**Technical Notes 45A - Brick Masonry Noise Barrier Walls - Structural Design**
**April 1992**

**Abstract:** Rationally designed brick masonry noise barrier walls provide an attractive wall form with reliable structural function. This *Technical Notes* addresses the structural design of pier and panel, pilaster and panel, and cantilever brick noise harrier walls. Suggested design methodology and design examples are provided. The information presented in this *Technical Notes* can be applied with slight modifications to the many design schemes and loading demands of noise barrier walls. The result is an attractive noise barrier wall with the durability and versatility inherent in brick masonry structures.

**Key Words:** brick, cantilever, noise barrier, pier, pilaster, structural design, wall system.

**INTRODUCTION**

*Technical Notes* 45 presented an introduction to acoustical, visual, structural, and construction considerations for brick masonry noise barrier walls. In continuation of the series, this *Technical Notes* addresses structural design considerations in greater detail and provides design examples. Recommended procedures are presented on the structural design of the wall system assuming acoustic and visual considerations are previously addressed. Design recommendations for noise barrier wall footings and caissons have not been addressed in this *Technical Notes.*

A design approach is presented which follows criteria contained in the *Building Code Requirements for Masonry Structures* (ACI 530/ASCE 5/TMS 402-92) and, where applicable, the *Load and Resistance Factor Design Manual of Steel Construction-First Edition* (AISC LRFD). Refer to *Technical Notes* 3 series for a discussion of the ACI/ASCE/TMS document.

**NOTATION[1]**

$A_s$ Area of steel, in.$^2$

$b$ Width of section, in.

$c$ Distance from extreme compression fiber to the neutral axis of the cross section, in.

$d$ Distance from extreme compression fiber to the centroid of tension reinforcement, in.

$E_m$ Modulus of elasticity of masonry in compression, psi

$E_s$ Modulus of elasticity of steel, psi

$f_a$ Calculated compressive stress in masonry due to axial load only, psi

$f_{cr}$ Modulus of rupture, psi

$f_b$ Calculated compressive stress in masonry due to flexure only, psi

$F_b$ Allowable compressive stress in masonry due to flexure only, psi

$f_m$ Compressive stress in masonry, psi

$f'_m$ Specified compressive strength of masonry, psi

$f_s$ Calculated tensile or compressive stress in reinforcement, psi

$F_s$ Allowable tensile or compressive stress in reinforcement, psi

$f_t$ Calculated tensile stress in masonry, psi

$F_t$ Allowable flexural tensile stress in masonry, psi

$f_v$ Calculated shear stress in masonry, psi

$F_v$ Allowable shear stress in masonry, psi

$f_y$ Specified yield stress for reinforcement, psi

$h$ Height of wall or panel, ft

$I_{cr}$ Moment of inertia of cracked masonry cross section, in.4

$I_e$ Effective moment of inertia, in.4

$I_g$ Moment of inertia of uncracked masonry cross section, in.4

$I_x$ Moment of inertia of steel pier about the strong axis, in.4

$j$ Ratio of distance between centroid of flexural compressive forces and centroid of tensile forces to depth

$k$ Ratio of distance between compression face and neutral axis to distance between compression face and centroid of tensile forces

$M$ Design moment, ft-lb

$M_n$ Nominal moment strength, ft-lb

$M_o$ Overturning moment, ft-lb

$M_{px}$ Moment due to spanning between piers, ft-lb

$M_{py}$ Moment induced by $M_{px}$ due to plate effects, ft-lb

$M_r$ Resisting moment, ft-lb

$M_u$ Ultimate moment strength, ft-lb

$M_w$ Moment due to spanning between caissons, ft-lb

$n$ Elastic moduli ratio, $E_s/E_m$

$P$ Design axial load, lb

$p$ Reinforcement ratio, $A_s/bd$

r Radius of gyration, in.

t Thickness of wall or panel, in.

V Design shear force, lb

$V_n$ Nominal shear strength, lb

$V_{px}$ Shear due to spanning between piers, lb

$V_u$ Ultimate shear strength, lb

W Lateral load, lb/ft

x Width of footing, ft

y Depth of footing, ft

$\Delta$ Deflection, in.

$\phi_b$ Resistance factor

[1]Metric equivalents:

1 in. = 25.4 mm

1 ft = 0.3048 m

1 lb = 4.448 N

1 psi = 0.006895 N/mm$^2$

## SELECTION OF A WALL SYSTEM

As discussed in *Technical Notes* 45, there are three typical brick masonry noise barrier wall systems: cantilever walls, pier and panel walls, and pilaster and panel walls. Preliminary consideration of design parameters can help select the wall system that is most appropriate and efficient without having to develop and compare three separate designs.

### Cantilever Noise Barrier Walls

A cantilever wall system is better suited for shorter noise barriers, i.e. walls that are 12 ft (3.7 m) or less in height. In most instances, taller cantilever walls are less desirable because strip footings become too massive. Cantilever walls are more efficient for shorter heights because they are likely the easiest and most economical to construct, and will require the least quality control and inspection. This is because construction techniques used are similar to building wall construction familiar to mason contractors.
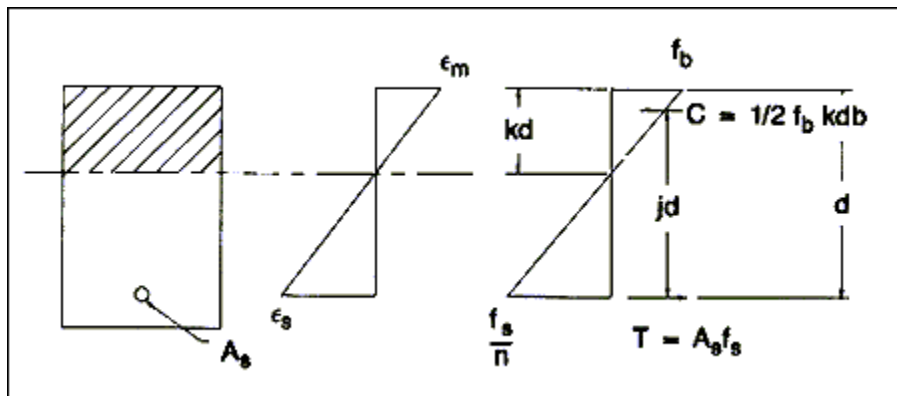
### Pier and Panel Noise Barrier Walls

Pier and panel wall systems are best for quick site erection. Brick panels can be prefabricated on or off site, or laid in place. Also, pier caissons are typically constructed faster and require less concrete than strip footings. Strip footings under the panel are not required, as the panel can span from pier to pier. Material costs for pier and panel wall systems will typically be the least of the three systems. Disadvantages of the pier and panel system include increased construction supervision and inspection, tight construction tolerances for pier-to-panel connections, and increased costs to install the panels.

**Pilaster and Panel Noise Barrier Walls**

Pilaster and panel walls, like pier and panel walls, typically utilize caissons for quick foundation construction. Wall construction is done on site, as the panel is built integral with the pilaster. This requires panel support between caissons during construction. Supervision and inspection are required to ensure proper construction. However, construction tolerances are more liberal than those for pier and panel systems. Generally, pilaster and panel wall systems permit longer pier spacing and taller wall height. A pilaster and panel wall assembly is structurally more efficient than a pier and panel wall assembly, as a fixed condition may be developed at the pilaster-panel connection.

