Example Problem: Buried Concrete Basement Wall Design

Problem Statement

Provide a detailed design for a new buried concrete basement wall in a single-story masonry building using the given information below.

Given Information

See Figure 1 for general layout and dimensions of wall section.

Given Information	Additional Guidance
Applicable Design Code is ACI 318-05	*referenced by major building codes (IBC, etc)*
Concrete compressive strength, f_c' = 4,000 psi	*common industry value; see project guidelines*
Reinforcement yield strength, f_y = 60,000 psi	*common industry value; ASTM A615, Grade 60*
Soil equivalent fluid pressure = 60 psf/ft	*obtain from geotech report; varies widely*
Use 2 ft additional soil surcharge to account for compaction pressure	*common technique for short walls and vehicle loading; other techniques exist for deeper walls*
Ground water table is deep below structure	*no buoyancy concerns; simplifies soil loading*
Structure in a low seismic region	*seismic forces do not control design*
Top slab acts as diaphragm	*pinned top support for wall*
Total service-level vertical dead load on wall = 2.5 kips/ft	*reasonable value for example purposes; determine load path and sum loads to get value*
Total service-level vertical live load on wall = 1.5 kips/ft	*reasonable value for example purposes; determine load path and sum loads to get value*

Designer's Assumptions

- Design wall with fixed base (*propped cantilever*)
- Neglect corner regions (*wall spans one-way only*)
- Top slab is in place and has achieved full strength prior to backfilling (*no construction case considered in example*)
- Use center to center of supports dimension of 15 feet for both moment and shear calculations (*simplification, will be conservative for shear calculations*)
- No vehicular traffic around building
- No eccentricities associated with vertical load (*simplification for example purposes only*)

Discussion: *In practice, a designer would also need to consider a partially fixed and/or pinned base. Realistically, the base support acts somewhere between fully fixed and fully pinned depending on the soil and reinf detailing. A design check would also be needed at the building corners where the wall will span both vertically and horizontally.*

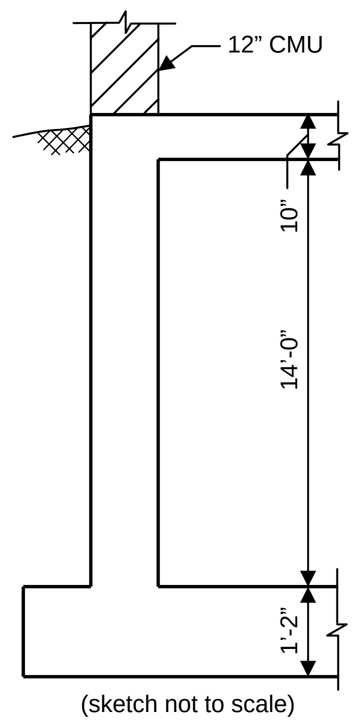


Figure 1

Calculations	References

NOTE: All references are for ACI 318-05 (UNO)

Load Determination

Soil Pressure:

- Max soil pressure, q_{max}, at base = (60 psf/ft)(15 ft + 2 ft)
  = 1020 psf
  = 1.02 ksf

- Min soil pressure, q_{min}, at top = (60 psf/ft)(2 ft)
  = 120 psf
  =0.12 ksf

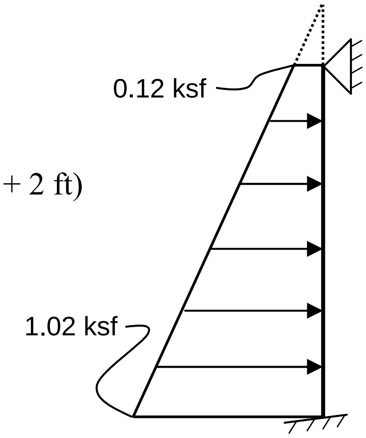
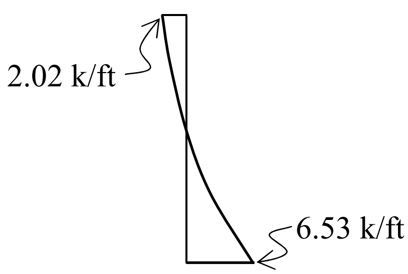


