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Masonry Wall Collapse

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Table of Contents

Discussion 4

Collapse 17

Inspection 18

Structural Analysis and Discussion 21

Conclusion 24

LIST OF FIGURES

Figure 1	CMU wall Plan
Figure 2	CMU wall elevations
Figure 3	CMU wall section
Figure 4	Exterior wall section
Figure 5	Control Joint detail
Figure 6	Typical interior bracing detail
Figure 7	Schematic CMU wall plan showing bracing location at East Wall
Figure 8	Section showing interior as built bracing at east wall
Figure 9	Top end of brace without positive connection
Figure 10	Two CMU blocks on top of plank near rebar
Figure 11	Brace spacing requirement for masonry wall between control joints
Figure 12	General Notes
Figure 13	Quality assurance plan as required by construction documents
Figure 14	General Notes
Figure 15	Quality assurance plan as required by construction documents
Figure 16	Collapsed wall
Figure 17	Collapsed wall
Figure 18	Collapsed wall
Figure 19	Collapsed wall
Figure 20	Collapsed wall
Figure 21	Collapsed wall

REPORT

The Division of Occupational Safety and Health of the State of Tennessee requested the Directorate of Construction, OSHA National Office, to provide assistance in the investigation and causal determination of the April 18, 2013 collapse of a masonry wall during construction of the Goodwill Retail Store in Hendersonville, TN. As a result of the wall collapse, two employees were killed and one was injured. Our investigation and evaluation were based on the information provided by the Division of Occupational Safety and Health of the State of Tennessee. Please note that we did not visit the incident site.

Discussion

The project consisted of construction of a one-story Goodwill Retail Store, approximately 170' wide x 180' long (see figure 1).

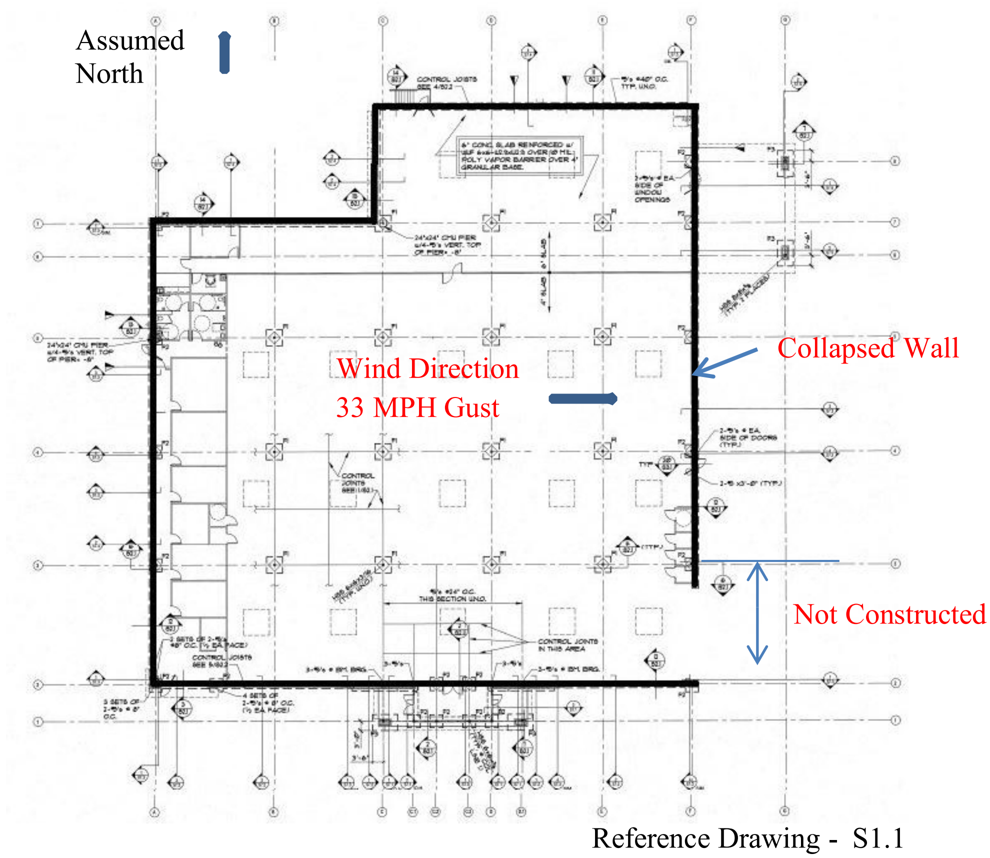
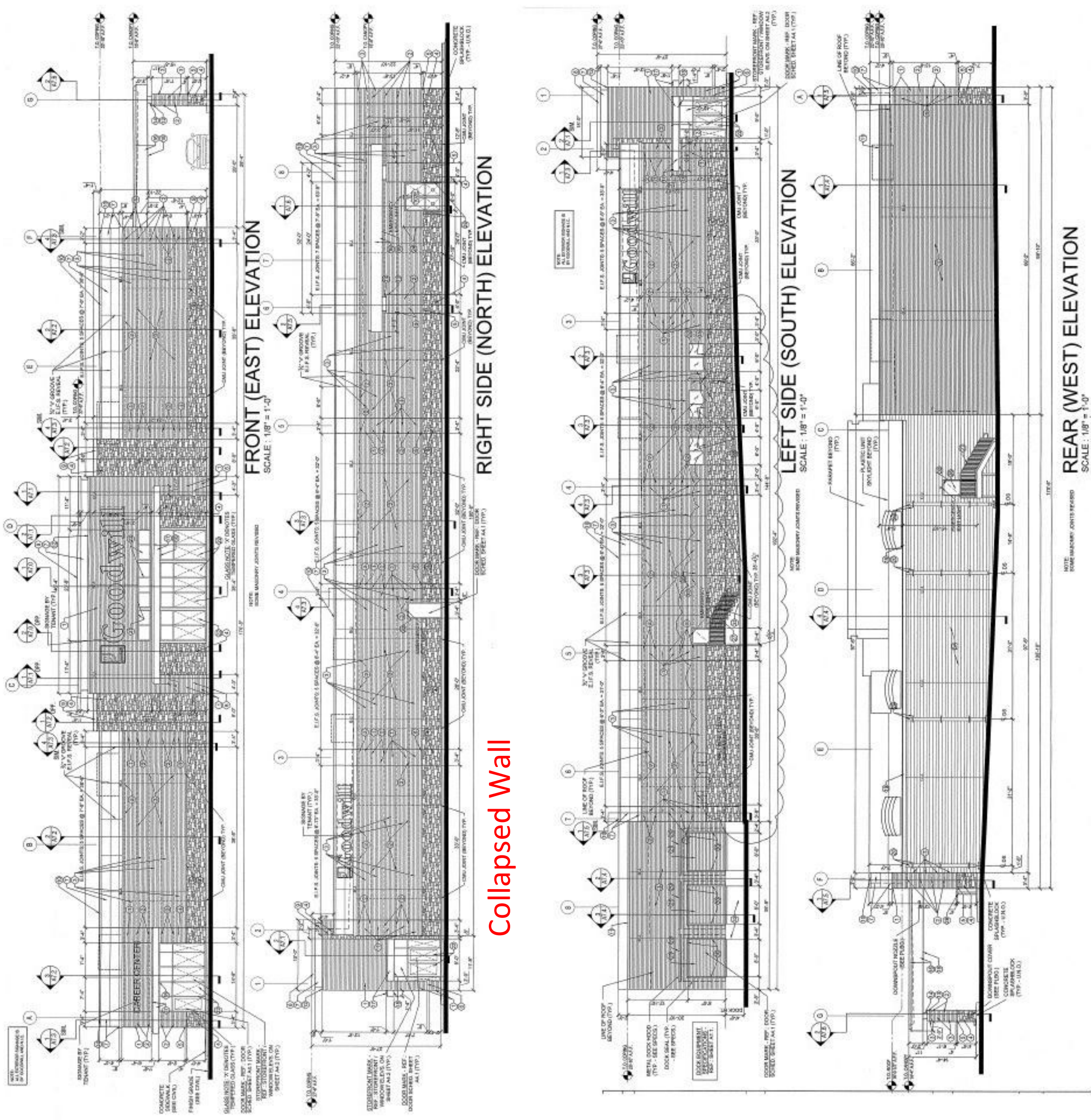


