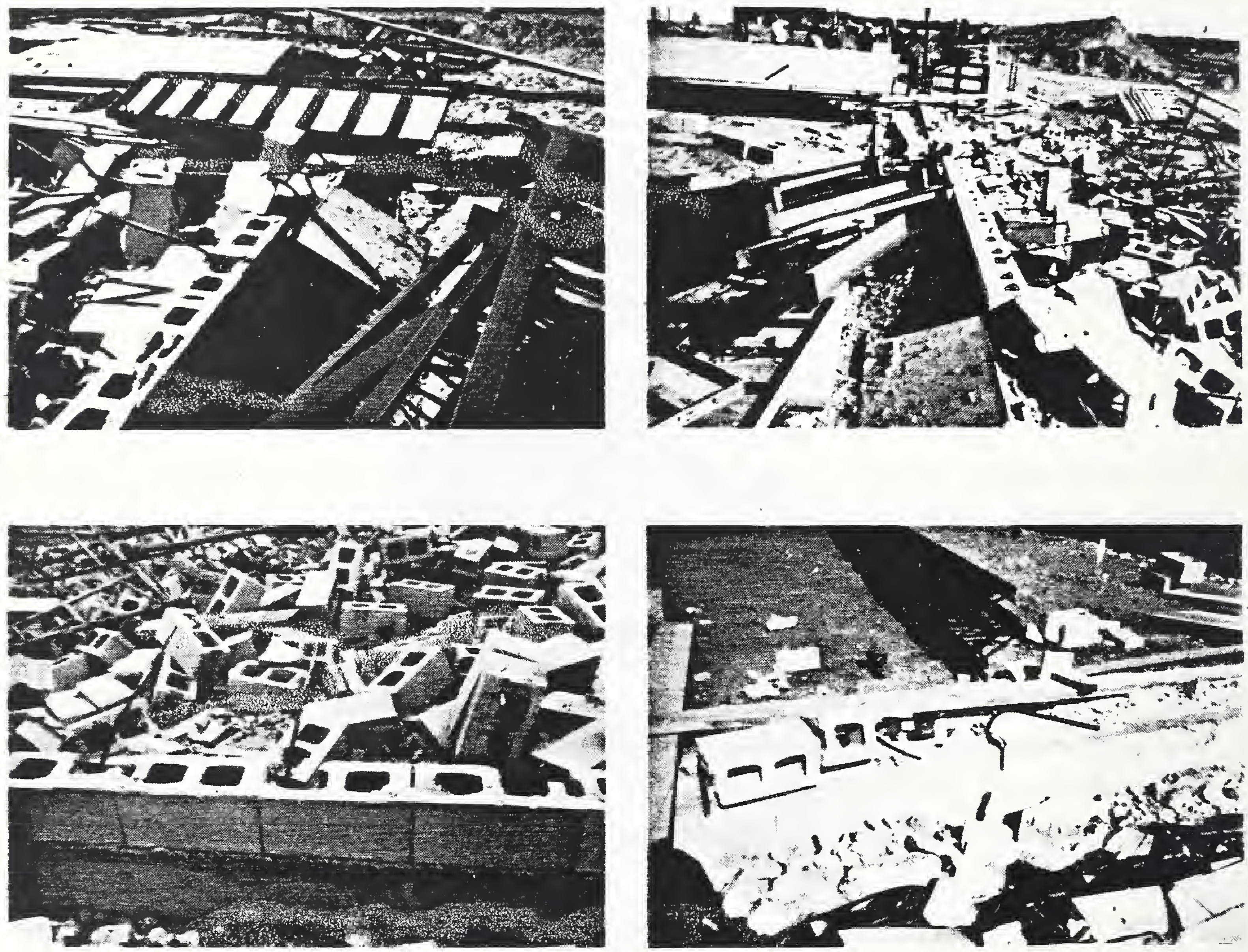


Figure 16: Failure Surface; Almost No Evidence of Effective Mortar Bonding Can Be Detected.

at 50 psf and 75 psf for the 8 in. and 12 in. walls, respectively. The center of gravity of the wall is calculated to be 2.44 inches horizontally from the leeward center of rotation and its position could vary $\pm 1/4$ in. due to construction tolerances. The resulting calculated resisting moment is approximately between the limits of 17,070 ft.-lb. and 20,980 ft.-lb. (with an allowance for the weight of the lintel). The upper bound of the calculated resisting moment is less than the 26,900 ft.-lb. estimated moment attributable to the wind load.