Figure 2

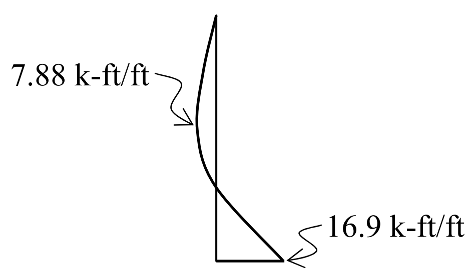
Service-Level Shear and Moment from Soil Pressure at base of wall:

- From third-party software: $V_{soil,max}$ = 6.53 k/ft
  $M_{soil,max}$ = 16.9 k-ft/ft



Shear (V_{soil})

Figure 3



Moment (M_{soil})

Figure 4

> **Discussion:** *In practice, a designer would need to check both the positive and negative moments and shears in the wall for all load combinations. A designer can optimize the amount of reinforcement at individual locations (i.e. inside face vs outside face). To limit constructability concerns and associated cost impacts, a designer should limit the use of multiple reinforcement sizes or spacing callouts.*

Factored Shear and Moment from Soil Pressure at base of wall:

- Shear: Vu = 1.6(6.53 k/ft) = 10.4 k/ft
- Moment: Mu = 1.6(16.9 k-ft/ft) = 27.0 k-ft/ft

Sec 9.2; Eq 9-2

Factored Vertical Axial Force from building, elevated slab above and self-weight:

- Total service dead load: 2.5 k/ft + (14 ft)(1 ft)(0.15 kcf) = 4.6 k/ft
- Axial: Pu = 1.2(4.6 k/ft) + 1.6(1.5 k/ft) = 7.9 k/ft

Note: by inspection, the other load combinations in Section 9.2 do not control.

Calculations	References
Overview of ACI 318 Chapter 14 (Walls) Requirements	

• Example must satisfy Sections 14.2 and 14.3 plus 14.4, 14.5 or 14.8.
  o Section 14.2 deals with general requirements
  o Section 14.3 deals with minimum reinforcement requirements
  o Sections 14.4, 14.5 and 14.8 provide design methods (only one of which is used in a given design problem)
	Sec 14.2.2

• Shear design must meet Section 11.10 (Special provisions for walls) requirements.	Sec 14.2.3

• Minimum reinforcement requirements are:
  o Vertical reinf – assume 0.15% of gross concrete area (assumption is valid if bar size is #5 or larger, conservative if not)
  o Horizontal reinf – assume 0.25% of gross concrete area
  o Vertical shear reinf – 0.25% of gross concrete area
  o Horizontal shear reinf – larger of Equation 11-32 and 0.25% of gross concrete area
	Sec 14.3.2; 14.3.3; 11.10.9.2; 11.10.9.4; Eq 11-32

• Use a maximum spacing of 18 inches (valid for walls thicker than 6 inches)	Sec 14.3.5
• Section 14.5 (Empirical design method) is not applicable for this example due to the lateral load. The resultant load would be greater than h/6. An example using this method can be found in PCA Notes, see Additional Reading section below. • Section 14.8 (Alternative design of slender walls) is not applicable for this example due to the fixed base assumption. An example using this method can be found in PCA Notes, see Additional Reading section below. • Since Sections 14.5 and 14.8 are not applicable, the designer must use Section 14.4 (Walls designed as compression members). This section refers to flexure and axial requirements in Chapter 10.	
Shear Design	
• Assume a 12 inch thick wall	

• Concrete shear strength: $V_C = \left(2 + \frac{4}{\beta}\right)\sqrt{f_C'}\,b_0 d$
  • for a wall, β approaches ∞
  • use a unit length approach, so b_0 = 12 in/ft
  • assume d = 9.5 inches based on #8 bar and 2 inches of cover (conservative for smaller bars)
	Sec 11.12.2.1; Eq 11-33