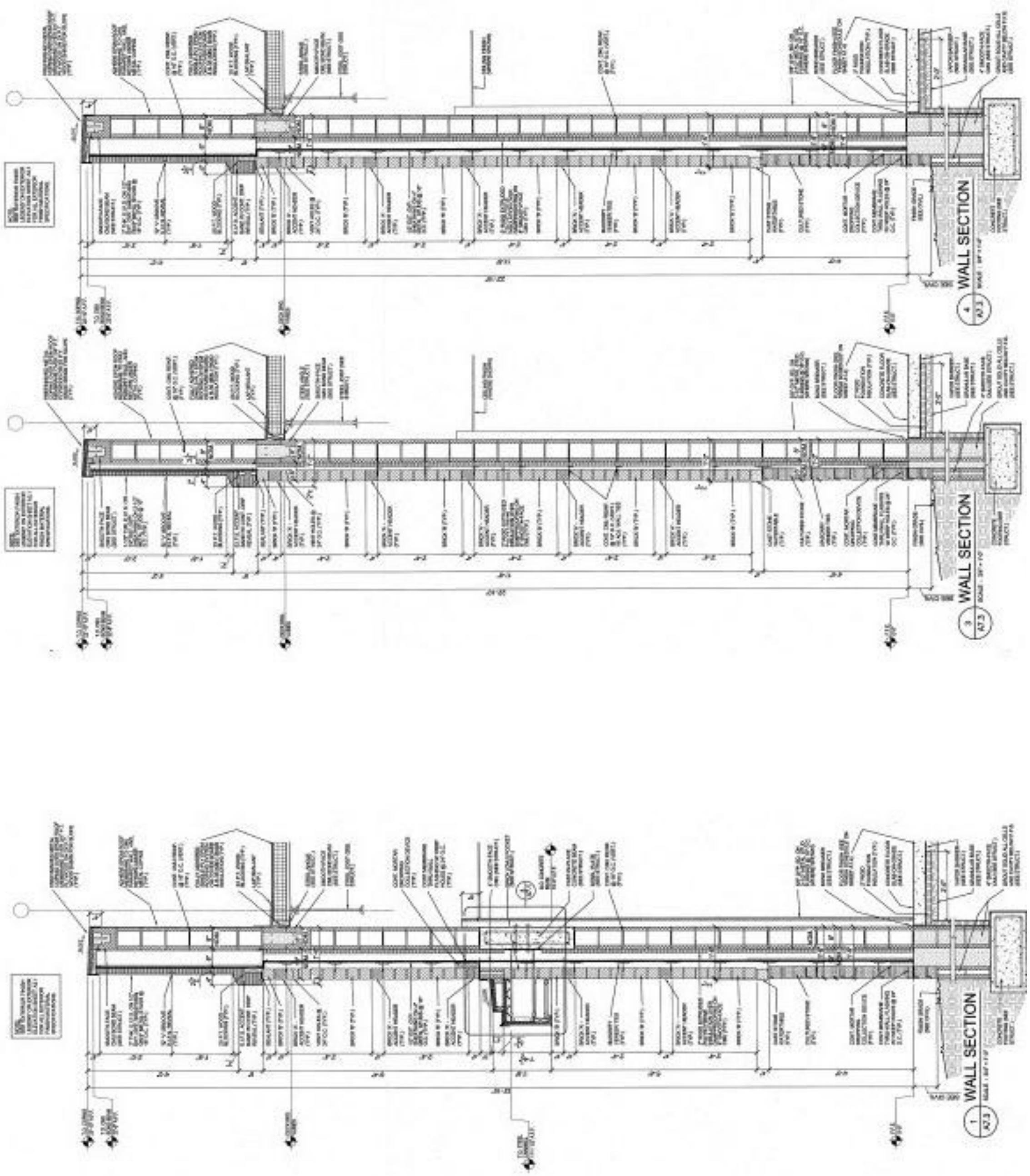
Figure 1 – CMU wall Plan

For the purpose of this report, the wall that fell is identified as the east wall by the field personnel, although contract drawings identify it as the north wall. The exterior non-load bearing walls consisted of 8" thick partially grouted Concrete Masonry Units (CMU) supported on 2' wide x 1' deep concrete footing (see Figures 1 to 3).



Reference drawing – A5.0

Figure 2 – CMU wall elevations



Reference Drawing - A7.3

Figure 3 – CMU wall section

The exterior walls were part of the lateral load-resisting system and would have acted as shear walls when the building was completed. The ground floor consisted of 4"/6" thick concrete slab on grade.

The roof was to consist of metal deck 1 ½" deep x 22 gage supported on 24"/26" deep steel joists spanning approximately 35' in the north-south direction. The steel joists were designed to be resting on 32" deep steel joists girders spanning in the east-west direction for a span of approximately 34 feet. The steel joists were to be supported on square hollow steel section columns spaced approximately at 34' on center.

The one-story building is owned by Goodwill Industries of Middle Tennessee, Inc. The following were key participants of the project:

Owner:	Goodwill Industries of Middle Tennessee, Inc.
Architect:	H. Michael Hindman Architects (HMHA), P.C. of Brentwood, TN
Structural Engineer:	EMC Structural Engineers (EMC), P.C. of Nashville, TN
General Contractor:	Solomon Builders, Inc. of Nashville, TN
Masonry Contractor:	Shannon Tayes dba Tayes Masonry of Smithville, TN
Structural testing & insp.:	Beaver Engineering,Inc., of Nashville, TN

The construction for the project began in early March 2013. The concrete footing 2' wide x 1' deep for the CMU wall was poured approximately one week before the CMU wall construction began. For the beginning and completion dates of the CMU wall construction see table below.

TABLE 1

Beginning and completion dates for CMU wall construction

8" thick CMU wall x 24' high	Beginning of construction	Completion of construction
West wall	March 18, 2013	March 26, 2013
East wall (architect referred as north wall)	March 27, 2013	April 3, 2013
North wall	April 2, 2013	April 5, 2013
South wall	April 4, 2013	April 12, 2013
Loading dock wall	April 16, 2013	April 22, 2013

The CMU walls consisted of hollow concrete blocks and were partially grouted using a low lift grouting method. The wall was reinforced with #5 rebars at 40" on center (see Figure 4).