4.4 <u>Resisting Moment of Mortar and Grouted Dowels at the Base of the Wall</u>

In accordance with available records, the mortar and grout at the base of the wall at the time of collapse was more than one week old and should have achieved at least half its design strength. Had the wall been constructed in accordance with the plans and specifications, it is estimated that the resistance of the mortar bed, based on 23 psi tensile strength (the NCMA allowable tensile strength, which is estimated to be 1/2 the ultimate strength) and an approximate 1.5 in. width of the mortar beds would have been approximately 14,000 ft-lb.

Since the specified embedment of the dowels in both, the foundation and the grouted core was 2.5 ft, which is about twice the embedment depth required for the development of their full tensile strengths, it is reasonable to assume that at the time of the collapse the dowels could have resisted a moment of the order of 140,000 ft-lb which corresponds to their specified minimum yield strength of 40,000 psi. Thus the combined resisting moment attributable to the mortar bed and the grouted dowels would have been on the order of 154,000 ft-lb, which is more than five times the estimated maximum moment caused by the wind. This leads to the conclusion that the wall collapse would have been highly improbable, had the wall been constructed in accordance with the plans and specifications. As it was, the evidence indicates that the dowels were ineffective and the mortar bond was at best partially effective.

5. ANOMALIES IN THE CONSTRUCTION SEQUENCE THAT CONTRIBUTED TO THE RISK OF COLLAPSE

In addition to the deficiencies in the bracing and the as-built condition of the wall, there were two aspects of the construction sequence which contributed to the risk of collapse: the construction of the concrete masonry portion of a composite brick-block wall ahead of the brick portion; and the construction of the 60 ft wide and 23 ft high wall in advance of the steel columns on its ends and the abutting wall at its east end which could have provided lateral support. Had the brick and block wythes of the wall been built simultaneously, the base of the wall would have increased from 7 5/8 to 11 5/8 inches and the resisting moment to overturning provided by the gravity load would have more than doubled and probably prevented the collapse from happening. Likewise, had the wall been tied to the steel columns at its ends, and the steel columns secured in accordance with the plans, the collapse would probably not have occurred, even though the wall was inadequately braceed.

6. PROBABLE COLLAPSE MECHANISM

The estimated moment acting on the base of the wall as a result of the wind load was approximately 26,900 ft-lb. The leeward shores, as installed, had a load capacity which produced a moment at the base of the wall of less than 7,600 ft-lb, and would have failed if the shores alone had to resist the wind load.

The estimated moment resistance against overturning provided by the weight of the wall alone would have been approximately between 17,000 and 21,000 ft-lb, had the wall been constructed to accepted tolerances. The estimated moment resistance provided by the tensile strength of the mortar at the base of the wall would have been approximately 14,000 ft-lb, and that of the grouted dowels approximately 140,000 ft-lb, had the wall been constructed in accordance with plans and specifications. However, in the as-built wall, the moment resistance of the mortar was probably smaller than 14,000 ft-lb. and the dowels were ineffective.

The braces were very flexible and therefore contributed little to the load resistance when the wall initially began to tilt in the leeward direction.

Since the only sound reported by eye witnesses before the wall impacted on the ground was that of failing timber, it is unlikely that any significant mortar bond failure occurred during the collapse, since such a failure tends to be associated with a sharp, explosive noise. Thus, probably there was no effective mortar bond at the time of the failure.

Once the center of gravity of the wall moved approximately 2.5 inches in the leeward direction due to tilting of the wall, the resisting moment provided by the weight of the wall disappeared. At this latter tilt displacement the estimated force in the struts of the braces was 630 lb and the resisting moment of the bracing, which was very flexible, was 7,300 ft-lb. Thus at this tilt displacement, the resisting moment of the wall decreased from approximately 19,000 ft-lb or more at the onset of tilting to zero, while at the same time the resisting moment provided by the braces increased from zero to approximately 7,300 ft-lb. This indicates that if there was a wind gust strong enough to overcome the initial resisting moment provided by the dead weight of the wall (and possibly the mortar bond), the resisting moment decreased as the wall tilted in the leeward direction causing the wall to overturn. It should also be noted that, as the center of gravity of the wall moved beyond the leeward edge of its base, the weight of the wall itself produced an overturning moment. Thus even if the wind gust subsided at this point, the weight of the wall alone could have collapsed the braces.

The calculations presented indicate that, in the absence of effective dowel connections, the collapse of the wall under the prevailing wind conditions was a probable event.

