Evaluation of Concrete Masonry Unit Walls for Lateral Natural Phenomena Hazards Loads

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EVALUATION of CONCRETE MASONRY UNIT WALLS for LATERAL NATURAL PHENOMENA HAZARDS LOADS (U)

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ABSTRACT

Older single-story facilities (Pre-1985 vintage) are commonly constructed of structural steel framing with concrete masonry unit (CMU) walls connected to columns and roof girders of the steel framing system. The CMU walls are designed for lateral wind and seismic loads (perpendicular to the wall) and transmit shear loads from the roof diaphragm to the foundation footings. The lateral loads normally govern their design.

The structural framing system and the roof diaphragm system are straightforward when analyzing or upgrading the structure for NPH loads. Because of a building’s design vintage, probable use of empirical methodology, and poor design basis documentation (and record retention); it is difficult to qualify or upgrade CMU walls for lateral Natural Phenomena Hazards (NPH) loads in accordance with References 1, 2 and 3. This paper discusses three analytical approaches and/or techniques (empirical, working stress and yield line) to determine the collapse capacity of a laterally loaded CMU wall, and compares their results.

INTRODUCTION

Older single-story Department of Energy (DOE) Savannah River Site (SRS) facilities (Pre-1985 vintage), which are categorized as General Use or Low Hazard Facilities, are commonly constructed of structural steel framing with concrete masonry unit (CMU) walls connected to columns and roof girders of the steel framing system. The roof system is comprised of corrugated decking that is welded to steel girders and joists and serves as a shear diaphragm to transmit lateral loads to the CMU shear walls. The CMU walls are a critical structural element of the entire building’s structural system. They support portions of the roof’s dead loads, resist lateral wind and seismic loads (perpendicular to the wall), and serve as shear walls (used in lieu of diagonal bracing) to transfer lateral loads from the roof to the foundation footings. These lateral loads normally govern the design.

Since older CMU walls are more difficult to qualify or upgrade for lateral NPH loadings, this paper presents three analytical approaches and/or techniques (empirical, working stress, and yield line) which may be used to determine the collapse capacity of existing laterally loaded CMU structures, and compares their results. The analytical approaches and/or techniques are the Standard Building Code’s (SBC) empirical method, the Masonry Designers’ Guide and Uniform.
Building Code (UBC) working stress design, and the yield line methodology, commonly used for reinforced concrete structures. The empirical method provides code allowable pressure loads for CMU walls with certain height (or width) to wall thickness ratios (aspect ratios). The UBC working stress design analyzes laterally loaded CMU walls as a beam or plate type members using different boundary conditions, code limits and the material’s yield strength. The yield line methodology equates the External Work (EW) imposed on the masonry structure due to lateral pressure loads to the Internal Work (IW) based on the ultimate moment carrying capacity (energy dissipation) of the CMU walls.

In addition to evaluating CMU walls to code requirements, determining the elastic or collapse capacity (reserve capacity) is sometimes necessary to permit increasing the annual probability of exceedence for a facility that is being evaluated for NPH criteria. The methods discussed herein provide a means to do this.

BUILDING DESCRIPTION

The building is essentially rectangular in configuration. It is 250 feet long by 175 feet wide by 19 feet high. One end is wider by about 50 feet which induces additional torsional effects on the building. It is predominately single story except for one end bay that is 24 feet wide. This end bay is two stories. For the single story portion, the building is comprised of structural steel columns tied together at the roof by perimeter and interior roof girders, a roof diaphragm framing system, and CMU walls. The roof diaphragm system consists of corrugated metal decking rigidly attached to the roof’s joists and girder system. This system transfers loads to the CMU walls. The CMU walls are supported vertically to footings at the base of the wall with dowels and mortar, and to the roof girder’s bottom flange by inverted channels. Vertical reinforcement were not provided as required by code except around openings. They are supported horizontally at the columns by reinforcement bars spaced every 4 feet. These bars are anchored in blocks adjacent to the columns with concrete mortar. Also horizontally, the CMU walls have DUR-O-WAL reinforcement spaced every 16 inches. The CMU walls consist of approximately 35 rectangular CMU wall panels around the building’s perimeter. They range in size from approximately 16 feet wide by 16.33 feet high to 24.5 feet wide by 17.42 feet high. About one-half of the CMU wall panels have openings. The openings have vertical reinforcement from the base of the wall to the bottom flange of the roof girder. The second story is comprised of steel columns, roof/floor girders, purlings and metal siding. In some cases girder/column connections are designed as moment connections.

The analytical approaches and techniques discussed in this paper were used to evaluate the subject building for NPH criteria.