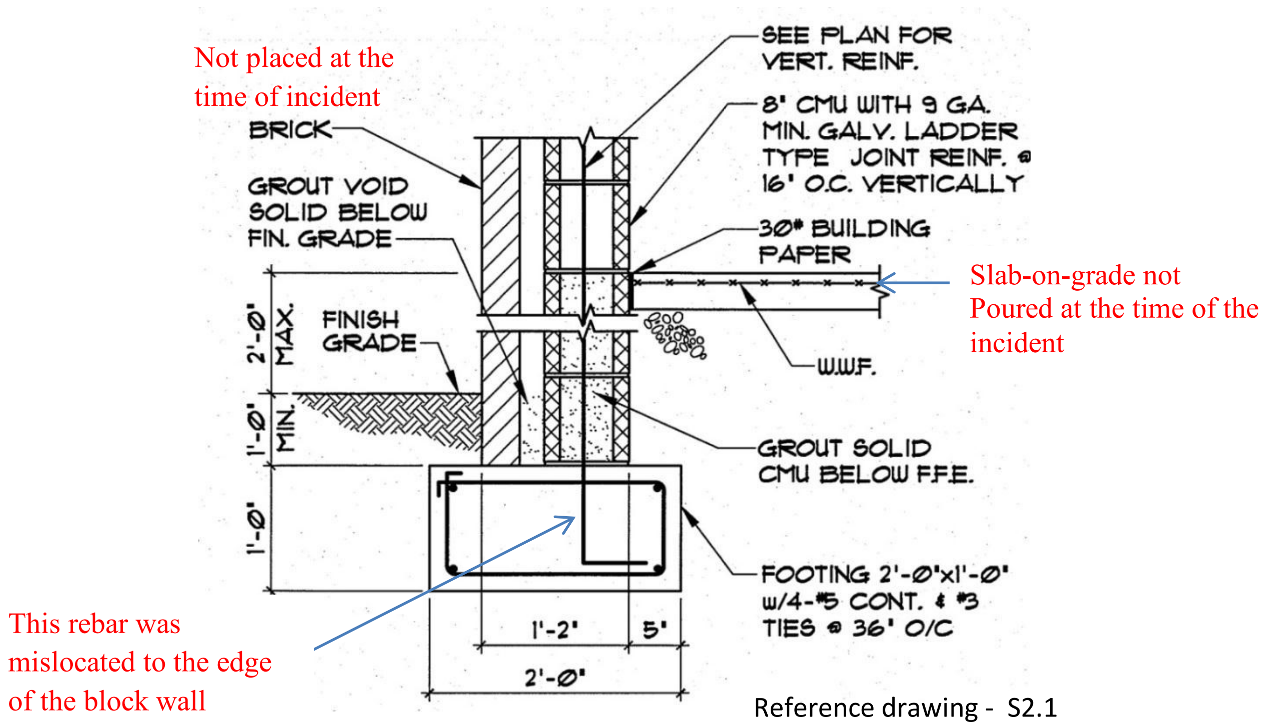


Figure 4 – Exterior Wall Section

The architect specified the maximum spacing of the control joints to be at 25' on center in the horizontal direction (see Figure 5).

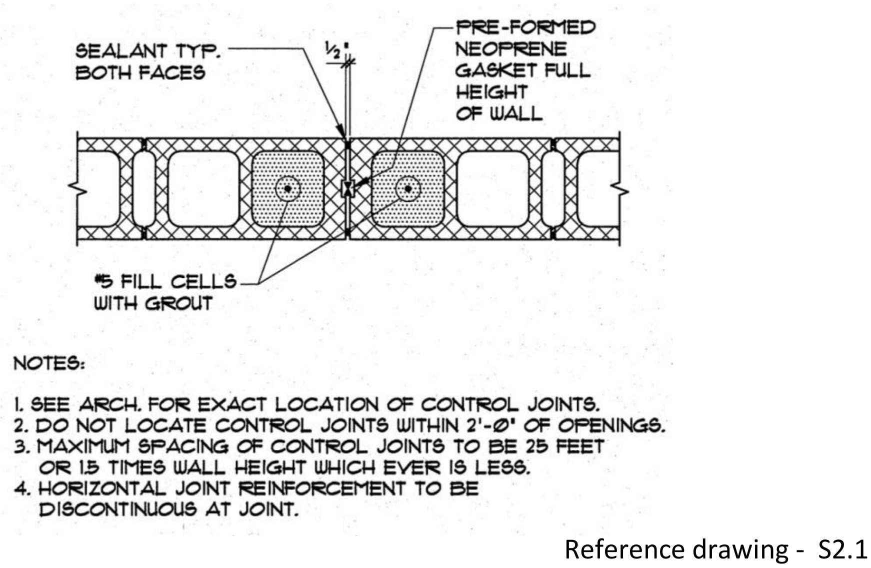


Figure 5 – Control Joint detail

All cells of the masonry blocks from the top of the footing to the first floor level were fully grouted.

Bond beams were used at window openings, roof level and at the top of the parapet. The bond beams at the top of the masonry wall were reported to be discontinuous at the control joints. The ground floor slab, also known as the first floor slab, was not poured at the time of the incident. During construction, the masonry contractor had provided six bracings against wind for the entire length of each wall. Three braces were located at the interior (similar to Figure 6) and three on the exterior faces of each wall.

Figure 6 – Typical interior bracing detail

A few days prior to the collapse, all three exterior braces on the north, south and west walls were removed but on the east wall only two exterior braces were removed. The middle exterior brace was left intact on the east wall that fell (see figure 7).

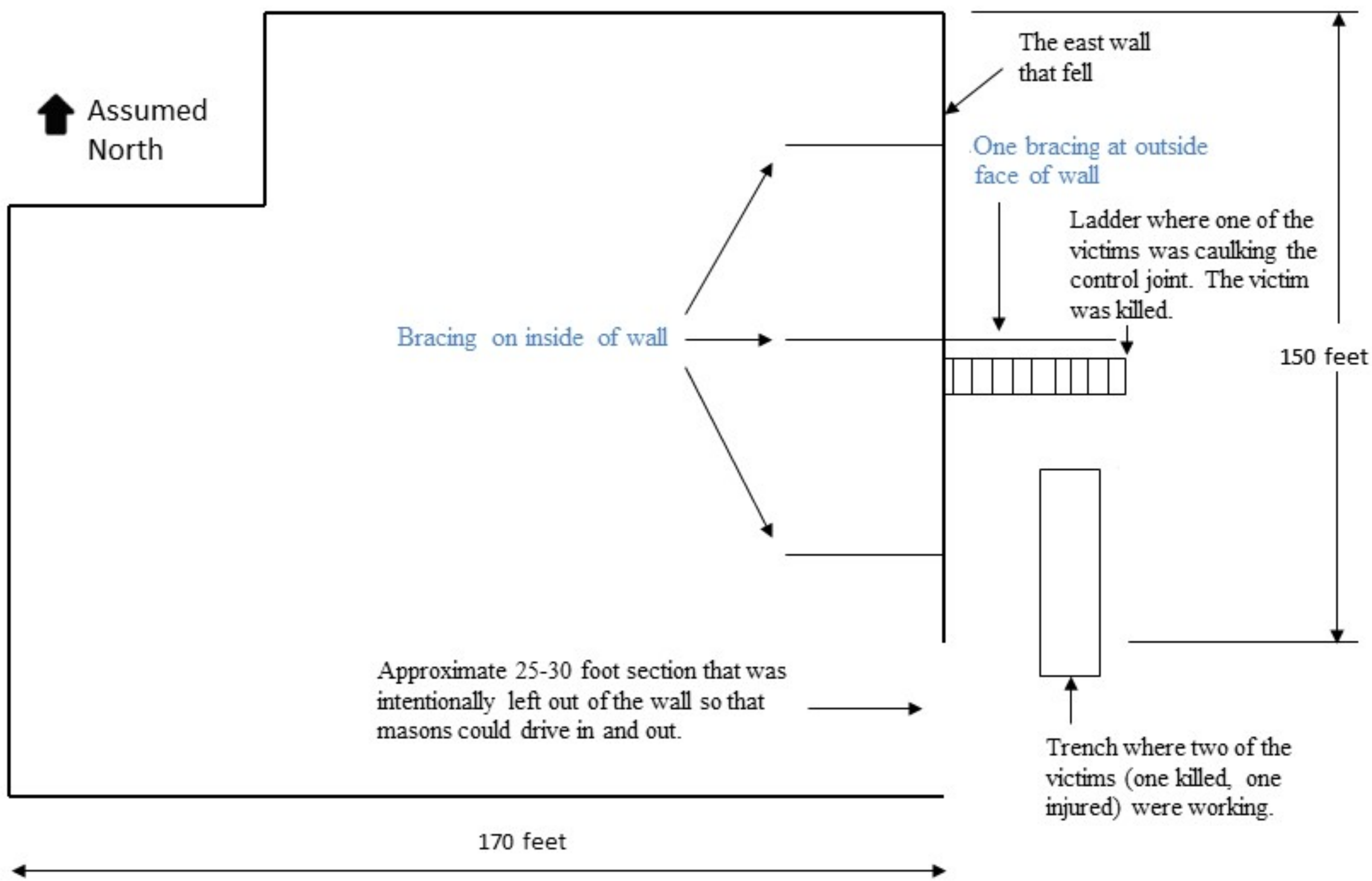


Figure 7 _ Schematic CMU wall plan showing bracing location at east wall

For the installation of the bracing, the contractor installed 2x10 vertical members abutting to the face of the walls (see Figure 8).