7. COMPLIANCE OF BRACING WITH EXISTING PROVISIONS

The question also should be asked whether the shoring satisfied existing standards and regulations. The relevant regulations and standards are summarized below:

20 CFR 1926.700 (a) (Applicable OSHA regulation)

General.

All equipment and material used in concrete construction and masonry work shall meet the applicable requirements for design, construction, inspection, testing, maintenance and operation as prescribed in ANSI A 10.9-1970, Safety Requirements for Concrete Construction and Masonry Work.

ANSI A10.9-1970, Safety Requirements for Concrete Construction and Masonry Work:

12.5 Shoring and Bracing

Masonry walls shall be temporarily shored and braced until the designed lateral strength is reached, to prevent collapse due to wind or other forces.

ANSI A10.9-1983

11.5 Shoring and Bracing

Masonry walls shall be shored and/or braced until the designed lateral strength is reached, or the top supporting members are in place to prevent collapse due to wind or other forces. The support of bracing shall be designed by or under the supervision of a qualified person to withstand a minimum of 15 pounds per square foot. Braces or shores shall be secured in position.

ANSI A41.1 American Standard Code Requirements for Masonry

11.9 Precautions During Erection

11.9.1 Bracing to Resist Lateral Loads.

Masonry walls in locations where they may be exposed to high winds during erection shall not be built higher than 10 times the thickness, unless adequately braced or until provision is made for the prompt installation of permanent bracing at the floor or roof level immediately above the story under construction.

National Concrete Masonry Association Standard, April 1985:

4.8.1 ... Adequate precautions shall be taken to prevent damage to walls during erection by high winds or other causes. ...

ANSI A58.1-1982, Minimum Design Loads for Buildings and Other Structures

6.1.1 Wind Loads During Erection and Construction Phases.

Adequate temporary bracing shall be provided to resist wind loading on structural components and structural assemblages during the erection and construction phases

6.4.2.1 Minimum Design Wind Loading

The wind load used in the design of the main force resisting system for buildings and other structures shall be not less than 10 lbf/ft^2 multiplied by the area of the building or structure projected in a vertical plane normal to the wind direction.

In the calculation of design wind loads for components or cladding of buildings, the pressure difference between opposite faces shall be taken into consideration. The combined design pressure shall not be less than 10 lbf/ft^2 acting in either direction normal to the surface.

It can be seen that explicit provisions are provided in ANSI A10.9-1983, which requires bracing to resist a 15 psf horizontal pressure and in ANSI A58.1-1982 which requires a minimum design pressure of 10 psf. Other standards leave the determination of what is adequate shoring to the judgment of engineers.

Opinions by various experts on lateral pressures that temporary shoring of masonry walls should be designed to resist were presented at a June 17, 1986 public hearing on a draft revision

of 20 CFR Part 1926-"Concrete and Masonry Safety Standards", which was presented in a "Notice of proposed rulemaking" published in the Federal Register on September 18, 1985:
Section (a) (2) of the draft revision states that :" The lateral supports for masonry walls shall be capable to withstand a load of 15 pounds per square foot (73.2 kg/m^2) applied to the wall". The commentary notes that the 15 psf requirement is based on the ANSI A10.9-1983 safety requirements.

In the dicussion on the proposed revision, a professional engineer testified on behalf of the masonry industry committee and suggested to delete the 15 psf requirement "to avoid unnecessary economic penalties on masonry construction." He also stated that: " Average wind speeds in selected cities seldom exceeds 10 miles per hour for construction located in cities. If construction is in the suburbs, or on flat terrain, the average speed would increase to 10 to 15 mph. If we consider gusting effects, the wind might reach a speed of 35 miles per hour. This, however, would result in a horizontal pressure of approximately 5 psf, a situation taken into account by reputable contractors."

In the same hearing, a private consultant stated that: " While it may be true that lack of bracing rather than improper design of the bracing is the primary cause of wall collapses, at the same time I regard lateral loading of 15 psf as being a nominal requirement and not one that should be difficult to meet."

Another opinion was presented by a university professor: "... it is necessary to arrive at an acceptable lateral design load. It is not reasonable to prescribe loads corresponding to the 50 year recurrent wind speed applicable to permanent buildings for a wall standing temporarily unbraced for a fraction of a year. If a one quarter of a year long unbraced condition is acceptable, a 12.5 year recurrent wind speed would give the same annual probability of 0.02 as used for permanent buildings".

In the case of Pawtucket, RI, the fastest mile wind speed 30 ft above ground, with a 12.5 year mean recurrence interval would be approximately 66 mph, resulting in a design wind pressure of approximately 12 psf.

