TEK 14-24

DESIGN OF REINFORCED CONCRETE MASONRY DIAPHRAGM WALLS

INTRODUCTION

Masonry is a versatile and robust structural system. The available variety of materials, shapes and strengths offers countless opportunities to create many types of masonry elements. Masonry’s versatility offers a continuous spectrum of systems from unreinforced to reinforced or post-tensioned. One example of such versatility is reinforced diaphragm walls. While not specifically mentioned in Building Code Requirements for Masonry Structures (TMS 402) (ref. 1), reinforced diaphragm walls can be designed and constructed using criteria in that standard.

Diaphragm walls are cellular walls composed of two wythes of masonry with a large cavity or void; the wythes of which are bonded together with masonry ribs or crosswalls (see Figure 1). The ribs are connected to the wythes in such a way that the two wythes act compositely, thereby giving a fully composite section. This TEK covers the structural design of reinforced diaphragm walls. See TEK 3-15, Construction of Reinforced Concrete Masonry Diaphragm Walls,(ref. 2) for information on constructing diaphragm walls.

Figure 1 shows an example of a diaphragm wall constructed with concrete masonry units and its associated terminology. The reinforced wythes can be fully or partially grouted. The exterior face can be treated as the weathering side of the wall as shown in Figure 1, or an anchored veneer can be used on the exterior face. The internal cavity (void) of the diaphragm wall is left open.
ADVANTAGES

Reinforced diaphragm walls present several opportunities for masonry design. These include:

1. Diaphragm construction can efficiently create strong, stiff walls with individual units bonded together. Consider the economy of building a 24-in. (610-mm) thick wall with two 6 in. (152 mm) wythes and a 12 in. (305 mm) cavity rather than a solid 24 in. (610 mm) wall.

2. Thick diaphragm walls can be designed to span much further horizontally or vertically than single wythe walls or conventional composite walls. It is also possible to make very tall walls by virtue of the large sectional stiffness (ref. 3).

3. The greater thickness of diaphragm walls can also be used to replicate historic walls (buildings of Gothic style, monasteries, etc.) using modern methods.

4. The walls can have exposed, finished surfaces inside and out, and those finishes can be different because they are created by two individual wythes of masonry units.

5. The exterior wythe can be flashed and drained similar to the conventional back-up of an anchored veneer in cavity wall construction as detailed in TEK 19-5A, *Flashing Details for Concrete Masonry Walls*, or for single wythe walls per TEK 19-2B, *Design for Dry Single-Wythe Concrete Masonry Walls* (refs. 4, 5).

6. The large interior voids allow for placement of insulation and utilities.

7. These walls can generate significant out-of-plane load capacity while supporting in-plane lateral loads.

8. The two distinct wythes provide a resilient system that can resist debris penetration from a high wind event and also provide great protection to potential blasts. With the high out-of-plane lateral load resistance, these walls can provide a good option for safe rooms or community rooms in tornado and hurricane regions.

HISTORICAL PERSPECTIVE

Unreinforced diaphragm walls have been used in Great Britain for decades. Many have been built using both concrete and clay masonry (Reference 3 provides wall diaphragm design criteria for concrete masonry assemblies used in Great Britain). The philosophy for unreinforced masonry in flexure is that the mortar controls the flexural tensile resistance and the composite of masonry and mortar controls both the compressive and shear stresses.
Valuable characteristics of unreinforced diaphragm walls are that the net section properties are easily calculated and they have a large moment of inertia. Given that they are thick, unreinforced diaphragm walls are effective at resisting out-of-plane loads and are inherently very stiff. Unreinforced walls, however, often crack before deflections control the performance. To further increase the bending resistance of unreinforced diaphragm walls, many walls in Great Britain have been post-tensioned. The post-tensioning tendons are often placed in the void, unbonded and unrestrained, and protected from corrosion.

Unreinforced diaphragm walls have been used for sports halls, swimming pools, theaters, cinemas and other buildings that require tall walls. Other applications include tall freestanding walls, retaining walls, and replicating historical construction.