**DESIGN ASSUMPTIONS**

It has been widely accepted that masonry stress-strain behavior is similar to that of concrete. Thus, design assumptions made for masonry under working stress and strength conditions are analogous to assumptions made in concrete design. Figure 1 depicts the assumed stress-strain relationship for masonry in flexure under working loads. In all cases, the principles of equilibrium and compatibility of strains of masonry materials are assumed to apply. Assumptions made following a working stress design are as follows: 1) plane sections before bending remain plane after bending, 2) moduli of elasticity of masonry and steel remain constant, 3) reinforcement is completely bonded to masonry, and 4) in cracked masonry members, the tensile capacity of masonry is neglected.



**Straight Line Stress Distribution**

**FIG. 1**

In this *Technical Notes,* a number of additional assumptions will be made to facilitate design. It is assumed that the brick masonry will be reinforced. Most noise barrier wall applications demand tall slender walls to meet

acoustic requirements and minimize material costs and land use. Reinforcing is required for brick masonry to meet these criteria. Additional assumptions placed on both material properties and wall behavior are as follows.

## Material Properties

Grout and concrete are assumed to have compressive strength equal to or greater than the masonry compressive strength, and elastic moduli of the masonry and the grout are assumed to be equal. The method of transformation of areas may be used in lieu of these assumptions.

## Wall Behavior

Masonry walls are plate structures. Thus, a masonry wall loaded perpendicular to its plane will experience strain along its length and its height. However, the traditional masonry wall design approach is to use the strip method. In this method, a one foot wide section of wall is designed considering one span direction. Strains perpendicular to the strip span direction are ignored. For cantilever walls, this method is nearly exact, as plate effects are negligible. Pier and panel and pilaster and panel walls, however, exhibit wall behavior which can make plate effects significant. This does not mean a rigorous plate analysis is necessary for these walls. Rather, a few simple observations and assumptions can be made to simplify design.

For pier and panel and pilaster and panel wall systems, the panel is subject to three different deflection conditions: a horizontal simple span between piers or pilasters subject to wind or seismic load, a horizontal simple span between caissons subject to panel weight, and a vertical cantilever span subject to deflection of the pier or pilaster. If the panel is free to deflect both in-plane and out-of-plane, the moment due to simple spanning between piers or pilasters, $M_{px}$, and the moment due to simple spanning between caissons, $M_w$, are combined by vector addition to calculate the maximum design moment for the horizontal span of the panel. However, the panel must be flush with the ground to avoid noise penetration under the wall. Thus, the panel may, in fact, be supported along its entire length by the ground. This is significant, because the design moment in this case is solely $M_{px}$, the moment about the weak axis of the panel. This condition will require the most amount of horizontal reinforcement for the panel. In the panel design examples that follow it is assumed that the ground supports the entire length of the panel.

Because of plate effects, $M_{px}$ will induce moment about the horizontal axis as well, denoted as $M_{py}$. The strip solution does not and cannot calculate $M_{py}$, as plate effects are ignored in this method. However, plate analysis shows that $M_{py}$ can be significant and that the ratio of $M_{py}$ to $M_{px}$ increases as the height to length ratio of the panel increases. As an approximation, $M_{py}$ is calculated as one tenth the height to length ratio times $M_{px}$. $M_{py}$ is a maximum at the middle of the panel. However, moment due to vertical cantilever deflection of the wall is a maximum at the bottom of the panel. Thus, the design moment about the horizontal axis is the greater of: 1) the moment due to vertical cantilever deflection at the bottom of the panel, or 2) the sum of $M_{py}$ and the moment due to vertical cantilever deflection at the middle of the panel.

Vertical cantilever deflection of the panel is a function of the rigidity of the piers or pilasters. If the piers or pilasters are very rigid, cantilever deflection of the panel will be negligible. However, optimal flexural design may result in less rigid piers or pilasters with considerable deflection, especially when steel piers are used. Induced tensile stresses in the panel must be within allowable tensile stresses for unreinforced masonry if the panel cannot be reinforced in the vertical direction. Thus, deflection criteria will often govern pier and pilaster design.

Reinforced brick masonry pilaster and panel and pier and panel wall systems are typically very rigid, so deflections in many cases will be small. However, the deflection of the pilaster must be calculated considering the ratio of applied moment to cracking moment. Cracking moment is calculated using the gross moment of inertia of the pier or pilaster.

In the pilaster and panel design example that follows, a two span continuous panel is assumed. Thus, the pilaster panel interface is assumed to be a fixed connection for the middle pilaster, and a simple connection for the two

exterior supports. This allows for expansion joints at the simple supports to accommodate horizontal expansion of the panels.

Lastly, compression steel in the pilaster is usually ignored in design. If consideration of the increased compressive strength due to the compression steel is made, the steel must be properly confined within the pilaster with lateral or spiral ties.

**DESIGN PROCEDURE**

It is important to establish a set design procedure to ensure an accurate and comprehensive noise barrier wall design. The following nine steps are presented as a guide to the structural design of a brick masonry noise barrier wall. Additional criteria may be warranted for a particular wall design scheme.

1) Determine required wall height based upon acoustical considerations.

2) Determine critical lateral and axial load combinations on wall elements. Loads should be determined according to the recommendations of the local building code or as contained in the document *Minimum Design Loads for Buildings and Other Structures* (ASCE 7). For the examples that follow, inertial wall force due to seismic base shear is divided by wall surface area for comparison with wind loads.

3) To determine required reinforcement, assume j = 0.9:

$$A_s \text{ req'd} = M/F_s jd$$

4) Calculate masonry compressive stresses and the steel tensile stress:

$$f_b = 2M/jkbd^2$$

$$f_a = P/bkd$$

$$f_s = M/A_s jd$$

5) Check the allowable compressive stress in masonry and the tensile stress in steel (Table 1). Axial compression and buckling seldom govern design of noise barrier wall elements. However, axial compression must be included to calculate the maximum flexural compression. Note that allowable stresses for wind or seismic load conditions may be increased by one-third over those given in Table 1.

## TABLE 1
### Summary of ACI 530/ASCE 5/TMS
### 402 Allowable Stresses[1]

**Allowable Flexural Compressive Stress**

$F_b + 1/3f'_m$

**Allowable Shear Stress for Flexural Members**

Where reinforcement is not provided to resist entire shear:

$F_v = \sqrt{f'_m}$          not to exceed 50 psi

where reinforcement is provided to resist entire shear:

$F_v = 3.0\sqrt{f'_m}$          not to exceed 150 psi

**Allowable Steel Stress**

| | |
|---|---|
| Grade 40 or 50 reinforcement | $F_s = 20$ ksi |
| Grade 60 reinforcement | $F_s = 24$ ksi |
| Wire reinforcement | $F_s = 30$ ksi |