LOADING REQUIREMENTS

The building was initially designed as a low hazard facility for a seismic load of 0.1g (W) [where W = DL + .25 (LL)] and a wind speed of 84 mph. It was later evaluated per UBC seismic load of 0.0525 (W,) [where W, = DL] and a 84 mph wind speed. It has recently been evaluated per DOE STD-1020 criteria of 0.11g (W,) seismic load and a 84 mph wind.

The CMU walls were qualified seismically, but could not be qualified locally to code requirements for the 84 mph wind speed using the working stress design, primarily due to the lack of vertical reinforcement. Three additional scenarios were investigated. First, determine the wind speeds permitted by the empirical method. Second, determine the wind speeds allowed using the working stress design with elastic limits. Third, determine the wind speeds that would result in collapse of the building’s CMU walls.

METHODOLOGY

Analytical approaches and/or techniques, which could be used to evaluate CMU walls for wind pressure loadings, are the Standard Building Code’s (SBC) empirical method, the Masonry Designers’ Guide and Uniform Building Code (UBC) working stress design, and the yield line methodology.
The empirical method, based on test results, provides code allowable pressure loads for CMU walls with certain height (or width) to wall thickness ratios (aspect ratios). The Masonry Designers' Guide and UBC working stress design analyzes laterally loaded CMU walls as a beam or plate type members using different boundary conditions and code or elastic limits. The yield line methodology equates the External Work (EW) imposed on the masonry structure due to lateral pressure loads to the Internal Work based on the ultimate moment capacity of the CMU walls. Each of these methods are discussed in more detail below.

**EMPIRICAL METHOD**

For the building described above, two codes were reviewed for empirical limitations for CMU wall panels. They were the Standard Building Code (SBC) - 1976, Section 1405, and SBC - 1985, Section 1405. The SBC - 1976, Table 5, provides the type of masonry wall (wall construction) in a vertical column and the maximum ratio of unsupported height or length to wall thickness (H/t) in the horizontal column for mortar types M, S, N, & O. The SBC - 85, Table 1405.1, provides the wall construction type in a vertical column and the maximum ratio of unsupported height or length to wall thickness (H/t) in the horizontal column for design wind pressure loads of 15 psf, 20 psf, 25 psf and 30 psf.

To use these tables the CMU walls shall be supported either horizontally or vertically (whichever distant is the lesser) at right angles to the face of the wall at intervals not exceeding those in Tables 5 or 1405.1.

Lateral support shall be provided by intersecting walls, pilasters, columns, or other vertical members of sufficient strength to provide the required support when the distant is measured horizontally; or by floors, roofs, or other horizontal structural elements of sufficient strength to provide the required support when the distant is measured vertically.

Table 5 in SBC-1976 does not specify a wind speed, but the figure in the chapter on minimum design loads (equivalent to Figure 1205.1 of SBC - 1985) specifies a wind speed of 100 to 110 mph for the region where this building is located. It was concluded that these wind speeds were used in tests to develop Table 5. Therefore, by satisfying the requirements in the table, the CMU wall panel is adequate for 100 mph wind speeds at a minimum.

Table 1405.1 from SBC - 1985 specifies wind pressures, p. From Table 1205.1

\[ p = 0.00256 \left( \frac{V^3}{H/30} \right) \]  

(1)

Where:

- \( V \) = wind speed velocity
- \( H \) = the wall height above grade.

The wind pressure loads in Table 1405.1 include the shape factors from Section 1205. Knowing the wind pressure from Table 1405.1 and \( H \) from the design documents, a wind speed velocity, \( V \), can be calculated using Equation (1).

For purposes of this paper Table 1405.1 and Equation (1) were used to compute wind speeds for the CMU panels in Figures 2 through 4.

**WORKING STRESS DESIGN METHODOLOGY**

The working stress design was used to determine the allowable wind speeds based on code requirements and the yield strength of the material. Presented below is a brief discussion of the working stress design method from Reference 5 and calculation of wind speed capacities using CMU wall moment capacities, which were based on the working stress design.

Unreinforced masonry design is based on allowable tension in the masonry. Reinforced masonry design is based on design neglecting tensile strength of masonry and relying on reinforcement to resist tension. Assumptions common to both are:
1. Plane sections remain plane before and after bending.

2. Masonry components (units, mortar, etc.) combine to form a homogeneous member.

3. Stress is proportional to strain.

4. Modulus of elasticity of the reinforcement remains constant throughout the working load range.

5. Tensile forces are resisted only by the tensile reinforcement. The tensile strength of the masonry units, mortars, and grout is normally neglected in flexure analysis and design. However, for NPH evaluation, it may be considered, using caution.

6. Reinforcement is completely surrounded by and bonded to masonry material, and full composite action between the two materials is assumed.

Figure 1 presents commonly used symbols for flexural analysis. The depth to the centroid of the reinforcing steel is designated by the letter $d$. Location of the neutral axis from the compression face is designated $kd$, where $k$ is the ratio of the distant from the extreme compression fiber to the neutral axis location divided by the depth $d$.