Figure 2 shows a fire station in Great Britain with posttensioned diaphragm sidewalls (arrows). These walls provide lateral stability for the building in both directions. As with traditional masonry buildings, the sidewalls are shear walls and resist loads acting on the front and rear of the building. In the transverse direction (plane of the overhead doors), the large openings leave short pier sections. Therefore, the diaphragm walls are designed to act as cantilever walls to provide the transverse building stability. This is a unique design solution because most masonry buildings do not depend upon the out-of-plane strength and stiffness of the walls to provide stability against lateral loads. Diaphragm walls, however, can be designed with sufficient thickness to develop the necessary out-of-plane strength and stiffness.

Figure 3 shows a cross-section of a bridge abutment and a photograph of the completed bridge where unreinforced post-tensioned brick diaphragm walls were used. Various bridges also use diaphragm walls for the cantilever wingwalls.

Diaphragm walls have not been specifically addressed by name in codes and standards in the United States. Even the definition of a diaphragm wall does not exist, however, Building Code Requirements for Masonry Structures (TMS 402) includes design methodologies for unreinforced masonry using allowable stress design and strength design, as well as design criteria for composite assemblies. Therefore, unreinforced and reinforced diaphragm walls can be designed using the existing standards, despite the fact that there is no specifically stated diaphragm wall criteria.
REINFORCED DIAPHRAGM WALLS

Even though unreinforced masonry is possible in areas of the United States, reinforced masonry is more widely adopted. Most regions require reinforcement for commercial masonry construction based upon the *International Building Code* (IBC) (ref. 6). TMS 402 provides design methodologies for reinforced masonry using allowable stress methods, post-tensioning, and strength design. These provisions can all be applied to reinforced concrete masonry diaphragm walls.

Design Detailing

Regardless of the design method utilized, there are some detailing criteria that apply equally to all reinforced diaphragm walls. These criteria are outlined below.

a) Spacing of Ribs
The ribs of the reinforced diaphragm wall act as webs for out-of-plane loads and connect the wythes structurally to create a composite section.

It is preferable that the ribs be spaced so that the flanges are fully effective in resisting applied loads. This is controlled by TMS 402 Section 5.1.1.2 which governs wall intersections. For reinforced walls where both flanges experience compression and tension, TMS 402 requires the effective flange width on either side of the web to not exceed 6 times
the flange thickness or 0.75 times the floor-to-floor height. In addition, the effective flange width must not extend past a control joint.

Therefore, the effective clear spacing between ribs is \(12 \times t_{wythe}\) for walls without control joints (\(6 \times t_{wythe}\) from each rib), and the effective flange width is \(12 \times t_{wythe} + t_{rib}\). Figure 4 illustrates how this effective flange width is smaller when a control joint is located at a rib. When placing a control joint between ribs, the flanging effect does not extend past the control joint.

It is possible to extend the rib spacing beyond \(12 \times t_{wythe}\), however, only the \(12 \times t_{wythe} + t_{rib}\) portion of the flange can be considered to be effective in the design calculations.

b) Flange Thickness
The masonry unit selected for the flange wythe dictates the flange thickness \((t_{wythe})\). To accommodate reinforcement, a 6-in. (152-mm) concrete masonry unit is the smallest practical unit to be used. Larger units can be used to accommodate larger bars and provide larger compression areas.

c) Grouting
The choice of full vs. partial grouting is a function of design:
1. If the compression area required by out-of-plane design exceeds the face shell thickness of the wythe, the recommendation is to fully grout the flanges. Alternatively, the designer can use partial grouting and perform a T-beam analysis on the wall.

2. If the compression area does not exceed the face shell thickness of the wythe, either partial or full grouting can be used without using the more cumbersome T-beam analysis.

3. The ribs are often fully grouted, but they can also be designed with partial grouting.

d) Masonry Bond
TMS 402 Section 5.1.1.2.1 requires that intersecting walls be constructed in running bond for composite flanging action to occur. Therefore, reinforced diaphragm walls are always constructed in running bond.

e) Connecting the Ribs to the Wythes
TMS 402 Section 5.1.1.2.5 requires that the connection of intersecting walls conform to one of the following requirements:
1. At least fifty percent of the masonry units at the interface must interlock.

2. Walls must be anchored by steel connectors grouted into the wall and meeting the following requirements:
(a) Minimum size: 1/4 in. x 1-1/2 in. x 28 in. (6.4 x 38.1 x 711 mm) including a 2-in. (50.8-mm) long, 90-degree bend at each end to form a U or Z shape.
(b) Maximum spacing: 48 in. (1,219 mm).