[1]Allowable stresses for wind and seismic loading conditions may be increased by one-third.
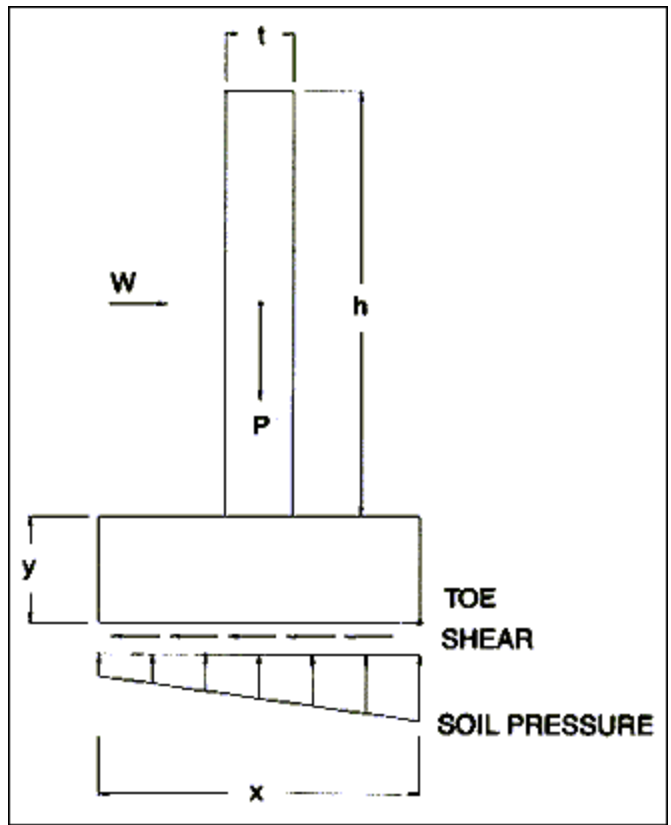
6) Calculate shear stress:

$f_v = V/bjd$

7) Check the allowable shear stress in masonry (Table 1). If exceeded, member must be reinforced for shear and the shear stress checked.

8) Design the pier or pilaster, if applicable. If the pier or pilaster is made of reinforced masonry, design will follow steps 3 through 7. If a steel pier is used, follow the design recommendations given in the *Load and Resistance Factor Design Manual of Steel Construction.* Flexural tensile stresses developed in unreinforced masonry panels due to pier or pilaster deflection may not exceed the allowable flexural tensile stresses given in Table 2.

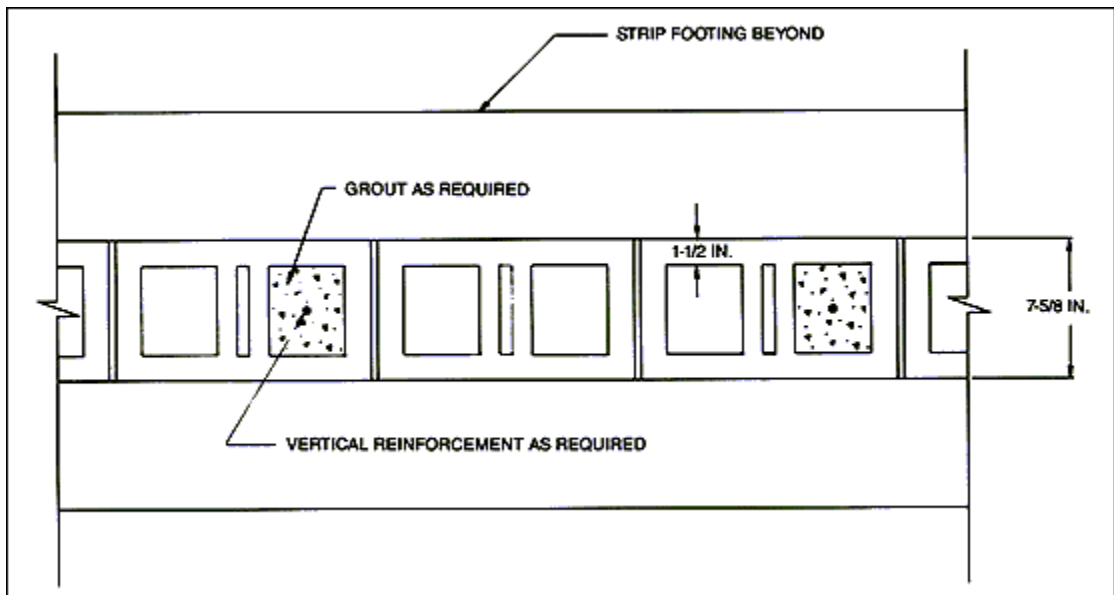[1]Allowable stresses for wind and seismic loading conditions may be increased by one-third.

[2]For partially grouted masonry allowable stresses shall be determined on the basis of linear interpolation between hollow units which are fully grouted or ungrouted and hollow units based on amount of grouting.

9) Calculate overturning and resisting moments, and sliding resistance (Fig.2). These are functions of the wall, footing, and caisson dimensions, as well as the soil pressure resistance. The factor of safety is the ratio of resisting moment, $M_r$, to the overturning moment, $M_o$. A factor of safety of 2 or greater is recommended.

**Wall/Footing Forces**

**FIG. 2**



**Hollow Brick Cantilever Wall**

**FIG. 3**

**DESIGN EXAMPLES**

**Design Example #1: Hollow Brick Cantilever Noise Barrier Wall**

Type S portland cement/lime mortar, $f'_m$ = 3600 psi,
$E_m$ = 3.0 x 10$^6$ psi, n = 9.7

Wall dimensions shown in Fig. 3, running bond, face shell bedding

Grade 60 reinforcement, $E_s = 29 \times 10^6 \, psi, \; n = 9.7$

Loads: wind = 20 psf, wall weight =73 psi

**Step 1:** Based on acoustical considerations, the wall height shall be 10 ft minimum.

**Step 2:** Critical load combinations result in the following design values (per ft of wall):

$$M = (0.5)(W)(h)^2 = (0.5)(20 \text{ psf}) (10\text{ft})^2 = 1000 \text{ ft-ob}$$

$$V = (W)(h) = (20 \text{ psf})(10\text{ft}) = 200 \text{ lb}$$

$$P = (\text{wall weight})(h) = (73 \text{ psf})(10 \text{ ft}) = 730 \text{ lb}$$

**Step 3**: Calculate required reinforcement.

$$A_{s \, req'd} = \frac{M}{F_s jd} = \frac{1000 ft - lb(12 in./ft)}{(4/3)((24 \, ksi)(0.9)(7.63 in./2)} = 0.109 in.^2$$

Try #5 bars @32 in. o.c., per foot of wall (Table 3).

$$pn = A_s n = \frac{(0.115 \text{ in.}^2)(9.7)}{bd \quad (12 \text{ in.})(7.63 \text{ in.}/2)} = 0.0244$$

$$k = \sqrt{(pn)^2 + 2pn} - pn$$
$$= \sqrt{(0.0244)2 + 2(0.0244)} - (0.0244) = 0.198$$
$$j = 1-k/3 = 0.934$$
$$kd = (0.198)(7.63 \text{ in.}/2) = 0.755 \text{ in.} < 1.5 \text{ in., within}$$
face shell, OK

**Step 4:** Calculate the masonry compressive stress and the steel tensile stress.

$$f_b = \frac{2M}{jkbd^2} = \frac{2(1000 \text{ ft-lb})(12 \text{ in./ft})}{(0.934)(0.198)(12 \text{ in.})(7.63 \text{ in.}/2)^2}$$
$$= 744 \text{ psi}$$

$$f_a = \frac{P}{bkd} = \frac{730 \text{ lb}}{(12 \text{ in.})(0.198)(7.63 \text{ in.}/2)} = 81 \text{ psi}$$

$$f_s = \frac{M}{A_s jd} = \frac{1000 \text{ ft-lb} (12 \text{ in./ft})}{(0.115 \text{ in.}^5)(0.934)(7.63 \text{ in.}/2)}$$
$$= 29.3 \text{ ksi}$$

[1]Area of steel listed is for one wire.