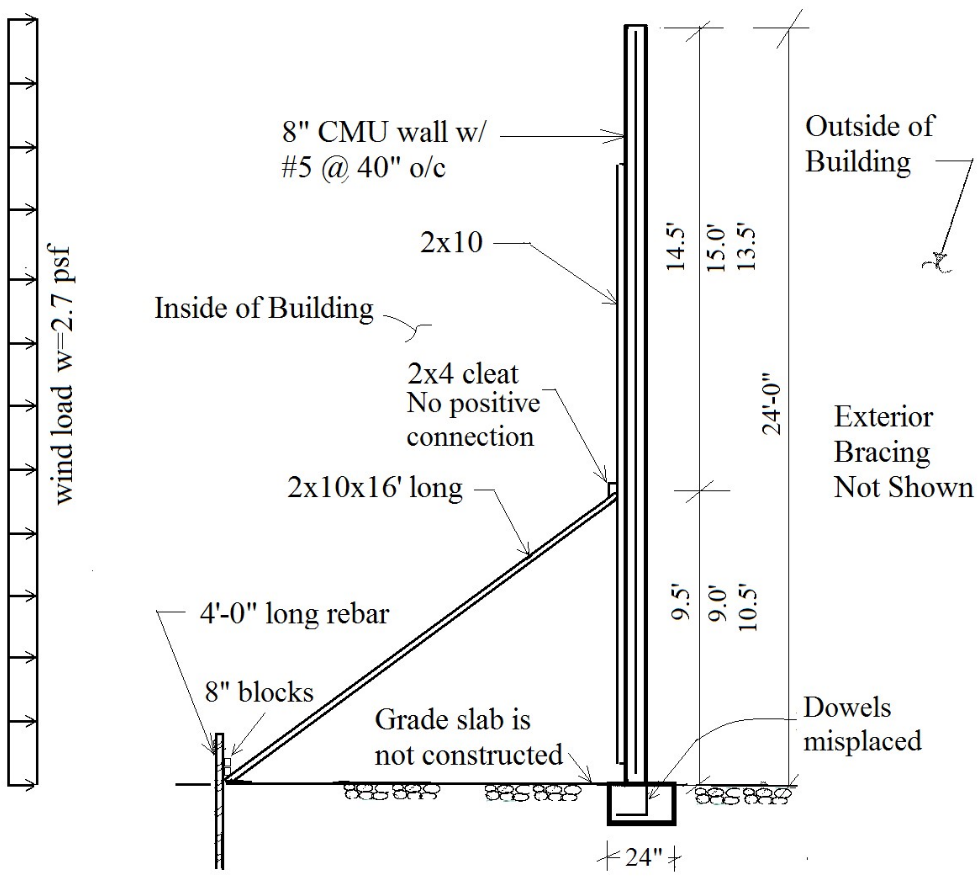


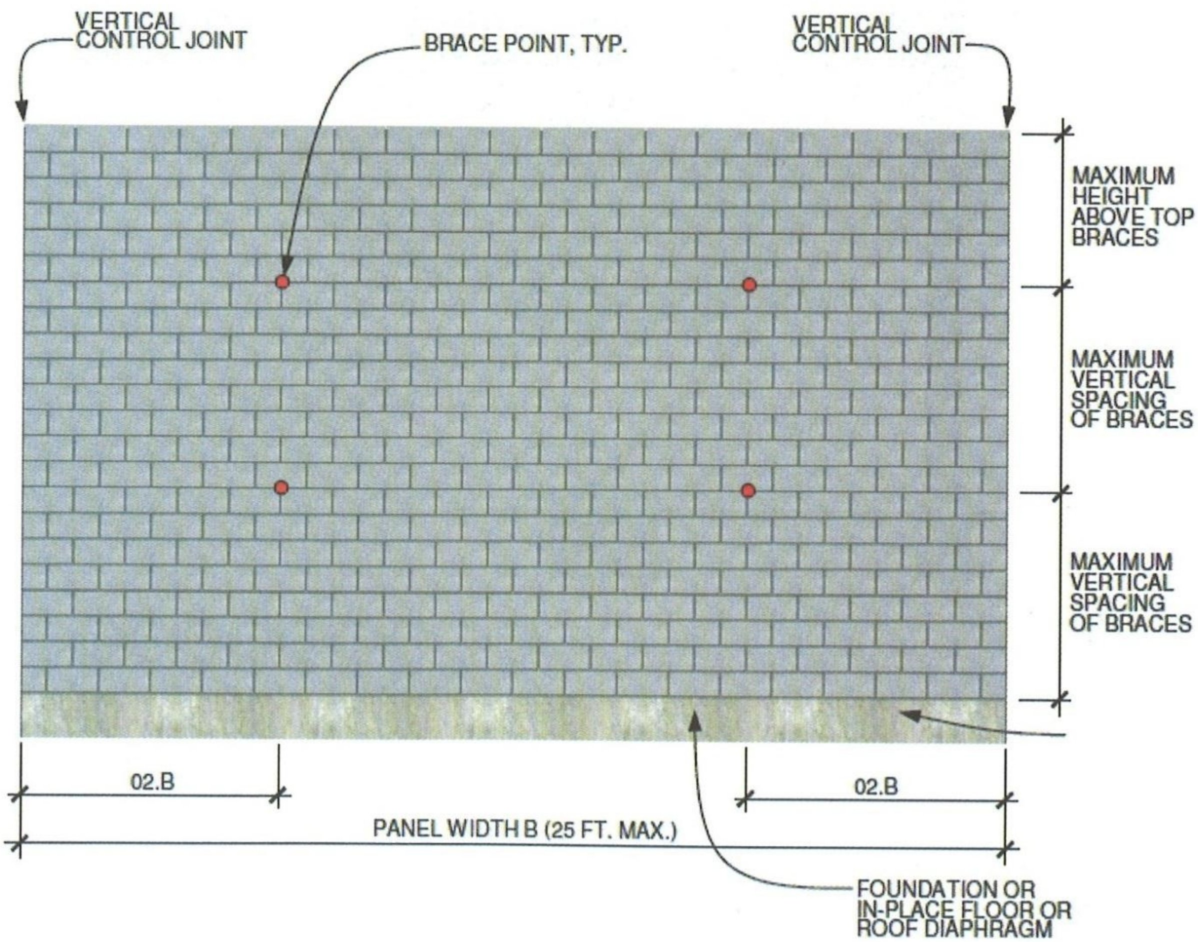
Figure 8 – Section showing interior as-built bracing at east wall

At mid-height of the vertical member, approximately 10' above, 2x4 horizontal cleats were provided. The 2x4 cleats were nailed to vertical members (see Figure 9). The diagonal bracing member consisted of 2x10 Southern Yellow Pine (SYP) OSHA scaffold plank, the top end of which was held underneath the cleats while the opposite end was held against 4' ($\pm$) long rebar. The rebar was embedded into the soil for a depth of approximately 2'-6" and projecting out around 1'-6". On top of the plank near the rebar, two CMU blocks were placed (see Figure 10).

Figure 9 - Top end of brace without positive connection

Figure 10 - Two CMU blocks on top of plank near rebar

The design of the bracing members was not performed by any contractor or by an engineer. Bracings were installed randomly based on the contractor's judgment. "Standard practice for Bracing Masonry Walls under construction," developed by the Council for Masonry Wall Bracing, was not followed (see Figure 11).



References:

1. Copyright by the Mason Contractors Association of America
2. "Standard practice for Bracing Masonry Walls Under Construction" developed by Council for Masonry Wall Bracing

Note: Industry practice required two braces between control joints

Figure 11 - Brace spacing requirement for masonry wall between control joints

The top of the wall remained as a free end, as roof framing and roof diaphragm were yet to be constructed.