Thus it can be seen that required design pressures in standard provisions range from 10 psf in ANSI A58 to 15 psf in ANSI A10.9, and that experts who testified in the June 1986 public hearing, while not always supporting inclusion of specific design pressure requirements in standards, suggested that design lateral pressures ranging from 5 psf to 15 psf should be used.

It is also of interest to note, that data on the effects of wind on human activities imply that fastest mile winds with speeds somewhere between 35 and 50 mph, measured 6 ft above ground, would cause the work to be stopped (derived from data in reference 2, pp 454 and 455). These wind speeds are associated with pressures between approximately 6 and 12 psf. Winds with lower speeds would probably not cause an interruption of work and thus workers would be exposed to their effect.

The shoring that was actually in place could only withstand horizontal pressures on the order of 0.5 psf, and additionally lacked the stiffness needed to prevent initial tilting of the wall when the wind-induced moment exceeded the resisting moment provided by the wall itself. The shoring also did not satisfy the provisions of the "National Design Specifications for Wood Construction" (8) which limit the length-to-thickness ratio of timber compression members to 50 (no load capacity could be assigned to these struts in accepted engineering practice).

8. SUMMARY

(1) It is estimated on the basis of available metereological information that the wind load acting on the wall at the time of the collapse produced overturning moments at the base of the wall on the order of 26,900 ft-lb.

(2) From an examination of the evidence it is deduced that at the time of the collapse two braces supported the leeward side of the wall, one near the east end of the wall and one near the west end of the wall. Each of these braces consisted of two 16-ft long 2x10 planks, one vertical and one diagonal. It is estimated that the two braces as installed had the combined capacity of resisting an overturning moment at the base of the wall of less than 7,600 ft-lb. Thus if the wind load had to be resisted by the braces alone, the failure of the braces under the prevailing wind conditions was a probable event.

The braces were very flexible and were inadequately supported at the ground level, and therefore provided little support to the wall when tilting was initiated. The diagonal struts of the braces were installed at a steep angle to the horizontal, creating the risk of failure of the braces, caused by upward sliding of the restraining vertical shoring plank.

(3) The collapse occurred as a result of the combined effect of inadequate bracing, ineffective dowel connections to the foundation, reduced effectiveness of mortar bonding near the base of the wall resulting from vinyl flashing which covered part of the top surface of the bottom course of masonry units and an unbonded 2 in. concrete cap on the foundation, and anomalies in the construction sequence which increased the risk of collapse. The collapse was a probable event under the prevailing wind conditions.

(4) Existing regulations and standards require bracing of masonry walls during construction against the effects of high winds. Most of the standards leave the decision of what is adequate bracing to engineers, however ANSI A10.9-1983 requires bracing to be designed for uniform lateral pressures of 15 psf and ANSI A58.1-1982 stipulates minimum lateral pressures of 10 psf. Experts testifying in a June 1986 public hearing on a draft revision of the applicable OSHA regulations, while not necessarily favoring inclusion of explicit values for design lateral pressures in the regulations, suggested design lateral pressures ranging from 5 to 15 psf. Work at construction sites would probably proceed at wind speeds causing lateral pressures of less than 6 psf.

It is deduced from available information, that the bracing that supported the leeward side of the wall at the time of the collapse had the capacity to support an ultimate horizontal pressure of not more than 0.5 psf, acting normal to the surface of the wall.

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11. ABSTRACT (A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here)

Results from a study to determine the cause of the October 28, 1985 collapse of a masonry wall under construction in Pawtucket, RI are presented. The wall was a 60 ft-3 in. long, 23 ft-6 in. high partially reinforced concrete masonry wall supported by wooden braces. The study included: inspection of construction plans and specifications; review of construction records and eyewitness accounts recorded immediately after the collapse, as well as testimony from OSHA inspectors and local building officials who visited the site a short time after the collapse; examination of photographs taken by OSHA inspectors and police investigators; analysis of meteorological data; and a stability analysis of the collapsed wall. It is concluded that the collapse was probably caused by a gust of wind which exerted lateral forces which exceeded the lateral-load capacity of the wall and its supporting wooden braces. Contributing factors were the lack of grout in the masonry cores which contained the steel reinforcement dowels and the inadequate anchoring of the dowels in the foundation.

12. KEY WORDS (Six to twelve entries; alphabetical order; capitalize only proper names; and separate key words by semicolons)

building; construction loads; construction safety; lateral bracing; masonry construction; stability; structural collapse; timber shoring; wind loads.

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