**Step 5:** Check compressive stress in masonry and the tensile stress in steel.

$$F_b = 4/3(1/3f'_m) = 1600 \text{ psi}$$
$$f_m = f_a + f_b = 81 \text{ psi} + 744 \text{ psi}$$
$$= 825 \text{ psi} < 1600 \text{ psi OK}$$

$$F_s = 4/3(24 \text{ ksi}) = 32 \text{ ksi} > 29.3 \text{ ksi OK}$$

**Step 6:** Calculate shear stress.

$$f_v = \frac{V}{bjd} = \frac{200 \text{ lb}}{(12 \text{ in})(0.934)(7.63 \text{ in.}/2)} = 4.7 \text{ psi}$$

**Step 7:** Check shear stress.

$$F_v = 4/3 \sqrt{f'_m} = 80 \text{ psi}, \ 4/3 (50 \text{ psi}) = 67 \text{ psi maximum}$$
Thus, $F_v = 67 \text{ psi} > 4.7 \text{ psi OK}$

**Step 8:** Does not apply.

**Step 9:** Neglect the soil pressure resistance to overturning. Calculate overturning and resisting moments, and the safety factor on overturning (Fig. 2). Note: Sliding must also be considered. Sliding is a function of the soil conditions, and is beyond the scope of this *Technical Notes.*

$M_o$ = (W)(h)(moment arm about toe of footing)

     = (20 psf)(10 ft)(5 ft +y)

$M_r$ = [P + (footing unit weight)(xy)](x/2)

     = [730 lb + (150 pcf)(xy)] (x/2)

Try a 2 ft x 4 ft footing

$M_o$ = 1400 ft-lb     $M_r$ = 3860 ft-lb

$$F.S. = \frac{M_r}{M_o} = 2.8 > 2 \underline{OK}$$

**Design Example #2: Steel Pier with 4 Inch Panel Noise Barrier Wall**

Type S portland cement/lime mortar, $f'_m$ = 3000 psi,
$E_m$ = 2.8 x $10^6$ psi
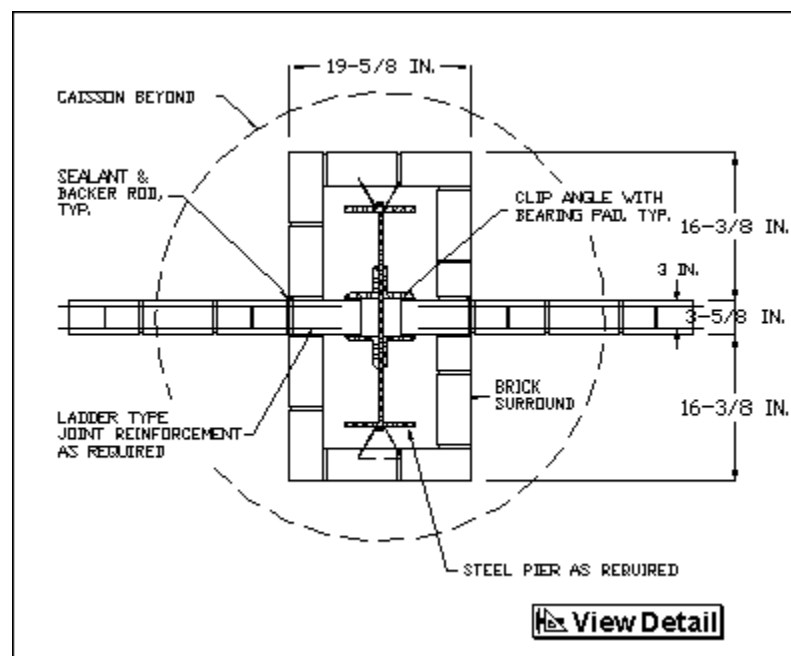
Wall dimensions shown in Fig. 4

Running bond, full bedding, spacing of piers shall be 15ft

Ladder type joint reinforcement: $f_y = 80ksi$, $E_s = 29 \times 106psi$, $n = 10.4$

Pier I-beam: Grade 50 W Section,

Loads: wind = 15psf, panel weight = 40psf, seismic = 25psf

# Noise Barrier Wall Design Example #2

## Steel Pier With 4 Inch Panel

**FIG. 4**

**Step 1:** Based on acoustical considerations, the wall height shall be 16 ft minimum.

**Step 2:** Critical load combinations result in the following design values:

a) For the panel span between piers (per foot of wall):

$$M_{px} = (1/8)(W)(L)^2 = (1/8)(25 \text{ psf}) < 15 \text{ ft}) = 703 \text{ ft-lb}$$

$$m_{py} = (1/10)(h/L)(M_{px}) = (1/10)(16 \text{ ft}/15 \text{ ft}) (703 \text{ ft-lb}) = 75 \text{ ft-lb}$$

$$V_{px} = (0.5)(25 \text{ psf})(15 \text{ ft}) = 188 \text{ lb}$$

b) For the panel span between caissons, assume the panel is supported along its entire base by the ground.

c) For an interior steel pier:

$$M = (0.5)(2)(V_{px})(h)^2 = (0.5)(2)(188 lb / ft)(16 ft)^2 = 48,130 \, ft{-}lb$$

$$M_u = 1.3M = 1.3(48,130 \, ft - lb) = 62,570 \, ft - lb$$

$$V = (2)(V_{px})(h) = (2)(188 \text{ lb/ft})(16 \text{ ft}) = 6016 \text{ lb}$$

$$V_u = 1.3V = 1.3 (6016 \text{ lb}) = 7820 \text{ lb}$$

**Step 3:** Calculate required reinforcement for the panel for the design moment $M_{px}$. Assume the distance from the extreme compression face to the joint reinforcement is 3 inches.

$$A_{s\,reqd} = \frac{M_{px}}{F_s jd} = \frac{703 \text{ ft-lb (12 in/ft)}}{(4/3)(30 \text{ ksi})(0.9)(3 \text{ in})}$$

Try ladder type W1.7 wire reinforcement in every course, with a 2.25 in. high brick.

Therefore, $A_s = 0.077 \text{ in.}^2$ per foot of panel (Table 4).

$$pn = \frac{A_s n}{bd} = \frac{(0.077 \text{ in.}^2)(10.4)}{(12 \text{ in.})(3 \text{ in.})} = 0.0222$$

$$k = \sqrt{(pn)^2 + 2pn} - pn$$
$$= \sqrt{(0.0222)2 + 2(0.0222)} - (0.0222) = 0.19$$

$$j = 1 - k/3 = 0.936$$

## TABLE 4
### Area of Steel Per Vertical Foot of Wall (in.$^2$)[1]

| Joint Reinforcement | | Brick Height | | |
|---|---|---|---|---|
| Size | Course | 2.25 in. | 2.75 in. | 3.63 in. |
| W1.1 | every | 0.050 | 0.041 | 0.033 |
| | alternate | 0.025 | 0.021 | 0.017 |
| | third | 0.017 | 0.014 | 0.011 |
| W1.7 | every | 0.077 | 0.064 | 0.051 |
| | alternate | 0.038 | 0.032 | 0.026 |
| | third | 0.026 | 0.021 | 0.017 |
| W2.1 | every | 0.095 | 0.079 | 0.063 |
| | alternate | 0.047 | 0.039 | 0.032 |
| | third | 0.032 | 0.026 | 0.021 |
| W2.8 | every | 0.126 | 0.110 | 0.084 |
| | alternate | 0.063 | 0.053 | 0.042 |
| | third | 0.042 | 0.035 | 0.028 |
| W4.9 | every | 0.221 | 0.184 | 0.147 |
| | alternate | 0.110 | 0.092 | 0.074 |
| | third | 0.074 | 0.061 | 0.049 |

[1]Area of steel listed is for one wire.