3. Intersecting reinforced bond beams must be provided at a maximum spacing of 48 in. (1,219 mm) on center. The area of reinforcement in each bond beam must be not less than 0.1 in.² per ft (211 mm²/m) multiplied by the vertical spacing of the bond beams in feet (meters). Reinforcement must be developed on each side of the intersection.

The use of bond beams in requirement 3 above is one way of handling the interface shear requirement. However, the equations below can also be used for this purpose:

For allowable stress design:
\[ f_v = \frac{V}{A_n} \text{TMS 402 Section 8.3.5.1.1 (Eqn. 8-21)} \]
where \( F_v \) is controlled by Section 8.3.5.1.2.

For strength design, the shear strength, \( f_v \), is controlled by TMS 402 Section 9.3.4.1.2.

f) Control (Movement) Joints
TEKs 10-2C, Control Joints for Concrete Masonry Walls—Empirical Method and 10-3, Control Joints for Concrete Masonry Walls—Alternative Engineered Method (refs.7, 8) are the industry standards for determining control joint spacing. Both were developed for single wythe walls with and without horizontal reinforcement.

There is no specific research on shrinkage characteristics of reinforced diaphragm walls.

Vertical reinforcement has no effect on crack control and vertical ribs would seem to act similarly. Until research becomes available, the current recommendation is to use the existing industry crack control recommendations to space control joints for reinforced diaphragm walls (refs. 7, 8).

Additional attention must be placed on the size of the corner control joints if the diaphragm walls are used to support out-of-plane loads (see Design Example).

Allowable Stress Design of Reinforced Diaphragm Walls

The allowable stress design (ASD) methodology for reinforced diaphragm walls is similar to reinforced single wythe wall design and is discussed in TEK 14-7C, Allowable Stress Design of Concrete Masonry (ref. 9). The maximum wall height is controlled by the loadings and slenderness effects. The slenderness effects are based upon the \( h/r \) ratio and prevent the wall from buckling.

Strength Design of Reinforced Diaphragm Walls
The strength design method has no specific limit on $h/t$, however, it has design criteria that limit service load deflections and ultimate moment capacity for out-of-plane loads. The service load deflections cannot exceed 0.7 percent of the wall height. For a 30-ft (9.1 m) wall, that is 2.5 in. (64 mm) of deflection for a simply supported wall.

There is an axial load capacity limitation when $h/t$ exceeds 30: the factored axial load for these walls must be limited to 5 percent of $f'_m$ based upon the gross section properties.

The design methodology is similar to single wythe design and is discussed in NCMA TEK 14-11B *Strength Design of Concrete Masonry Walls for Axial Load and Flexure* (ref. 10).

**Reinforced Concrete Masonry Diaphragm Walls Using Post-tensioned Masonry Design**

Post-tensioned masonry design of diaphragm walls is the same as single wythe design, however, the large void in diaphragm walls provides an opportunity for the tendons to be placed eccentrically as needed for the loadings. Placed inside the void, the tendons are generally unbonded and unrestrained, although adding restraint will improve the performance of the wall.

**Seismic Design**

TMS 402 and ASCE 7 (refs. 1, 11) provide additional criteria for seismic design of walls that need to be considered as for any other masonry wall. This includes the degree of grouting and the inclusion of prescriptive reinforcement.
Figure 3—Cross-Section of Diaphragm Walls for Abutment and Completed Bridge (courtesy of Malcolm Phipps)
Figure 4—Effective Flange Width, $b_{effective}$

**DESIGN EXAMPLE: WINGWALL DESIGN FOR A REINFORCED CONCRETE MASONRY MAINTENANCE STORAGE FACILITY**

Figure 5 shows the basic building layout for the design example. The front and rear walls are perforated with 20 ft x 20 ft (6.1 x 6.1 m) overhead doors for vehicle access. Control joints are shown over the door openings; the pier sections are 6 ft (1.8 m) in length. The endwalls have personnel access openings. Because the front and rear walls are perforated, the pier sections may not have sufficient in-plane stiffness and strength. Therefore, the endwalls should be designed to brace the building in both directions.

Although the roof structure is not shown, long-span joists bear on the front and rear sidewalls (i.e., the walls with the large perforations); the endwalls are nonloadbearing. The roof diaphragm would be designed to distribute the front-rear lateral loads to the endwalls, which must be designed as conventional shear walls. Conventional shear wall design is covered by the *Masonry Designer's Guide* (ref. 12) and is not covered here.