The contract documents required that the owner employ an independent testing company to perform site inspections and testing in accord with the quality assurance plan. The testing company was to retain a licensed structural engineer or an architect to perform periodic visual observations of the structure during construction for general conformance to the design drawings. The inspector was required to be an individual certified or experienced to perform such inspections (see Figures 12 to 15).

DESIGN AND CODE INFORMATION

1. ALL CONSTRUCTION SHALL CONFORM TO THE INTERNATIONAL BUILDING CODE, 2006 EDITION.
2. VERIFY EXISTING CONDITIONS AND ALL DIMENSIONS AND NOTIFY ARCHITECT OF ANY CONDITIONS WHICH CONFLICT WITH OTHER PLANS AND SPECIFICATIONS. STRUCTURAL DRAWINGS MUST BE COORDINATED WITH ARCHITECTURAL DRAWINGS. STRUCTURAL DRAWINGS ARE NOT INTENDED FOR BUILDING LAYOUT.
3. SHOP DRAWINGS WILL NOT BE REVIEWED BY THE DESIGNER UNTIL AFTER THE GENERAL CONTRACTOR HAS THOROUGHLY REVIEWED THE SHOP DRAWINGS, VERIFIED EXISTING CONDITIONS, AND COORDINATED THE SHOP DRAWINGS WITH OTHER AFFECTED TRADES. SUBMIT FOUR COPIES OF REVIEWED DRAWINGS FOR ENGINEER'S REVIEW. ONLY THREE SETS OF MARKED UP SHOP DRAWINGS SHALL BE RETURNED BY THE DESIGNER. REPRODUCTION OF STRUCTURAL DRAWINGS FOR SHOP DRAWINGS IS NOT PERMITTED.
4. THE STRUCTURE IS UNSTABLE UNTIL ALL LOAD BEARING WALLS ARE ERECTED AND STEEL MEMBERS ARE ERECTED, CONNECTIONS ARE COMPLETELY BOLTED AND/OR WELDED AND INSPECTED, THE STEEL DECK ATTACHED TO THE STEEL FRAMING, AND THE CONCRETE FLOORS PLACED AND ATTAINS 75% OF 28-DAY STRENGTH. UNTIL SUCH TIME, TEMPORARY BRACING IS REQUIRED. THE DESIGN ADEQUACY OF TEMPORARY BRACING AND SHORING IS THE SOLE RESPONSIBILITY OF THE CONTRACTOR.
5. DO NOT SCALE STRUCTURAL DRAWINGS, AND FOR LOCATION OF MISCELLANEOUS ITEMS (OPENINGS, BENT PLATES, INSERTS, ETC.) AFFECTING STRUCTURAL WORK, SEE ARCHITECTURAL, MECHANICAL, PLUMBING AND ELECTRICAL DRAWINGS.
6. LIVE LOADS:
   ROOFS: 20 PSF
7. ROOF LOADS:
   GROUND SNOW LOAD: 10 PSF
   SNOW EXPOSURE Ce: .9
   SNOW IMPORTANCE I: 1.0
   THERMAL FACTOR Ct: 1.0
   FLAT ROOF SNOW LOAD: 12 PSF
8. WIND LOADS:
   BASIC WIND SPEED: 90 MPH
   WIND IMPORTANCE I: 1.0
   WIND EXPOSURE FACTOR: B
   INTERNAL PRESSURE COEFFICIENT: .18
   CLADDING LOAD: 25 PSF
9. SEISMIC LOADS:
   SEISMIC USE GROUP: I
   SEISMIC IMPORTANCE Ie: 1.0
   .2 SEC SPECTRAL RESPONSE ACCELERATION Ss: .313
   1.0 SEC SPECTRAL RESPONSE ADDELERATION S1: .145
   SITE CLASS: B
   DESIGN SPECTRAL RESPONSE SDS: .209
   DESIGN SPECTRAL RESPONSE SD1: .096
   SEISMIC DESIGN CATEGORY: B
   RESISTING SYSTEM: INTERMEADIATE MASONRY SHEAR WALLS
   RESPONSE MODIFICATION FACTOR R: 3.5
   SEISMIC RESPONSE COEFFICIENT Cs: .06
   ANALYSIS PROCEDURE: EQUIVALENT LATERAL FORCE
   BASE SHEAR: 72 KIPS
10. SPECIAL LOADS FOR ITEMS TO BE DESIGNED BY OTHERS:
   STAIRS: 100 PSF
   HAND RAILS: 50 PLF
   VEHICLE BARRIERS: 6,000 POUNDS

SPECIAL INSPECTIONS AND TESTING

1. THE CONTRACTOR/OWNER SHALL EMPLOY AN INDEPENDENT TESTING COMPANY TO PERFORM SITE INSPECTIONS AND TESTING IN ACCORDANCE WITH THE QUALITY ASSURANCE PLAN SHEET S6.2.

STRUCTURAL OBSERVATIONS

1. THE CONTRACTOR/OWNER SHALL EMPLOY A LICENSED STRUCTURAL ENGINEER OR ARCHITECT TO PERFORM PERIODIC VISUAL OBSERVATIONS OF THE STRUCTURE DURING CONSTRUCTION FOR GENERAL CONFORMANCE TO THE DESIGN DRAWINGS.

FOUNDATION NOTES

1. FOUNDATION DESIGN IS BASED ON A REPORT MADE BY BEAVER ENGINEERING, INC. DATED MAY 17, 2012 (REPORT NO. 12-6407).
2. INDIVIDUAL FOOTINGS ARE DESIGNED TO BEAR ON UNIFORM SOIL CAPABLE OF SUPPORTING 3000 PSF. CONTINUOUS FOOTINGS ARE DESIGNED TO BEAR ON SOIL CAPABLE OF SUPPORTING 3000 PSF. DESIGN ASSUMES DIFFERENTIAL AND TOTAL SETTLEMENT ARE WITHIN ACCEPTED TOLERANCES FOR THE TYPE OF CONSTRUCTION USED.
3. THE SOIL BEARING CAPACITY AND CONSISTENCY SHALL BE VERIFIED FOR THE BUILDING LIMITS BY A REGISTERED GEOTECHNICAL ENGINEER WHEN FOUNDATION EXCAVATIONS HAVE BEEN CARRIED DOWN TO THE PROPOSED ELEVATIONS. THE BOTTOM OF ALL EXTERIOR FOOTINGS SHALL BE 2'-0" MINIMUM BELOW FINISHED GRADE.
4. WHERE FOOTING EXCAVATIONS ARE TO REMAIN OPEN AND MAY BE EXPOSED TO RAINFALL, THE EXCAVATIONS SHALL BE UNDERCUT AND A 3 INCH THICK MUD MAT OF 2000 PSI CONCRETE SHALL BE PLACED IN THE BOTTOM TO PROTECT THE BEARING SOILS.
5. WHERE FOOTING STEPS ARE NECESSARY, THEY SHALL BE NO STEEPER THAN 1 VERTICAL TO 2 HORIZONTAL, UNLESS SHOWN OTHERWISE ON PLANS.