**Step 4:** Calculate the masonry compressive stress and the steel tensile stress.

$$f_b = \frac{2M_{bx}}{jkbd^2} = \frac{2\,(703\ \text{ft-lb})(12\ \text{in/ft})}{(0.936)(0.19)(12\ \text{in.})(3\ \text{in.})^2}$$
$$= 878\ \text{psi}$$

$$f_s = \frac{M_{bx}}{A_s jd} = \frac{(703\ \text{ft-lb})(12\ \text{in/ft})}{(0.077\ \text{in.}^2)(0.936)(3\text{in.})}$$
$$= 39\ \text{ksi}$$

**Step 5:** Check compressive stress in masonry and the tensile stress in steel.

$$F_b = 4/3(1/3\ f_m') = 1333\ \text{psi} > 878\ \text{psi}\ \underline{OK}$$

$$F_s = 4/3\ (30\ \text{ksi}) = 40\ \text{ksi} > 39\ \text{ksi}\ \underline{OK}$$

**Step 6**: Calculate shear stress.

$$f_v = \frac{V_{ex}}{bjd} = \frac{188\ \text{lb}}{(12\ \text{in.})(0.936)(3\ \text{in.})} = 6\ \text{psi}$$

**Step 7:** Check shear stress.

$$F_v = 4/3\ \sqrt{f_m'} = 73\ \text{psi},\ 4/3\ (50\ \text{psi}) = 67\ \text{psi maximum}$$
Thus, $F_v = 67\ \text{psi} > 6\ \text{psi}\ \underline{OK}$

**Step 8:** Assume the pier design will be governed by deflection limitations of the panel, not the required flexural strength of the pier. Calculate the maximum vertical cantilever deflection the panel may undergo without exceeding the allowable tensile stress value for the panel found in Table 2. From Table 2:

$F_t = 4/3 \ (40 \ psi) = 53 \ psi$

The maximum moment that may be applied to the panel within the allowable tensile stress is calculated as follows:

$$M_{max} = \frac{F_t I_g}{c} = \frac{(53 \ psi)(1/12)(15 \ ft)(3.63 \ in.)^3}{(3.63 \ in./2)}$$

$$= 1740 \ ft\text{-}lb$$

Calculate the maximum load per vertical foot, $w_{max}$, which may be applied to the panel without exceeding $M_{max}$ in the panel. Two locations must be considered: 1) the bottom of the panel where the moment due to vertical cantilever deflection is a maximum and 2) the middle of the panel where the combination of $M_{by}$ and the moment due to vertical cantilever deflection is a maximum.

1) $w_{max} = \dfrac{2M_{max}}{L^2} = \dfrac{2(1740 \ ft\text{-}lb)}{(16 \ ft)^2} = 13.6 \ lb/ft$

2) $w_{max} = \dfrac{8(M_{max} - M_{by})}{L^2} = \dfrac{8(1740 \ ft\text{-}lb - 75 \ ft\text{-}lb)}{(16 \ ft)^2}$

Therefore, $w_{max} = 13.6 \ lb/ft$. Calculate the corresponding deflection of the panel.

$\Delta_{max} = \dfrac{w_{max} L^4}{8 I_g E_m} = \dfrac{(13.6 \ lb/ft)(16 \ ft)^4 (144 \ in.^2/ft^2)}{8[1/12(15 \ ft)(3.63 \ in)^3](2.8 \times 10^6 \ psi)}$

$= 0.096 \ in.$

Based on $\Delta_{max}$ and the out-of-plane load on the pier, $2V_{pc}$, calculate the minimum required moment of inertia of the pier.

$I_{reqd} = \dfrac{2V_{pc}L^4}{8E_s \Delta_{max}} = \dfrac{2(188lb/ft)(16 \ ft)^4 (1728 \ in.3/ft^3)}{8 (29 \times 10^6 \ psi)(0.096 \ in.)}$

$= 1912 \ in.^4$

Try a W 24 X 76, $I_x = 2100 \ in.^4 > 1912 \ in.^4$  OK

Check the moment strength of the pier:
$\phi_b M_n = 528 \ ft\text{-}kip > M_u = 62.6 \ ft\text{-}kip$  OK

Check shear strength of the pier:
$\phi_v V_n = (0.9)(0.6)(f_y)(A_{web})$
$= (0.9)(0.6)(50 \ ksi)(21 \ in)(0.44 \ in) = 249 \ kips$

**Step 9:** Sliding and overturning resistance of the caisson is a function of the lateral soil pressure and is beyond the scope of this *Technical Notes.*

**Design Example #3: Reinforced Brick Masonry Pier and 4 Inch Panel Noise Barrier Wall**

Type S portland cement/lime mortar, f'$_m$ = 3500 psi,

E$_n$ = 3.0 X10$^6$ psi

Wall dimensions shown in Fig. 5, running bond, full bedding

W1.7 ladder type joint reinforcement in every third course, brick height is 2.25 in.

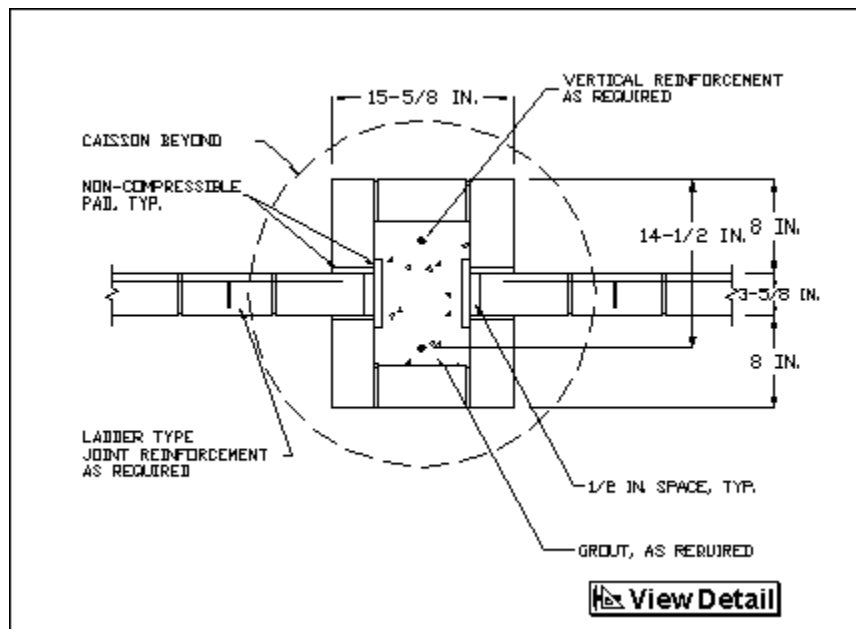Joint reinforcement: f$_y$ = 80 ksi, E$_s$ = 29 X 10$^6$ psi, n = 9.7

Pier reinforcement: f$_y$ = 60 ksi, E$_s$ = 29 X 10$^6$ psi, n = 9.7

Loads: wind = 20psf, seismic = 16 psf, panel weight = 40 psf, pier weight = 120 pcf

The maximum permissible span between piers will be used based on the given amount of joint reinforcement.