The roof diaphragm will not be used to brace the side-to-side lateral forces. For this example, the out-of-plane design (the large red arrow in Figure 5 depicts the out-of-plane load) will treat the endwalls as diaphragm walls acting as cantilevers to brace the building for the side-to-side lateral loads similar to Figure 2. This decision exempts the roof diaphragm from the strength and stiffness requirements for lateral loads that are perpendicular to the plane of the roof trusses. These requirements are typically met by horizontal braces between roof trusses.

Input:
Location: Coastal US, South Carolina
Loadings: ASCE 7-16, Part 2 for wind design
Masonry Standard: TMS 402, ASD method Because no bracing is used at the top of the wall, component and cladding loads will be used to design the wall.

1. **Proposed wall section**
   Use 6-in. (152-mm) concrete masonry units for wythes and 8-in. (203-mm) for ribs (see Figure 6).
   Masonry units: ASTM C90 (ref. 13), $f'_{m} = 2,000$ psi (13.8 MPa); unit weight 125 pcf (2,000 kg/m3)
   Reinforcement: ASTM A615 (ref. 14), Grade 60
   Grout: ASTM C476 (ref. 15), 2,000 psi (13.8 MPa)
   Mortar: ASTM C270 (ref. 16), Type S
   Begin the design assuming the wall is fully grouted and verify later (Item 10).

2. **Select control joint spacing**
   The three possible options are:
   a) Using TEK 10-2C (empirical method), space control joints at the lesser of $1.5h = 45$ ft (13.7 m), max 25 ft (7.62 m). The 25 ft (7.62 m) criteria governs. The required horizontal reinforcement in the walls is 0.025 in.$^2$/ft (TEK 10-2C, Table 1). This corresponds to two-wire W1.7 (9 gauge, MW11) joint reinforcement in each wythe at 16 in. (406 mm) on center vertically over the height of the wall (TEK 10-2C, Table 2).

   b) Using TEK 10-3 (alternative engineered method), space control joints at the lesser of $2.5h = 75$ ft or 25 ft (7.62 m). Again, the 25 ft (7.62 m) criteria governs. The required horizontal reinforcement in the walls is 0.0007$A_{n}$, which corresponds to two-wire W1.7 (9 gauge, MW11) wire joint reinforcement at 8 in. (203 mm) on center vertically over the height of the wall (TEK 10-3, Tables 1 and 2).

   c) Using TEK 10-3, space control joints at any length provided the horizontal reinforcement in the walls exceeds 0.002$A_{n}$ (TEK 10-3, Table 1 footnote 2). This corresponds to 0.135 in.$^2$/ft or one No. 6 (M#19) reinforcing bar in bond beams at 32 in. (813 mm) on center vertically over the height of the wall (TEK 10-3, Table 3 for fully grouted walls).
   To minimize the possible number of control joints in the diaphragm walls, select option c) with the horizontal bond beams. Provide control joints only at the corners (Figure 5).
   If the designer chooses to use horizontal joint reinforcement and not bond beams, the maximum control joint spacing would be 25 ft (7.62 m) using either options a) or b).
   While the inner wythe will generally be exposed principally to shrinkage with only minor thermal effects, it is common to reinforce both wythes similarly.

3. **Determine wind loads**
   From ASCE 7-16 Part 2, the suction load at the Exterior Zone (5) is calculated as 66.3 psf (3.17 kPa) (see Figure 6). In ASCE 7, wind loads are strength level. Roof dead load is ignored at the nonbearing wall.
4. Determine base of wall loads

\[ V_u = 66.3 \text{ psf} \times 30 = 1,989 \text{ lb/ft of wall (29.0 kN/m)} \]
\[ M_u = 66.3 \times (30)^2/2 = 29,835 \text{ ft-lb/ft of wall (132 kN-m/m)} \]
\[ V_{ser} = 0.6 \ V_u = 1,193 \text{ lb/ft of wall (17.4 kN/m)} \]
\[ M_{ser} = 0.6 \ M_u = 17,901 \text{ ft-lb/ft of wall (79.6 kN-m/m)} \]

Note: 0.6 reduces \( V_u \) to ASD per ASCE 7.