The following relationships and equations provide the basis for flexural design of rectangular sections with tension reinforcement only. They are used to calculate capacities for moments and shears in the masonry wall of various configurations.

$$E_s = \text{modulus of elasticity of steel}$$

$$E_m = \text{modulus of elasticity of masonry units}$$

$$A_s = \text{area of steel reinforcement}$$

$$b = \text{width of section}$$

$$d = \text{distant from surface to center of } A_s$$

$$p = A_s / bd$$

$$n = E_s / E_m$$

$$k = [(np)^2 + 2np]^{1/2} - np$$

$$j = (1 - k / 3)$$

Summing moments about the centroid of the tensile force results in an expression of the allowable applied moment as limited by the allowable bending compression stress, $F_{hc}$, in the masonry:

$$M_m = (bd)^2 (jk) (F_{hc}) / 2$$

Summing moments about the centroid of the compression force results in an expression for the allowable applied moment as limited by the allowable tensile stress in the reinforcement, $F_s$.

$$M_t = A_s j d F_s$$

Flexural elements may contain both compression steel and tension steel (doubly reinforced). Design of doubly reinforced masonry units are not covered in this paper. See Reference 5 for development of equations used in the design of doubly reinforced masonry units.
While not specifically addressed in this paper, masonry unit beams must be evaluated for shear. The allowable shear stress $F_v$ is calculated from Code 7.5.2.2.(a):

$$F_v = \left[ f_m \right]^{1/2} \leq 50 \text{ psi} \quad (8)$$

If the code allowable is exceeded, shear reinforcement must be added.

Knowing the moment capacity for a particular CMU wall configuration, the next step is to determine what wind pressure loading will produce this moment. One could take a strip of the CMU wall panel and calculate a moment based on fixed-end beam equations from Reference 12 (or other sources). For this building a CMU wall panel was treated as a rectangular flat plate with the vertical and bottom edges fixed and the top edge pinned. Using the tables from Reference 13, moment equations were generated for points of maximum moment for the rectangular CMU wall panel. Other references are available for computing moments and shears for flat rectangular plates (i.e. Roark, ACI and DOE documents). Substituting the working stress design moment capacities into the equation, wind pressure loads were computed. Knowing the wind pressure load, a wind speed was computed using Equation (1).

YIELD LINE METHODOLOGY

The Yield Line Methodology (yield line theory) offers a simplified nonlinear analytical method that can determine the ultimate bending capacity of flat reinforced concrete slabs and walls subject to distributed and concentrated loads.

Alternately, yield line theory, combined with rotational limits can determine the energy absorption capacity of plates subject to impulsive and impact loadings. This method is especially useful in evaluating existing structures that can not be qualified using conservative simplifying analytical assumptions. This method for reinforced concrete structures has been expanded to predict collapse loads for concrete masonry unit walls subjected to wind pressure loads perpendicular to the walls.

The yield line theory is a relatively simple analysis method, that is based on failure mechanisms (yielding) being lumped into discrete plastic hinges or yield lines. Thus the concrete masonry unit wall is idealized as a series of rigid bodies which are connected together by yield lines. The Yield Line Methodology equates the External Work (EW) imposed on the concrete masonry unit walls due to lateral pressure loads to the Internal Work (IW) based on the ultimate moment capacity of the masonry structure. Since the method is an iterative process of predicting the correct yield failure mechanism, the quality of the solution depends on the postulated yield line mechanism.

The ultimate load capacity of typical reinforced concrete slabs and CMU wall panels is usually governed by bending, and yield line theory calculates the ultimate bending capacity. Yield line theory does not evaluate the in plane shear of the wall, and additional calculations must be made to preclude a shear failure.

For the building in question several critical concrete masonry unit wall panels (with various dimensions and configurations) were investigated. See Figures 2, 3, and 4 below. For these panel sizes and configurations possible yield line failures were idealized, as annotated in the figures. Figures 3 and 4 are three-dimensional to show the angle $\theta_l$ as the panels deflect inward $\delta_l$ as work is being done. Figure 2 is two-dimensional for clarity purposes (becomes too detailed if the rotational angle and inward deflection are shown). The dimensions in all three figures are in feet. Yield line panels are annotated by a number in parenthesis (i). The edges of a panel with rotational capabilities are annotated with a number, i. A hinge is assumed to form at edges without numbers.
Using the basic principal that the summation of external work is equal to the summation of internal work, we have:

\[ \Sigma(IW)_i = \Sigma(EW)_i \]  

Knowing that the IW is equal to the energy dissipated by the moment capacity of the panel as it rotates through the angle \( \theta \). Equating this to the EW, which is the work done as a panel is displaced a finite displacement, \( \delta_i \). We have:

\[ \Sigma(M_i \theta_1 \xi_1) = \Sigma(A_1 U \delta \xi_1) \]  