**Noise Barrier Wail Design Example #3**

**Reinforced Brick Pier and 4 Inch Panel**

**FIG. 5**

**Step 1:** Based on acoustical considerations, the wall height shall be 10 ft minimum.

**Step 2:** Critical load combinations result in the following design values:

a) For the panel span between piers (L is in feet)(per foot of panel):

$M_{px} = (1/8)(W)(L)^2 = (1/8)(20 \text{ psf})L^2 = 2.5 L^2$

$V_{px} = (0.5)(W)(L) = (0.5)(20 \text{ psf})L = 10L$

b) For the panel span between caissons, assume the panel is supported along its entire base by the ground.

c) For an interior brick masonry pier:

$M = (0.5)(2)(V_{px})(h)^2 = (0.5)(2)(10L)(10 \text{ ft})^2 = 1000 L$

$V = (2)(V_{px})(h) = (2)(10 L)(10 \text{ ft}) = 200 L$

$P = (\text{pier weight})(b)(t)(h)$
$= (120 \text{ pcf})(15.63 \text{ in.})(19.63 \text{ in.})(10 \text{ ft}) = 2557 \text{ lb}$

**Step 3:** Calculate the maximum permissible spacing of piers with the given joint reinforcement based on the design moment, $M_{px}$. Assume the distance from the extreme compression face to the joint reinforcement is 3 inches.

$A_s = 0.026 \text{ in.}^2$ per foot of panel (Table 4).

$pn = \dfrac{A_s n}{bd} = \dfrac{(0.026 \text{ in.}^2)(9.7)}{(12 \text{ in})(3 \text{ in})} = 0.007$

$k = \sqrt{(pn)^2 + 2pn} - pn$
$= \sqrt{(0.007)^2 + 2(0.007)} - (0.007) = 0.111$

$j = 1 - k/3 = 0.963$

$M_{px} = A_s F_s jd$

$= (0.026 \text{ in.}^2)(4/3)(30 \text{ ksi})(0.963)(3\text{in.}/12 \text{ in./ft})$
$= 249 \text{ ft-lb}$

Set the maximum allowable moment equal to the applied moment to calculate the maximum permissible pier spacing.

$M_{px} = 249 \text{ ft-lb} = 2.5 L^2$

$L_{max} = 9.97 \text{ ft}$, try a pier spacing of 9 ft

$M_{px} = 2.5 (9\text{ft})2 = 203 \text{ ft-lb}$

**Step 4:** Calculate the masonry compressive stress and the steel tensile stress.

$f_b = \dfrac{2M_{px}}{jkbd^2} = \dfrac{2(203 \text{ ft-lb})(12\text{in/ft})}{(0.963)(0.111)(12 \text{ in})(3 \text{ in.})^2} = 422 \text{ psi}$

$f_s = \dfrac{M_{px}}{A_s jd} = \dfrac{(203 \text{ ft-lb})(12 \text{ in/ft})}{(0.026 \text{ in.}^2)(0.963)(3 \text{ in})} = 32.4 \text{ ksi}$

**Step 5:** Check the compressive stress in masonry and the tensile stress in steel.

$F_b = 4/3(1/3 \text{ f}'_m) = 1555 \text{ psi} > 422 \text{ psi}$ OK

$F_s = 4/3(30 \text{ ksi}) = 40 \text{ ksi} > 32.4 \text{ ksi}$ OK

**Step 6:** Calculate shear stress.

$f_v = \dfrac{V_{px}}{bjd} = \dfrac{10(9 \text{ ft})}{(12 \text{ in})(0.963)( 3 \text{ in})} = 3 \text{ psi}$

**Step 7:** Check shear stress.

$F_v = 4/3 \sqrt{f'_m} = 79$ psi, $4/3$ (50 psi) $= 67$ psi maximum

Thus, $F_v = 67$ psi $> 3$ psi OK

**Step 8:** Design the reinforced brick masonry pier. Repeat steps 3 through 7 with the following design values:

M = 1000 L = 1000 (9 ft) = 9000 ft-lb
V = 200 L = 200 (9 ft) = 1800 lb

**Step 3a:** Calculate required pier reinforcement.

$$A_{s\,reqd} = \frac{M}{F_s j d} = \frac{(9000 \text{ ft-lb})(12 \text{ in/ft})}{(4/3)(24 \text{ ksi})(0.9)(14.5 \text{ in})}$$

$$= 0.259 \text{ in.}^2$$

Try 2 #5 bars, per bar (Table 3). Ignore compression steel.

$$pn = \frac{A_s n}{bd} = \frac{(0.307 \text{ in}^2)(9.7)}{(15.63 \text{ in.})(14.5 \text{ in.})} = 0.013$$

$$k = \sqrt{(pn)^2 + pn} - pn$$
$$= \sqrt{(0.013)^2 + 2(0.013)} - (0.013) = 0.149$$
$$j = 1 - k/3 = 0.95$$

**Step 4a:** Calculate the masonry compressive stress and the steel tensile stress.

$$f_b = \frac{2M}{jkbd^2} = \frac{2(9000 \text{ ft-lb})(12 \text{ in/ft})}{(0.95)(0.149)(15.63 \text{ in})(14.5 \text{ in})^2}$$

$$= 458 \text{ psi}$$

$$f_a = \frac{P}{bkd} = \frac{2557 \text{ lb}}{(15.63 \text{ in})(0.149)(14.5 \text{ in})} = 75 \text{ psi}$$

$$f_s = \frac{M}{A_s j d} = \frac{(9000 \text{ ft-lb})(12 \text{ in/ft})}{(0.307 \text{ in}^2)(0.95)(14.5 \text{ in})} = 25.5 \text{ ksi}$$

**Step 5a**: Check compressive stress in masonry and the tensile stress in steel.

$F_b = 4/3(1/3 \text{ } f'_m) = 1555$ psi
$f_m = f_a + f_b = 75$ psi $+ 458$ psi
$\quad = 533$ psi $< 1555$ psi OK

$F_s = 4/3$ (24 ksi) $= 32$ ksi $> 25.5$ ksi OK

**Step 6a:** Calculate shear stress.

$$f_v = \frac{V}{bjd} = \frac{1800 \text{ lb}}{(15.63 \text{ in})(0.95)(14.5 \text{ in})} = 8 \text{ psi}$$

**Step 7a:** Check shear stress.

$F_v = 4/3 \sqrt{f'_m} = 79$ psi, $4/3$ (50 psi) $= 67$ psi maximum
Thus, $F_v = 67$ psi $> 8$ psi OK

With pier designed, check the induced vertical cantilever deflection in the panel. Tensile stresses must be within those given in Table 2 for unreinforced masonry. Determine if the pier is cracked. Assume that the modulus of rupture, $f_o$, for solid brick masonry is equal to $2 \sqrt{f'_m} = 118$ psi (Ref. UBC 91).

$I_s = (1/12)(b)(t_{pier})^3 = (1/12)(15.63 \text{ in})(19.63 \text{ in})^3$

$= 9850$ in$^4$

$M_c = \dfrac{f_o I_s}{c} = \dfrac{(118 \text{ psi})(9850 \text{ in}^4)}{(0.5)(19.63 \text{ in})(12 \text{ in/ft})} = 9870$ ft-lb

$M = 9000$ ft-lb $< 9870$ ft-lb. Thus, the pilaster is not cracked under service loads. Calculate the deflection of the pier based on uncracked section properties:

$\Delta = \dfrac{2V_m h^4}{8E_m I_s} = \dfrac{2(90 \text{ lb/ft})(10 \text{ ft})^4 (1728 \text{ in}^3/\text{ft}^3)}{8(3 \times 10^6 \text{ psi})(9850 \text{ in}^4)}$

$= 0.013$ in.