5. Determine effective in field of wall (solid region away from openings):

\[ b_{\text{effective}} = 12t_{\text{wythe}} + t_{\text{rib}} = 12(6 \text{ in.}) + 8 \text{ in.} = 80 \text{ in. (2,032 mm)} \]

6. Determine minimum \( t_{wall} \) to satisfy shear capacity

\[ V_{rib} = V_{ser} \times 80/12 = 7,953 \text{ lb (35.4 kN)} \]
\[ f_v = V_{rib}/A_{rib} = 7,953/[(7.63 \text{ in.}) \cdot t_{wall}] \] (TMS 402, Equation 8-21)
\[ F_v = 2 \sqrt{f_m \gamma_g} = 89 \text{ psi, assuming } M/Vd > 1.0 \text{ and } \gamma_g = 1.0 \] (TMS 402, Equation 8-24)

This produces \( t_{wall} \geq 11.7 \text{ in. (297 mm)} \)

Checking \( M/Vd = 17,901/[1,193 x (<1 \text{ ft})] = 15.0 > 1.0 \) OK
Shear is not an issue. The prescriptive requirements for the intersection of the ribs and flanges are sufficient.

7. Determine minimum \( t_{wall} \) due to moment capacity

Try a rib length of 1.5 courses of concrete masonry.

\[ t_{wall} = 15.625 \text{ in. unit} + 0.375 \text{ in. mortar joint} + 7.625 \text{ in. half unit} = 23.625 \text{ in. (600 mm)} \]
\[ d = 23.63 \text{ in.} - (5.63 \text{ in.}/2) = 20.82 \text{ in. (529 mm)} \]

Ignoring axial load,
\[ A_s \text{ (estimated)} = M_{ser}/(28.8d) \]
\[ = (17,901/1,000)(12 \text{ in./ft})/[28.8 \times 20.82] \]
\[ = 0.36 \text{ in.}^2/\text{ft} \] (0.77 mm$^2$/mm)

Maximum bar size = No. 6 = 0.75 in. (19 mm) per TMS 6.1.2.5. Try 2 No. 6 bars at 24 in. \( A_s = 0.44 \text{ in.}^2/\text{ft} \) (2- M#19 @ 610 mm). Check maximum area of reinforcement < 6 percent (TMS 6.1.2.4): 2 x 0.44/(4x6) is equivalent to 3.7 percent...OK

8. Determine wall dead load at base of wall

From TEK 14-13B (ref. 17): wall weight of 125 pcf 6 in. fully grouted concrete masonry = 62 psf (303 kg/m$^2$ )

Flange load: 2 wythes x 62 psf = 124 psf per ft

Rib load: \( [23.63 \text{ in.} - 2(5.63 \text{ in.})]/12 \times 84 \text{ psf/80 in.}/12 = 13.0 \text{ psf/ft of wall} \]

\[ P_{DL} = (124 + 13.0) \times 30 \text{ ft} = 4,110 \text{ lb/ft of wall (60 kN/m)} \]

9. Load combination
0.6 \( P_{DL} \) + 0.6\( W \) from ASCE 7-16 for ASD

Note: This one load combination is shown for this example. The designer must check all combinations required by ASCE 7.

\[
P = 0.6 P_{DL} = 0.6 \times 4,110 = 2,466 \text{ lb/ft (36 kN/m)}
\]

\[
M = 0.6 M_{u, \text{wind}} = M_{ser} = 17,901 \text{ ft-lb/ft (79.6 kN-m/m)}
\]

10. Determine \( n \)

From TMS 402 Section 4.2.2:

\[
E_s = 29,000,000 \text{ psi (200,000 MPa)}
\]

\[
E_m = 900 \text{ psi} \cdot 1 = 1,800,000 \text{ psi (12,410 MPa)}
\]

\[
n = E_s / E_m = 16.1
\]

For \( A_x = 0.44 \text{ in.}^2 / \text{ft} \) (from 7 above), \( n_{\rho} = nA_s / bd = 16.1(0.44)/12(20.82) = 0.028 \)

If \( P = 0 \), approximate \( k = \sqrt{n_{\rho}^2 + 2n_{\rho} - n_{\rho}} = 0.222; \)

\( j = 1 - (k/3) = 0.926 \)

\( k_d = 4.62 > t_{\text{face}} \) of 6-in. CMU but less than the wythe thickness. Axial load may increase \( k_d \). Therefore, grouting the full wythe is appropriate.