REINFORCED CONCRETE

1. ALL CONCRETE WORK SHALL CONFORM TO THE 'BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE,' (ACI 318-05).
2. REINFORCING STEEL SHALL BE DEFORMED BARS ASTM A-615 (GRADE 60).
3. THE COMPRESSIVE STRENGTH AT 28 DAYS OF ALL CAST IN PLACE CONCRETE SHALL BE:
   4000 PSI - SLABS-ON-GRADE
   4000 PSI - BEAMS
   3000 PSI - ALL OTHER CONCRETE
   (SEE CIVIL DRAWINGS FOR SITE CONCRETE STRENGTH REQUIREMENTS).
4. LAP SPLICES FOR REINFORCING BARS SHALL BE CLASS B IN ACCORDANCE WITH ACI 318-05, UNLESS NOTED OTHERWISE.
5. CLEAR CONCRETE COVER FOR REINFORCING STEEL:
   WALLS: 2" EXTERIOR FACES; 3/4" INTERIOR FACES
   MASONRY WALLS: LOCATE IN CENTER OF WALL (U.N.O.)
   BEAMS AND COLUMNS: 1-1/2" FORMED EDGES
   FOOTINGS: 2" FORMED EDGES; 3" CAST AGAINST GROUND
6. THE LONGITUDINAL REINFORCING STEEL IN BOND BEAMS, WALLS, AND FOOTINGS SHALL BE CONTINUOUS AROUND CORNERS. SEE TYPICAL DETAILS.
7. MECHANICAL VIBRATORS SHALL VIBRATE ALL CONCRETE.
8. UNLESS OTHERWISE DIRECTED BY THE OWNER, CONCRETE SLABS SHALL BE FINISHED TO THE FOLLOWING FLATNESS CRITERIA:
   SPECIFIED OVERALL F NUMBERS
   FLATNESS FF = 35
   LEVEL FL = 25
   MINIMUM LOCAL F NUMBERS
   FLATNESS FF = 24
   LEVEL FL = 17
9. COORDINATE ALL VAPOR RETARDERS, VAPOR BARRIERS, AND WATERPROOFING OF CONCRETE SLABS-ON-GRADE AND CONCRETE WALLS WITH FINISH MATERIAL REQUIREMENTS AND ARCHITECTURAL SPECIFICATIONS.
10. THE CONCRETE FILL ON COMPOSITE DECK SHALL BE LIGHTWEIGHT STRUCTURAL CONCRETE (107-113 PCF) WITH 4% TO 7% ENTRAINED AIR AND DEVELOP A MINIMUM COMPRESSIVE STRENGTH OF 3000 PSI (f'c) IN 28 DAYS.

CONCRETE MASONRY

1. MASONRY WALL CONTROL JOINTS SHALL BE LOCATED AS SHOWN ON THE ARCHITECTURAL DRAWINGS.
2. CONCRETE MASONRY SHALL CONFORM TO THE NATIONAL CONCRETE MASONRY ASSOCIATION SPECIFICATIONS, AND HAVE A DENSITY OF 125 PCF AND SHALL HAVE A MINIMUM PRISM STRENGTH (F'M) OF 1500 PSI.
3. GROUT FOR FILLING CONCRETE MASONRY CELLS SHALL CONFORM TO STANDARD SPECIFICATIONS FOR 'MORTAR AND GROUT FOR REINFORCED MASONRY,' ASTM C-476, AND SHALL HAVE A COMPRESSIVE PRISM STRENGTH (F'M) OF 3000 PSI AT 28 DAYS. THE SLUMP SHALL BE BETWEEN 9 INCHES AND 11 INCHES. WHERE THE MINIMUM DIMENSION OF ANY CONTINUOUS VERTICAL CELL IS 3 INCHES OR LESS, USE FINE GROUT, OTHERWISE USE COARSE (PEA GRAVEL) GROUT.
4. MORTAR FOR CONCRETE MASONRY SHALL BE TYPE 'S' AND SHALL CONFORM TO ASTM C-270.
5. MASONRY CONSTRUCTION SHALL BE BUILT IN LIFTS NOT TO EXCEED 4 FEET PRIOR TO GROUTING CORES. KEY NEXT GROUT LIFT INTO PRIOR LIFT BY STOPPING FIRST LIFT 2" BELOW TOP OF BLOCK.
6. ALL REINFORCING BARS IN FILLED CELLS SHALL BE DOWELED INTO FOOTINGS WITH STANDARD 90-DEGREE HOOKS AND DOWELED 7 INCHES INTO BOND BEAMS AT TOP OF WALLS.
7. MASONRY LAP SPLICES SHALL BE 48 BAR DIAMETERS (U.N.O.)
8. REINFORCEMENT IN WALLS SHALL BE PLACED IN THE CENTER OF THE WALL UNLESS NOTED OTHERWISE.

STRUCTURAL STEEL

1. ALL STRUCTURAL STEEL WORK SHALL CONFORM TO THE AISC 'MANUAL OF STEEL CONSTRUCTION ALLOWABLE STRESS DESIGN' THIRTEENTH EDITION.
2. STRUCTURAL STEEL ROLLED SHAPES SHALL BE ASTM A-992 GRADE 50 UNLESS NOTED OTHERWISE. STRUCTURAL STEEL PLATES AND ANGLES SHALL BE ASTM A-36.
3. STRUCTURAL PIPE COLUMNS SHALL BE ASTM A-53, TYPE E OR S, GRADE B. STRUCTURAL TUBES SHALL BE ASTM A500, GRADE B.
4. STEEL FRAMING CONNECTIONS SHALL BE BOLTED OR WELDED. BOLTS SHALL BE 3/4 INCH DIAMETER MINIMUM AND SHALL BE ASTM A-325-N, UNLESS NOTED OTHERWISE.
5. USE DIRECT TENSION INDICATORS AND HARDENED WASHERS WITH ALL HIGH STRENGTH BOLTS OR USE LOAD INDICATOR BOLTS.
6. STEEL JOISTS SHALL BE DESIGNED, FABRICATED AND ERECTED IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS OF THE STEEL JOIST INSTITUTE, LATEST EDITION. STEEL JOISTS SHALL BE GRADE 50 STEEL.
7. METAL DECK SHALL BE INSTALLED IN ACCORDANCE WITH THE STEEL DECK INSTITUTE SPECIFICATIONS, LATEST EDITION.
8. WELD WASHERS SHALL BE USED WITH METAL DECK THINNER THAN 22 GAUGE.
9. ANCHOR BOLTS SHALL BE ASTM A-307 HEADED BOLTS. MINIMUM ANCHOR BOLT EMBEDMENT SHALL BE 12 BOLT DIAMETERS UNLESS NOTED OTHERWISE. CLEAN ANCHOR BOLTS OF ALL GREASE, DIRT, ETC., BEFORE INSTALLATION.
10. FRAMED BEAM CONNECTIONS SHALL DE DESIGNED BY A QUALIFIED PROFESSIONAL ENGINEER EMPLOYED BY THE FABRICATOR TO DEVELOP THE REACTION SHOWN FOR THE ENDS OF BEAMS ON STRUCTURAL PLANS. IN NO CASE SHALL THE LENGTH OF THE FRAMED CONNECTION BE LESS THAN 1/2 THE 'T' DIMENSION OF THE BEAM WEB. WHERE REACTIONS ARE NOT SHOWN, THE CONNECTION SHALL DEVELOP ONE HALF THE ALLOWABLE UNIFORM LOAD FOR LATERALLY SUPPORTED BEAMS AS SHOWN IN PART 2 OF THE AISC MANUAL.
11. WELDS SHOWN ON THE STRUCTURAL DRAWINGS ARE THE MINIMUM REQUIRED BY DESIGN. THE FABRICATOR'S DRAWINGS SHALL SHOW WELDS AND THEY SHALL CONFORM TO AWS SPECIFICATIONS. ALL WELDING SHALL BE DONE WITH E-70 SERIES ELECTRODES.
12. HARDENED WASHERS SHALL BE INSTALLED OVER SHORT SLOTTED OR OVERSIZE HOLES OCCURRING IN AN OUTER PLY OF A CONNECTION.
13. THE STEEL JOIST MANUFACTURER SHALL INVESTIGATE THE ROOF JOISTS FOR A NET UPLIFT FORCE OF 15 PSF AND FURNISH THE NECESSARY FRAMING TO ENSURE PROPER JOIST PERFORMANCE UNDER UPLIFT DUE TO WIND AS WELL AS GRAVITY LOADING CONDITIONS.
14. PROVIDE SPECIAL JOIST SEATS WHERE REQUIRED BY NARROW BEARING CONDITIONS.
15. PAINT ALL STRUCTURAL STEEL THAT DOES NOT RECEIVE SPRAY-ON FIREPROOFING WITH ONE COAT OF RUST-INHIBITIVE PRIMER 2.5 MILS IN THICKNESS. THE COMPATIBILITY OF PRIMER AND ANY TOP COAT SHALL BE VERIFIED BEFORE ANY PAINTING IS PERFORMED. TOUCH-UP ALL EXPOSED METAL AFTER FIELD INSTALLATION. ALL STRUCTURAL STEEL WHICH IS EXPOSED TO THE ELEMENTS SHALL RECEIVE TWO COATS OF EXTERIOR ENAMEL WHICH IS COMPATIBLE WITH THE PRIMED SURFACE.
16. STRUCTURAL STEEL SHOP DRAWINGS SHALL INCLUDE COMPLETE DETAILS, CONNECTIONS, AND SCHEDULES FOR FABRICATION AND ASSEMBLY OF STRUCTURAL STEEL MEMBERS. STRUCTURAL STEEL SHOP DRAWINGS SHALL NOT INCLUDE MISCELLANEOUS STEEL.
17. STEEL JOIST AND JOIST GIRDER SHOP DRAWINGS SHALL BEAR THE SEAL AND SIGNATURE OF A REGISTERED ENGINEER IN THE STATE OF TENNESSEE CONFIRMING THE DESIGN OF JOISTS AND JOIST GIRDERS TO SJI SPECIFICATIONS AND FOR ALL LOADINGS SPECIFIED ON THE DRAWINGS. SHOP DRAWINGS WILL NOT BE REVIEWED BY THE DESIGNER UNTIL AFTER THE STRUCTURAL STEEL SUBCONTRACTOR AND GENERAL CONTRACTOR HAVE THOROUGHLY REVIEWED THE SHOP DRAWINGS, VERIFIED EXISTING CONDITIONS, AND COORDINATED THE SHOP DRAWINGS WITH OTHER AFFECTED TRADES.