Calculate the induced moment on the panel assuming the panel must maintain the curvature of the pier:

$W_{panel} = \dfrac{8E_m I_{panel} \Delta}{h^4}$

$= \dfrac{8(3 \times 10^6 \text{ psi})[(1/12)(9 \text{ ft})(3.63 \text{ in})^3](0.013 \text{ in})}{(10 \text{ ft})^4 (144 \text{ in}^2/\text{ft}^2)}$

$= 7.9$ lb/ft

$M_{base} = (0.5)(W_{panel})(h)^2 = (0.5)(7.9 \text{ lb/ft})(10 \text{ ft})^2$

$= 395$ ft-lb

Calculate the flexural tensile stress in the panel at the bottom of the wall.

$f_t = \dfrac{M_{base} c}{I_{panel}} = \dfrac{(395 \text{ ft-lb})(3.63 \text{ in}/2)}{(1/12)(9 \text{ ft})(3.63 \text{ in})^3} = 20$ psi

Check the allowable tensile stress value given in Table 2 :

$F_t = 4/3$ (40 psi) $= 53$ psi $> 20$ psi OK

Finally, the combination of $M_b$ and the moment due to vertical cantilever deflection at the middle of the panel must be checked.

$M_{by} = (1/10)(h/L)(M_{by}) = (1/10)(10 \text{ ft}/9 \text{ ft})(203 \text{ ft-lb})$

$= 23$ ft-lb

$M = M_{by} + (1/8)(W_{panel})(h)^2$

$= 23$ ft-lb $+ (1/8)(7.9 \text{ lb/ft})(10 \text{ ft})^2 = 122$ ft-lb

$M_{base} = 395$ ft-lb $> 122$ ft-lb, therefore, OK. If M is greater than $M_{base}$, use M to calculate $f_t$ and check the allowable tensile stress, $F_t$.

**Step 9:** Sliding and overturning resistance of the caisson is a function of the lateral soil pressure and is beyond the scope of this *Technical Notes.*

**Design Example #4: Reinforced Brick Pilaster and 6 inch Panel Noise Barrier Wall**

Type S portland cement/lime mortar, $f'_m = 3000$ psi,

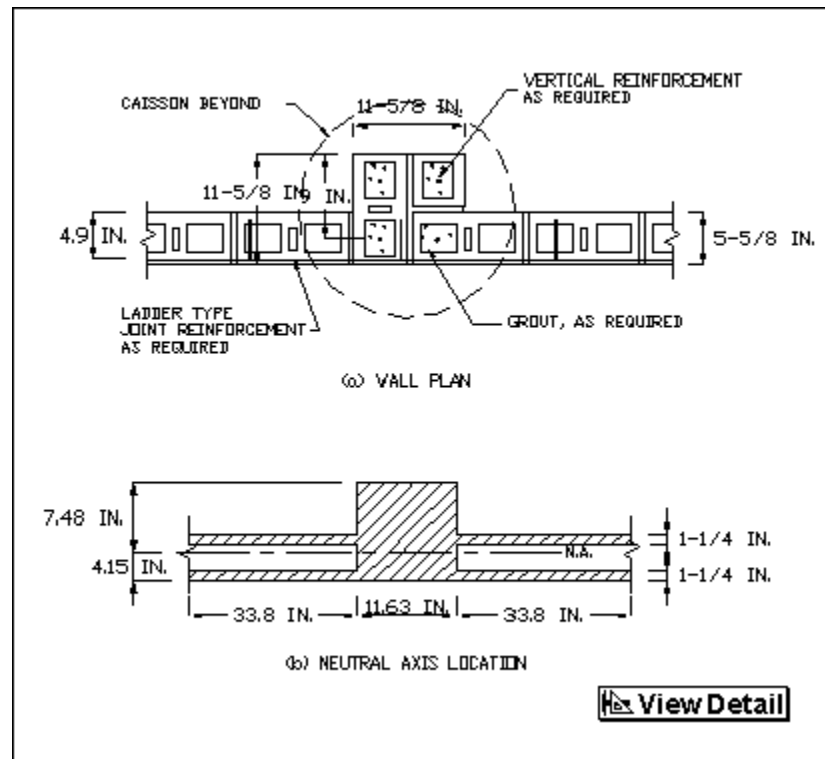$E_m = 2.9 \times 10^6$ psi

Wall dimensions shown in Fig. 6.

Running bond, face shell bedding, spacing of pilasters shall be 10 ft

Ladder type joint reinforcement: $f_y = 80$ ksi,

$E_s = 29 \times 10^6$ psi, n = 10

Pier reinforcement: $f_y = 40$ ksi, $E_s = 29 \times 10^6$ psi, n = 10

Loads: wind = 15 psf, panel weight = 40 psf, pilaster weight = 120 pcf



**Noise Barrier Wall Design Example #4**

**Reinforced Brick Pilaster and 6 inch Panel**

**FIG. 6**

**Step 1:** Based on acoustical considerations, the wall height shall be 6 ft minimum.

**Step 2:** Critical load combinations result in the following design values:

a) For the panel span between piers, assume a two span continuous condition. The critical cross section is at the support or pilaster. Thus, joint reinforcement must be continuous through the pilaster.

$M_{px} = (1/8)(W)(L)^2 = (1/8)(15 \text{ psf})(10 \text{ ft})^2 = 188$ ft-lb/ftof panel

$V_{px} = (0.625)(W)(L) = (0.625)(15 \text{ psf})(10 \text{ ft}) = 94$ lb/ft of panel

b) For the panel span between caissons, consider the panel as entirely supported by the ground.

c) For the pilaster:

$M = (0.625)(W)(L)(h)^2 = (0.625)(15 \text{ psf})(10 \text{ ft})(6 \text{ ft})^2$
$= 3375$ ft-lb

$V = (1.25)(W)(L)(h) = (1.25)(15 \text{ psf})(10 \text{ ft})(6 \text{ ft}) = 1125$ lb

$P = $ (pilaster weight)(b)(t)(h)
$= (120 \text{ pcf})(11.63 \text{ in})(11.63 \text{ in})(6 \text{ ft}) = 676$ lb

**Step 3:** Calculate required reinforcement for the panel for the design moment, $M_{px}$. Assume the distance from the extreme compression face to the joint reinforcement is 4.9 inches.

$$A_{s\,req'd} = \frac{M_{px}}{F_s jd} = \frac{(188 \text{ ft-lb})(12 \text{ in/ft})}{(4/3)(30 \text{ ksi})(0.9)(4.9 \text{ in})}$$
$$= 0.013 \text{ in}^2$$

Try W1.7 ladder type joint reinforcement in every third course, brick height is 3.63 in.