11. Design for \( P_{DL} \) and \( M \)

(see Figure 7)

From statics: \( P = C - T \)

\[
M = C \times e_m + T \left( d - t_{\text{wall}}/2 \right)
\]

Per foot: \( C = 1/2(kd)f_m \times 12 \text{ in.} \)

\[
f_m = E_m e_m
\]

\[
T = A_s f_s
\]

\[
f_s = E_s e_s
\]

\[
e_m = t_{\text{wall}}/2 - kd/3
\]

From strain compatibility: \( e_m/kd = e_s(d - kd) \)

\[
(f_m / E_m)/kd = (f_s / E_s)/(d - kd) \rightarrow f_s = n [(d - kd)/kd] f_m
\]

Therefore, \( C = 6(kd)f_m \)

\[
T = 0.44(16.1)((20.82 - kd)/kd)) f_m
\]

\[
= 7.08((20.82 - kd)/kd)) f_m
\]

Solving for \( P = C - T \) and \( M = C e_m + T \left( d - t_{\text{wall}}/2 \right) \)

gives \( kd = 4.86 \text{ in. (123 mm)} \) and \( f_m = 417 \text{ psi (2.9 MPa)} \)

Checking:

\[
C = 12,160 \text{ lb (54 kN)}
\]

\[
T = 9,695 \text{ lb (43 kN)}
\]

\[
P = 2,466 \text{ lb (10.9 kN)} \text{ OK}
\]

\[
e_m = t_{\text{wall}}/2 - kd/3 = 10.2 \text{ in. (259 mm)}
\]

\[
M = C e_m + T \left( d - t_{\text{wall}}/2 \right)
\]

\[
= 12,160(10.2)/12 + 9,695(20.82 - 23.63/2)/12
\]

\[
= 17,607 \text{ ft-lb approx.} = M = 17,901 \text{ ft-lb OK}
\]

Check:

\[
f_m = 417 \text{ psi} \leq F_b = 0.45 f'_m = 900 \text{ psi (6.2 MPa)}
\]
OK (TMS 402 8.3.4.2.2)

\[ f_s = 16.1 \left( \frac{20.82 - 4.86}{4.86} \right) \quad 417 \text{ psi} = 22,047 \text{ psi (152 MPa)} \]

\[ f_s < F_s = 32,000 \text{ psi (221 MPa)} \quad \text{OK (TMS 402 8.3.3.1)} \]

TMS 402 Section 8.3.4.2.1 requires an additional check for Pa alone. The design engineer is generally advised to perform this check. However, it rarely controls for diaphragm walls due to the stiff wall section. For this example there is no applied axial load, so the check is not required.

Therefore, this section checks using two No. 6 bars at 24 in. on center (two M#19 at 610 mm) in a fully grouted diaphragm wall. Note that this only applies to the end zone in suction. The design calculations should be repeated:

a. for pressure load on the end zone,

b. for pressure and suction over the interior zone,

c. over the height of the wall to reduce the amount of vertical reinforcement, and

d. the design should be checked adjacent to control joints and openings.

Using the walls to support of out-of-plane loads requires the foundations to be designed and detailed for the cantilever walls.

12. Horizontal span of exterior wythe

Check that the exterior wythe is adequately reinforced to span horizontally between the ribs. From the control joint spacing analysis (Item 2), we have No. 6 bars (M#19) in bond beams at 32 in. (813 mm) on center vertically. The span between ribs is 80 in. or 6.67 ft (2,032 mm) from Item 5.

Because the No. 8 (M#25) vertical bars occupy the center of the wythe, the horizontal bars are offset such that the \( d \) value for the bond beam is:

\[ d = \frac{5.625}{2} - \frac{1.0}{2} - \frac{0.75}{2} = 1.93 \text{ in. (49 mm)} \]

Load on the bond beam = 66.3 psf x 32 in./12

= 177 pfl (2.6 kN/m)

\[ M_u = 177 \times (6.67)^2 / 8 = 984 \text{ ft-lb (1.34 kN-m)} \]

\[ b_{\text{effective}} = 32 \text{ in. (813 mm)} \]

\[ a = \frac{A_s f_y}{0.8 (f'_m)b} \]

\[ = \frac{0.44(60,000)}{0.8(2,000)32} \]

\[ = 0.52 \text{ in. (13 mm)} \]

\( < \) face shell thickness \( (t_{\text{face}} = 0.75 \text{ in., 19 mm}) \) OK...no need for T-beam analysis

\[ M_u \text{ capacity} = \Phi[A_s g'_y (d - a/2)] = 0.9 [0.44(60,000)(1.93 - 0.52/2)/12] = 3,306 \text{ ft-lb (4.5 kN-m)} \]

> 984 ft-lb (1.34 kN-m) OK

Therefore, bond beams at 32 in. (813 mm) o.c. vertically with No. 6 bars (M#19) works for both crack control and lateral loads. The same is used on the interior wythe.