Reference drawing S4.1

Figure 12 – General Notes (S4.1)

Reference drawing S4.2

Figure 13 – Quality assurance Plan as required by construction documents

SPECIAL INSPECTIONS AND TESTING

1. THE CONTRACTOR/OWNER SHALL EMPLOY AN INDEPENDENT TESTING COMPANY TO PERFORM SITE INSPECTIONS AND TESTING IN ACCORDANCE WITH THE QUALITY ASSURANCE PLAN SHEET S62.

STRUCTURAL OBSERVATIONS

1. THE CONTRACTOR/OWNER SHALL EMFLOY A LICENSED STRUCTURAL ENGINEER OR ARCHITECT TO PERFORM PERIODIC VISUAL OBSERVATIONS OF THE STRUCTURE DURING CONSTRUCTION FOR GENERAL CONFORMANCE TO THE DESIGN DRAWINGS.

From drawing S4.1, see figure 12

Figure 14 - General Notes

SPECIAL INSPECTOR SHALL PERFORM PERIODIC INSPECTIONS TO VERIFY THE FOLLOWING,

1. AS MASONRY CONSTRUCTION BEGINS, THE FOLLOWING SHALL BE VERIFIED TO ENSURE COMPLIANCE,
   A. PROPORTIONS OF SITE-PREPARED MORTAR.
   B. CONSTRUCTION OF MORTAR JOINTS.
   C. LOCATION OF REINFORCEMENT AND CONNECTORS.

2. THE INSPECTION PROGRAM SHALL VERIFY:
   A. SIZE AND LOCATION OF STRUCTURAL ELEMENTS.
   B. TYPE, SIZE, AND LOCATION OF ANCHORS, INCLUDING OTHER DETAILS OF ANCHORAGE OF MASONRY TO STRUCTURAL MEMBERS, FRAMES OR OTHER CONSTRUCTION.
   C. SPECIFIED SIZE, GRADE, AND TYPE OF REINFORCEMENT.
   D. PLACEMENT OF MASONRY DURING COLD WEATHER (TEMPERATURE BELOW 40 DEGREES FAHRENHEIT) OR HOT WEATHER (TEMPERATURE ABOVE 90 DEGREES FAHRENHEIT).
3. PRIOR TO GROUTING, THE FOLLOWING SHALL BE VERIFIED TO ENSURE COMPLIANCE,
   A. CLEANLINESS OF GROUT SPACE.
   B. PLACEMENT OF REINFORCEMENT AND CONNECTORS.
   C. PROPORTIONS OF SITE-PREPARED GROUT.
   D. CONSTRUCTION OF MORTAR JOINTS.
4. COMPLIANCE WITH REQUIRED INSPECTION PROVISIONS OF THE CONSTRUCTION DOCUMENTS AND THE APPROVED SUBMITTALS SHALL BE VERIFIED.

SPECIAL INSPECTION SHALL PERFORM CONTINUOUS INSPECTIONS TO VERIFY

THE FOLLOWING:

1. GROUT PLACEMENT SHALL BE VERIFIED TO ENSURE COMPLIANCE WITH CODE AND CONSTRUCTION DOCUMENT PROVISIONS.
2. PREPARATION OF ANY REQUIRED GROUT SPECIMENS, AND/OR PRISMS SHALL BE OBSERVED.

From drawing S4.2, see figure 13

Figure 15 – Quality assurance Plan as required by construction documents

Periodic inspections to be performed by the inspector included the following items which were to be verified to ensure compliance.

- Location of reinforcement and connection
- Size, grade and type of reinforcement
- Placement of reinforcement and connections

The inspector was required to keep records of all inspections, including test results, and was required to furnish reports to the Building Official and to the design professionals.

Based on the above requirements, the owner retained Beaver Engineering, Inc. (Beaver) to perform testing and inspection of structural components during construction of the project but with a somewhat reduced scope of work. During an interview with OSHA personnel, Beaver acknowledged that verification of rebars regarding their size and location was part of their responsibilities as reflected in Beaver's inspection reports. The signed contract between the owner and Beaver contained the following scope of work.

- Sample and test proposed soil or rock to be used as controlled fill.
- Observe proof rolling of exposed subgrade and recommend acceptance or further undercutting.
- Test and observe foundation bearing capacity.
- Perform QA/QC concrete tests according to project specifications.
- Perform QA/QC masonry tests according to project specifications.
- Report all test results to interested parties.

The inspector visited the site prior to the placement of concrete. The Code Inspector from the City of Hendersonville, TN visited the site only infrequently during construction.

Collapse

On April 18, 2013 at approximately 9:45 A.M. under a west wind with gusts of 33 mph, the east wall collapsed outwards towards the east. The remaining three walls at the perimeter of the building did not collapse. At the time of the east wall collapse, three employees were installing a backflow preventer in a trench that ran parallel to the east wall (see Figure 6). One of those employees was killed and another employee in the trench was injured when the wall fell outwards. Also, an employee on a ladder caulking the masonry control joints on the east wall (near the middle of the wall at the 2nd control joint, see Figure 6) was killed when the wall fell over him. During the review of the collapsed east wall photos (see Figure 16 to 21), the following items were noticed.

- Three interior braces and one exterior brace on the east wall fell along with the wall.

- The base of the entire east wall overturned outward and was completely separated from the top of the footing with no bent rebar dowels either coming out from the footing or from the wall.
- Some of the wall dowels from the footing to the wall were observed to have fractured at the base of the wall.
- Parts of the masonry blocks were disintegrated and turned into rubble.
- At certain areas of the fallen wall, cracks were visible.
- Parts of the bond beams were completely disintegrated and rebars from the bond beams were exposed, visible and were bent.
- At certain locations, the marks of the fractured rebars were visible either at the center of the CMU wall or at the edge of the CMU wall.