From Table 4, $A_s = 0.017 \text{ in}^2$ per vertical foot of panel.

$$pn = \frac{A_s n}{bd} = \frac{(0.017 \text{ in}^2)(10)}{(12 \text{ in})(4.9 \text{ in})} = 0.003$$

$$k = \sqrt{(pn)^2 + 2pn} - pn$$
$$= \sqrt{(0.003)^2 + 2(0.003)} - (0.003) = 0.073$$
$$j = 1 - k/3 = 0.976$$

$$kd = (0.073)(4.9 \text{ in})$$
$$= 0.36 \text{ in} < 1.25 \text{ in, within face shell, OK}$$

**Step 4:** Calculate the masonry compressive stress and the steel tensile stress.

$$f_b = \frac{2M_{px}}{jkbd^2} = \frac{2(188 \text{ ft-lb})(12 \text{ in/ft})}{(0.976)(0.073)(12 \text{ in})(4.9 \text{ in})^2}$$
$$= 220 \text{ psi}$$

$$f_s = \frac{M_{px}}{A_s jd} = \frac{(188 \text{ ft-lb})(12 \text{ in/ft})}{(0.017 \text{ in}^2)(0.976)(4.9 \text{ in})} = 27.7 \text{ ksi}$$

**Step 5:** Check compressive stress in masonry and the tensile stress in steel.

$F_b = 4/3(1/3 \ f'_m) = 1333 \text{ psi} > 220 \text{ psi OK}$
$F_s = 4/3(30 \text{ ksi}) = 40 \text{ ksi} > 27.7 \text{ ksi OK}$

**Step 6:** Calculate shear stress.

$$f_v = \frac{V_{px}}{bjd} = \frac{94 \text{ lb}}{(12 \text{ in})(0.976)(4.9 \text{ in})} = 2 \text{ psi}$$

**Step 7:** Check shear stress.

$F_v = 4/3 \sqrt{f'_m} = 73$ psi, $4/3$ (50 psi) = 67 psi maximum

Thus, $F_v = 67$ psi > 2 psi OK

**Step 8:** Design the reinforced brick masonry pilaster. Repeat steps 3 through 7.

**Step 3a:** Calculate required pilaster reinforcement.

$$A_{s\,reqd} = \frac{M}{f_s jd} = \frac{(3375 \text{ ft-lb})(12 \text{ in/ft})}{(4/3)(20 \text{ ksi})(0.9)(9 \text{ in})}$$

$$= 0.188 \text{ in}^2$$

Try 4 #3 bars, $A_s = 0.22$ in$^2$ per pair of bars (Table 3).
Ignore compression steel.

$$pn = \frac{A_s n}{bd} = \frac{(0.22 \text{ in}^2)(10)}{(11.63 \text{ in})(9 \text{ in})} = 0.021$$

$$k = \sqrt{(pn)^2 + 2pn} - pn$$

$$= \sqrt{(0.021)^2 + 2(0.021)} - (0.021) = 0.185$$

$$j = 1 - k/3 = 0.938$$

**Step 4a:** Calculate the masonry compressive stress and the steel tensile stress.

$$f_b = \frac{2M}{jkbd^2} = \frac{2(3375 \text{ ft-lb})(12 \text{ in/ft})}{(0.938)(0.185)(11.63\text{in})(9 \text{ in})^2}$$

$$= 496 \text{ psi}$$

$$f_a = \frac{P}{bkd} = \frac{676 \text{ lb}}{(11.63 \text{ in})(0.185)(9 \text{ in})} = 35 \text{ psi}$$

$$f_s = \frac{M}{A_s jd} = \frac{(3375 \text{ ft-lb})(12 \text{ in/ft})}{(0.22 \text{ in}^2)(0.938)(9 \text{ in})} = 21.8 \text{ ksi}$$

**Step 5a:** Check compressive stress in masonry and the tensile stress in steel.

$$F_b = 4/3(1/3 f'_m) = 1333 \text{ psi}$$

$$f_m = f_a + f_b = 35 \text{ psi} + 496 \text{ psi}$$
$$= 531 \text{ psi} < 1333 \text{ psi OK}$$

$$F_s = 4/3 \text{ (20 ksi)} = 27 \text{ ksi} > 21.8 \text{ ksi OK}$$

**Step 6a:** Calculate shear stress.

$$f_v = \frac{V}{bjd} = \frac{1125 \text{ lb}}{(11.63 \text{ in})(0.938)(9 \text{ in})} = 12 \text{ psi}$$

**Step 7a:** Check shear stress.

bined pilaster and panel cross section is used to determine if cracking will occur in the panel. Therefore, determine if the pilaster and panel cross section is cracked. Assume the modulus of rupture, $f_r$, for hollow grouted brick masonry is equal to $4.0\sqrt{f'_m}$

= 219 psi (Ref. UBC 91). Refer to Fig. 6(b) for the location of the centroid of the combined cross section. Panel section properties are based on face shells only. ACI/ASCE/TMS code requires the maximum length of flange when determining stress distribution to be 6 times the thickness of the flange, i.e. 6 (5.63 in.) = 33.8 in. in each direction.

$$I_{base} = (1/12)(b)(t)^3 + (b)(t)(7.48\ in.-5.81\ in)^2$$

$$= (1/12)(11.63in)(11.63\ in)^3$$
$$+ (11.63\ in)(11.63\ in)(1.67\ in)^2$$
$$= 1900\ in^4$$

$$I_{panel} = 2(1/2)(2)(33.8\ in)(1.25\ in)^3$$
$$+(2)(33.8\ in)(t_{fs})(4.15\ in. - 0.63)^2$$
$$+(2)(33.8\ in)(t_{fs})(4.15\ in. - 5\ in.)^2$$
$$= 2(1/12)(67.5\ in)(1.25\ in)^3 + (67.5\ in)$$
$$(1.25\ in)(3.52\ in)^2 +(67.5\ in)(1.25\ in)$$
$$(0.85\ in)^2$$
$$= 1130\ in^4$$

$$I_g = I_{base} + I_{panel} = 3030\ in^4$$

$$M_{cr} = \frac{f_r I_g}{c} = \frac{(219\ psi)(3030\ in^4)}{(7.48\ in)(12\ in/ft)} = 7393\ ft\text{-}lb$$

M = 3375 ft-lb < 7393 ft-lb. Thus, the pilaster and panel cross section is not cracked under service load. Check the flexural tensile stress in the panel based on uncracked section properties.

$$f_t = \frac{M_c}{I_g} = \frac{(3375\ ft\text{-}lb)(4.15\ in)(12\ in/ft)}{3030\ in^4} = 55\ psi$$

The allowable flexural tensile stress, , is determined from Table 2. The percent of grouted to ungrouted area is used to interpolate between 68 psi and 25 psi for fully grouted and ungrouted hollow unit masonry, respectively, giving an allowable flexural tensile stress:

$$F_t = 4/3[25\ psi + (68\ psi - 25\ psi)(\#\ of\ grouted\ cores)/$$
$$(\#\ of\ cores)]$$
$$= 4/3\ [25\ psi + (43\ psi)(4/22)] = 44\ psi < 55\ psi$$

Therefore, try partial grouting to increase . If 4 more cells are grouted, the flexural tensile stress, $f_t$, reduces to 49 psi, and the allowable flexural tensile stress, $F_t$, equals 54 psi > 49 psi, <u>OK</u>.

**Step 9:** Sliding and overturning resistance of the caisson is a function of the lateral soil pressure and is beyond the scope of this *Technical Notes.*

**SUMMARY**

This *Technical Notes* discusses the structural design of brick masonry noise barrier walls. Design procedures are given following working stress analysis provided in the ACI/ASCE/TMS standard. Example designs are presented for recommended design of pier and panel, pilaster and panel and cantilever brick noise barrier walls.

The information and suggestions contained in this *Technical Notes* are based on the available data and the experience of the technical staff of the Brick Industry Association. The information and recommendations contained in this publication must be used in conjunction with good engineering judgment and a basic understanding of the properties of brick masonry and related construction materials. Final decisions on the use of

the information contained in this *Technical Notes* are not within the purview of the Brick Industry Association and must rest with the project designer and owner.

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