13. Check deflection at top of the wall for a cantilever
Using loads and section properties for $b_{effective}$.

$$\delta = 1/4 \left[ M_{cr} \frac{h^2}{E_m I_g} \right] + 1/4 \left[ (M_{ser} - M_{cr}) \frac{h^2}{E_m I_{cr}} \right]$$

This equation is a modified version of TMS 402 Equation 9-26. The first modification was converting Equation 9-26 from strength to service loads. Equation 9-26 was developed for a simply supported wall. The second modification was converting it to a cantilever wall.

$$I_g = b \left( \frac{t_{wall}}{2} \right)^3 - (b - t_{rib}) \left( \frac{t_{wall} - 2t_{wythe}}{2} \right)^3$$

$$I_g = (80 \text{ in.}) \left( \frac{23.63}{2} \right)^3 - (80 - 7.63) \left( \frac{23.63 - 2(5.63)}{2} \right)^3$$

$$I_g = 87,963 - 11,415 = 76,578 \text{ in.}^4 (0.032 \text{ m}^4)$$

Note: The void only reduces the $I_g$ by 13% from a completely solid wall, yet the area reduction is approximately 47%. This highlights a significant benefit of a tall diaphragm wall.

$$S_g = I_g / (t_{wall}/2)$$

$$= 76,578/11.82 = 6,479 \text{ in.}^3 (0.11 \text{ m}^3)$$

$$M_{cr} = S_g f_r /12 \text{ in./ft}$$

$$f_r = 163 \text{ psi from TMS 402 Table 9.1.9.2}$$

$$M_{cr} = 88,006 \text{ ft-lb (119 kN-m)}$$

$$M_{ser} = 0.6(M_u) \times 80/12$$

$$= 0.6 (29,835 \text{ per ft of wall}) \times (80/12)$$

$$= 119,340 \text{ ft-lb (162 kN-m)}$$

$$I_{cr} = n[A_s + (P_{ut} t_{sp})/(f_y 2d)](d - c)^2 + (bc^3)/3 \text{ TMS 402 Eq. 9-30}$$

$$c = (A_{Sy} + P_{ul})/(0.64f'_m b) \text{ TMS 402 Eq. 9-31}$$

$$= [(0.44 \times 80/12)(60,000) + (1.2P_{DL})]/[0.64(2,000)(80)]$$

$$= [(176,000) + (1.2(4,110 \times 80/12))]/102,400$$

$$= 2.0 \text{ in. (52 mm)}$$

$$I_{cr} = 16.1(0.44(80)/12 + (32,880)(23.63)/(60,000 \times 2 \times 20.82))$$

$$x (20.82 - c)^2 + (80c^3)/3$$

$$= 16.1(2.93 + 0.31)(18.82)^2 + (80 \times 2.0^3)/3$$

$$= 16.1[1,143] + 213$$

$$= 18,696 \text{ in.}^4 (0.007 \text{ m}^4)$$

Note: this is approximately 24% of $I_g$

$$\delta = 1/4 \left[ M_{cr} \frac{h^2}{E_m I_g} \right] + 1/4 \left[ (M_{ser} - M_{cr}) \frac{h^2}{E_m I_{cr}} \right] = 1/4 (0.99 \text{ in.}) + 1/4 (1.18) = 0.54 \text{ in. (14 mm)}$$

Provide the control joints between the endwalls and the front/rear walls. Construct with sealant that has a shear capacity of 50% of the joint thickness, the joint thickness should exceed 2 x 0.56 in. = 1.12 in. (28 mm). See white arrow on Figure 5.
Figure 5—Maintenance Facility for Design Example 1

Figure 6—Wall Sections
SUMMARY

Reinforced concrete masonry diaphragm walls provide opportunities for engineers to design:

a) very tall walls, and
b) brace walls using the diaphragm walls as cantilevers. For buildings, these are two unique options that are not normally available from traditional masonry walls.