Where:

- \( M \) = Ultimate moment capacity (FT-KIPS/FT of yield line panel length) of the yield line panel (i) along a particular edge. For diagonal edges of a panel it will be required to resolve the moment capacities at the base, top or height edges of the yield line panel along the diagonal edge. For example, the moment along edge 3 in Figure 3 would be:
\[
M_i = M_2(\sin^2 \alpha_i) + M_1(\cos^2 \alpha_i) \quad (11)
\]

When resolving moments from two edges perpendicular to one another to a diagonal edge, care should be taken in using the correct moments and angles.

\[\theta = \text{Angle through which work is being done.} \quad \theta_i = \delta_i \text{ divided by } l_{3i}.\]

\[l_{1i} = \text{Length along the edge of a yield line panel with moment capacities.}\]

\[l_{2i} = \text{Distant from edge of the panel (i) to the deflected point, } \delta_i.\]

\[A_i = \text{Area of yield line panel (i).}\]

\[U = \text{External work or wind pressure load (ksf) being applied to the yield line panels, which is the term being solved for.}\]

\[\delta_i = \text{the maximum deflection of the yield line panel (i).}\]

Using the wall panels geometry and configuration, all values can be computed except for \(U\) and \(\delta_i\). Substituting known values into both sides of the equation, we get an expression where we solve for \(U\). See the expression below.

Knowing the material properties, geometry and boundary conditions of the panel in Figure 3, \(M_i, \theta_i, \alpha_i, l_{1i}\) and \(l_{3i}\) were computed. Substituting these values into Equation (10), we have:

\[(255.2)(U\delta_i) = 12.5 (\delta_i) \quad (12)\]

\[U = 0.049\ \text{ksf}\]

From Reference 12, \(U = 0.00256(k_v)(1.3)^2\) \(\text{(13)}\)

Solving for \(V\), wind speed velocity, we have

\[V = [U(1000 \text{ ksfi})(0.00256)(k_v)(1.3)^2]^{1/2} \quad (14)\]

Substituting known quantities, the estimated wind speed to collapse the concrete masonry unit panel in Figure 3 is:

\[V = 131\ \text{mph}\]

Using this same approach, the estimated wind speeds to collapse the concrete masonry unit panels in Figures 2 and 4 are 199 mph and 100 mph respectively.

CONCLUSIONS

For the three panels in Figures 2, 3 and 4, maximum wind speeds have been computed using the three methodologies discussed in this paper for an 8 inch CMU wall thickness. These wind speeds are listed below. \(V_e\) is the wind speed based on the empirical method. \(V_w\) is the wind speed based on the working stress method with elastic limits. \(V_y\) is the wind speed based on the yield line method.

Panel 1:

\[V_e = 69\ \text{mph}\]
\[V_w = 57\ \text{mph}\]
\[V_y = 199\ \text{mph}\]

Panel 2:

\[V_e = 69\ \text{mph}\]
\[V_w = 68\ \text{mph}\]
\[V_y = 131\ \text{mph}\]

Panel 3:

\[V_e = 69\ \text{mph}\]
\[V_w = 68\ \text{mph}\]
\[V_y = 100\ \text{mph}\]

As one would expect the working stress method gives a lower wind speed velocity than the empirical or yield line method because it is based on the working stress method (code equations) with elastic limits, which have inherent conservatisms.
The yield line method gives a higher wind speed velocity than the empirical or working stress method because the energy absorption capabilities of the structure are being utilized, as well as the development of moment capacities at both edge and midspan of the the CMU wall panel. The wind speed would be around 96 mph using the empirical method if the CMU wall panels for this building were treated as cavity walls (i.e. The wall thickness should be the 8 inch CMU wall plus the 4 inch brick wall, or 12 inches.). Reference 4 allows this to be done if the brick and masonry units are adequately tied together. The results in this paper are based on 8 inches for consistency. Based on the CMU wall panels evaluated, the results presented above, and the intent of Reference 4, it is believed that the yield line method will also produce higher wind speeds for cavity walls to cause building collapse, than either the empirical or working stress method. Care should be exercised when using the empirical data to ensure that all the boundary conditions and caviates have been met.

The yield line method is an upper bound solution and the quality of the solution is dependent upon the assumed yield line mechanism. Normally, several yield line failures have to be evaluated (i.e. Several failure mechanisms were evaluated for Panels 2 and 3, which are the same.), since the specific geometry of the yield line is not obvious initially. Different geometries are analyzed and the specific geometry with the lowest capacity is adopted.

Concrete and steel attributes of the building were not addressed in this paper, but References 14 (concrete) and 15 (steel) provide code/criteria requirements to use when evaluating concrete and steel structures for NPH loadings.

REFERENCES:


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