Inspection

The structural testing and inspection was performed by Beaver's representative. Beaver made observations of the structural components on March 18, 19, 20, and 21, 2013 and prepared a summary report for the week ending March 23, 2013. The inspection report for March 19, 2013 stated *"I was on site to observe reinforcing steel and concrete placement for the east, west, and south exterior wall footings and the entrance canopy pier footings.* ***I observed reinforcing steel construction noting bar placement, bar sizes, proper ties, and required clearances. The reinforcing steel appeared to meet project specifications."***

The above statement indicated that the contractor had placed wall dowels at the required locations (i.e., at the center of the CMU wall) but that is not supported by the photographs taken after the incident, and in statements made by the masonry contractor. When the masonry contractor began to build the wall, he noticed that the wall dowels (rebars) of the east wall at many locations (at north and south of the 6'-0" wide door opening) were offset from the correct location. He notified the general contractor of the misplacement of the dowels. Rather than stopping the work and getting guidance from the structural engineer of record, the general contractor advised the masonry contractor to bend the rebars and maneuver them in the block cells. Since the rebars were bent and placed near the inside edge of the CMU wall rather than being at the center of the CMU wall, rebars were not effective in resisting lateral loads arising out of the westerly wind. If the general contractor or inspector had promptly reported the misplacement of the rebars to the structural engineer of record, the structural engineer would have recommended corrective measures and the incident could have been prevented. One of the corrective measures was to drill new holes for the rebars in the footing at the center of the masonry wall and epoxy grout to meet the design intent.

Figure 16 - Collapsed wall

Figure 17 - Collapsed wall

Figure 18 - Collapsed wall

Figure 19 - Collapsed wall

Figure 20 - Collapsed wall

Figure 21 - Collapsed wall

<u>Structural Analysis and Discussion</u>

The purpose of the structural analysis was to:

1. determine whether the as-built masonry wall was adequate to resist the wind speed of 33 mph at the time of the collapse.
2. determine whether the temporary bracings in the manner they were installed could have supported the wall against wind loads imposed upon it at the time of the collapse.
3. determine whether the installation of the temporary bracings was properly done in accord with the applicable industry standards.

The following documents were reviewed.

1. HMHA architectural drawings dated November 1, 2012.
2. EMC structural drawings dated August 14, 2012.
3. Information including photographs related to the CMU wall received from the Division of Occupational Safety and Health of the State of Tennessee.

The structural analysis was limited to the collapsed east wall. The following assumptions were made for the analysis.

1. The density of the 8" thick hollow CMU wall was considered to be 101 pounds per cubic foot based on the contractor's average testing results of the blocks.
2. The height of the CMU wall was considered to be 24'-0" from the top of the wall footing.

3. The CMU wall was reinforced with # 5 at 40" on center and was considered grouted where rebar occurred.
4. The average dead weight of the CMU wall was considered as the wall was partially grouted to calculate the resisting moment under self-weight against overturning.
5. Based on architectural drawings, vertical control joint at the east wall was considered at 25'-0" on center (see Figure 5).
6. Three interior bracings and one exterior bracing at the east wall were considered to resist the wind loads. For calculation purposes, the top of the brace was considered to be 9'-0" above the top of the footing, and the bottom of the brace was assumed to be supported at the ground level. The CMU wall's upper height of 15' above the top of the brace was considered as a free-standing wall (see Figure 8).
7. The length of the brace was considered to be 16'. Bracing member used was 2x10 SYP OSHA plank.
8. The brace was considered to be pinned at both ends. The top of the brace was snugly fitted underneath the cleats (see Figure 9) while the bottom of the brace was held against the rebar embedded into the ground (see Figure 10). There was no positive connection between bracing members.
9. According to the Hendersonville Fire Department, the west wind speed including gust was considered to be 33 mph at the time of the incident.
10. The bracing of the CMU wall was analyzed for lateral loads between control joints at 25 feet on center.
11. The axial capacity in compression of the bracing member was checked using the strength design method. Load factors or strength reduction factors were not used in deriving the failure load of the bracing in compression.
12. Bracings were considered ineffective in resisting the tension loads, since no positive connections were placed between the brace and the wall.

The analysis indicated that if the contractor had placed the rebar dowels correctly in the footing at the center of the CMU wall, the incident would not have occurred because the masonry had gained adequate strength in 15 days, and the grouted rebar would have provided adequate flexural strength to resist the lateral loads. A significant number of the dowel reinforcements of the east wall that fell were misplaced to the outside edge of the masonry wall instead of being at the center of the wall. This compromised the flexural capacity of the free-standing wall under the lateral load coming from the west at the time of the incident. In addition, overturning moment due to lateral wind load was much higher than the resisting moment induced by the self-weight of the CMU wall.

The masonry contractor provided too few braces between the control joints of the 150'-long masonry wall that fell. In this project, the control joints in the masonry wall were designed and detailed as a complete separation (see Figure 5) similar to an expansion joint which necessitated a minimum of two braces for the masonry walls between the control joints (see Figure 11). Only three interior and three exterior braces for the entire wall were provided instead of the twelve specified in the industry standard, “Standard practice for Bracing Masonry Walls under construction” developed by the Council for Masonry Wall Bracing. Two exterior braces of the east wall were removed a few days prior to the collapse. All required braces should have been left in place until permanent supporting elements were constructed, e.g., the roof deck and its attachments to the bond beam at the top of the wall. If the masonry contractor had provided an adequate number of braces as per the industry standard, this incident could have been avoided.

Even the few braces that were provided did not meet the industry standards because they were not anchored to the wall either by bolts or screws. They were susceptible to sliding and falling off the walls. The wall could maintain its capability to prevent overturning either by the presence of an adequate number of braces properly fastened to the wall or by the internal strength derived from the flexural capacity due to rebars. In this case, neither was provided.

It is interesting to note that the west wall, opposite to the one that failed, did not collapse. The west wall had three interior braces to resist the west wind. In addition, the west wall was laterally restrained by the intersecting walls at the north and the south ends. We also believe that the dowels for the west wall were not misplaced and had developed adequate flexural strength as sufficient time of three weeks had elapsed for the grout to gain strength. In contrast, the east wall had only one exterior brace, no intersecting walls at the ends, and a significant number of misplaced dowels which were ineffective in resisting flexural bending under the west wind.

Conclusion

Based upon the above, we conclude that:

1. The masonry contractor provided too few braces between the control joints of the 150'-long masonry wall that fell. Only three interior and three exterior braces for the entire wall were provided instead of the twelve specified in the industry standards. Two exterior braces were prematurely removed a few days before the incident. All required braces should have been left in place until permanent supporting elements were constructed, e.g., roof deck and its attachments to the bond beams at the top of the wall. If the masonry contractor had provided an adequate number of braces as per the industry standard, this incident could have been avoided.
2. The inspector retained by the owner performed poorly by stating in his inspection report for the week ending March 23, 2013 that the ***"reinforcing steel appeared to meet the project specifications".*** In fact, a significant number of the dowel reinforcements of the east wall that fell were misplaced to the outside edge of the masonry wall instead of being at the center of the wall. This compromised the flexural capacity of the free-standing wall under the lateral load coming from the west at the time of the incident. If the inspector had promptly reported this misplacement, this incident could have been prevented despite the insufficient number of braces provided by the masonry contractor.
3. The general contractor, when made aware of the misplacement of the dowel bars by the masonry contractor, imprudently advised the masonry contractor to bend the bars and place them in the block cells. The general contractor should have stopped the work and asked for guidance from the engineer of record. New rebars at the center of the wall could have been drilled and epoxy grouted to meet the intent of the design. Bending the bars and placing them in the wall cells did little to improve the flexural capacity of the wall when the wind came from the west. It would have helped if the wind came from the east.
4. This wall collapse was waiting to happen since the free-standing masonry wall approximately 24' high was anchored to the footing at the edge of the wall instead of at the center of the wall, and due to the solitary exterior bracing leaning against the wall without any positive connection. The masonry at the time of the incident was approximately 20 days old and

should have been able to resist a wind speed of 33 mph if the wall was dowelled at its center into the footing as called for in the structural drawings.

5. The few braces that were provided did not meet the industry standards because they were not anchored to the wall either by bolts or screws. The braces were susceptible to sliding and falling off the wall.
6. The contractor violated OSHA standard 1926.706(b) which states that "*all masonry walls over eight feet in height shall be adequately braced to prevent overturning and to prevent collapse unless the wall is adequately supported so that it will not overturn or collapse. The bracing shall remain in place until permanent supporting elements of the structure are in place.*"