NOTATIONS

- $A_n$ = net cross-sectional area of a member, in.$^2$ (mm$^2$)
- $A_s$ = area of nonprestressed longitudinal tension reinforcement, in.$^2$ (mm$^2$)
- $b$ = width of section, in. (mm)
- $b_{\text{effective}}$ = effective width of section, in. (mm)
- $C$ = resultant compressive force, lb (N)
- $c$ = distance from the fiber of maximum compressive strain to the neutral axis, in. (mm)
- $d$ = distance from extreme compression fiber to centroid of tension reinforcement, in. (mm)
- $E_m$ = modulus of elasticity of masonry in compression, psi (MPa)
- $E_s$ = modulus of elasticity of steel, psi (MPa)
- $e_m$ = eccentricity of axial load, in. (mm)
\( F_m \) = allowable compressive stress, psi (MPa)
\( f_m \) = calculated compressive stress in masonry due to axial and flexure, psi (MPa)
\( F_v \) = allowable shear stress, psi (MPa)
\( f_v \) = calculated compressive stress in masonry due to flexure, psi (MPa)
\( F_s \) = allowable tensile or compressive stress in reinforcement, psi (MPa)
\( f_a \) = calculated compressive stress in masonry due to axial load only, psi (MPa)
\( f'_m \) = specified compressive strength of clay masonry or concrete masonry, psi (MPa)
\( f_r \) = modulus of rupture, psi (MPa)
\( f_s \) = calculated tensile or compressive stress in reinforcement, psi (MPa)
\( f_y \) = calculated shear stress in masonry, psi (MPa)
\( f_y \) = specified yield strength of steel for reinforcement and anchors, psi (MPa)
\( h \) = effective height of wall, in. (mm)
\( I_{cr} \) = moment of inertia of cracked cross-sectional area of a member, in.\(^4\) (mm\(^4\))
\( I_g \) = moment of inertia of gross cross-sectional area of a member, in.\(^4\) (mm\(^4\))
\( j \) = ratio of distance between centroid of flexural compressive forces and centroid of tensile forces to depth, \( d \)
\( k \) = ratio of the distance between the compression face of an element and the neutral axis to the effective depth \( d \)
\( M \) = maximum moment at the section under consideration, in.-lb (N-mm)
\( M_{cr} \) = nominal cracking moment strength, in.-lb (N-mm)
\( M_{ser} \) = service moment at midheight of a member, in.-lb (N-mm)
\( M_u \) = factored moment, magnified by second-order effects where required by the code, in.-lb (N-mm)
\( n \) = modular ratio, \( E_s/E_m \)
\( P \) = axial load, lb (N)
\( P_{DL} \) = axial load due to dead load, lb (N)
\( P_u \) = factored axial load, lb (N)
\( r \) = radius of gyration, in. (mm)
\( S_g \) = section modulus of the gross cross-sectional area of a member, in.\(^3\) (mm\(^3\))
\( T \) = resultant tensile force, lb (N)
\( t \) = nominal thickness of member, in. (mm)
\( t_{face} \) = specified thickness of masonry unit faceshell, in. (mm)
\( t_{rib} \) = specified thickness of diaphragm wall rib, in. (mm)
\( t_{sp} \) = specified thickness of member, in. (mm)
\( t_{wall} \) = specified thickness of wall, in. (mm)
\( t_{wythe} \) = specified thickness of the masonry wythe, in. (mm)
\( V \) = shear force, lb (N)
\( V_{rib} \) = shear capacity (resisting shear) of diaphragm wall rib, lb (N)
\( V_{ser} \) = service level shear force, lb (N)
\( V_u \) = factored shear force, lb (N)
$W =$ wind load, psf (kPa)
$\gamma_g =$ grouted shear wall factor
$\delta =$ moment magnification factor
$\varepsilon_m =$ compressive strain of masonry
$\varepsilon_s =$ strain of steel
$f =$ strength reduction factor
$\rho =$ reinforcement ratio

References


17. Concrete Masonry Wall Weights, TEK 14-13B. National Concrete Masonry Association, 2008

Keywords

allowable stress design  crosswalls  design example  diaphragm

diaphragm walls  reinforced masonry  ribs  tall walls  void