Note: this equation now matches Eq. 11-3 in Section 11.3.1.1, which is the basic shear equation for non-prestressed members.	
$V_C = \frac{2\sqrt{4000}(12)(9.5)}{1000} = 14.4\text{k/ft}$	

Calculations	References

Shear Design Continued

Required Strength: $\varphi Vn \geq Vu$ — Eq 11-1

where: Vn = nominal shear strength provided
Vu = factored shear force
= 10.4k/ft (from above)
$\varphi = 0.75$ — Sec 9.3.2.3

$$Vn = Vc + Vs$$ — Eq 11-2

where: Vc = nominal shear strength provided by concrete
Vs = nominal shear strength provided by steel

Neglecting any contribution from Vs (simplification, avoids special detailing), simplifies the equation to:

$$\varphi Vn = (0.75)(14.4) = 10.8\text{k/ft} \geq Vu = 10.4\text{k/ft}$$ **(OK)**

$\therefore$ A 12 inch thick wall is adequate for shear

Notes:

- This design is a conservative approach yet is common in industry, a designer could potentially reduce the wall thickness required for shear by:
  - Using Section 11.1.3 to move the critical section up a distance d from the base of the wall (this section applies for a soil case and mat fndn)
  - Calculate Vs and perform additional checks for any special detailing required, such as wall ties
  - Reduce Vu by reducing span to match the critical section
- A designer should be aware that by reducing the wall thickness d will be reduced as well, leading to increased steel requirements for flexure and axial forces. Additional slenderness concerns and second order effects need to be addressed with thin walls. Steel congestion can also be a concern if the steel to concrete ratio increases too much. Maintaining a straightforward detailing layout at the expense of additional wall thickness often produces a more economical design due to savings in labor cost so long as the design is not overly conservative.

Discussion: *In practice, a designer would also need to check in-plane shear. For long walls with low-rise buildings on them, they are often found to be adequate by inspection. Tall, short walls, especially those in higher seismic areas, require additional design and often special detailing. Section 11.10 and Chapter 21 contain the requirements. See the 'Additional Reading' section for additional design aids.*

Calculations	References

Flexure and Axial Design

Vertical reinforcement at base of wall

- Using Section 14.4 design method, walls designed as compression members

- Based on preliminary investigation, try #6 bars at an 8 inch spacing (#6@8”). Design is for outside face but use on both faces for simplification.

Area of a #6 bar = 0.44 in^2 — Appendix E

$$As = (0.44\,in^2)\left(\frac{12\text{ in/ft}}{8\text{ in}}\right) = 0.66\,in^2/ft \quad \text{per face}$$

- Check minimum reinforcement, although by inspection it appears to be ok

$$\rho_\ell = \frac{Ast}{bh} = \frac{0.66(2)}{(12)(12)} = 0.0092 > 0.0015 \qquad \textbf{(OK)}$$

Sec 14.3.2(b)

where: h = wall thickness = 12 inches
b = length = 12 inches (for unit length)

Note: this ratio also meets the minimum for vertical shear reinf (0.0092>0.0025).

Check wall slenderness:

- Assume wall is a non-sway condition (based on rigidity of basement walls, concrete diaphragm at top support, fixed base, etc)

  - k = effective length factor = 0.7 (fix/pin end conditions) — Fig R10.12.1(a)
  - lu = unbraced length = 14 ft = 168 inches
  - r = radius of gyration = 0.3(12 in) = 3.6 inches — Sec 10.11.2

$$\frac{(k)(lu)}{r} = \frac{(0.7)(168in)}{3.6in} = 32.7$$

$$32.7 < 34 - 12\left(\frac{M_1}{M_2}\right) = 34 - 12\left(\frac{0}{27}\right) = 34$$

Sec 10.12.2

where: M_1 = smaller factored end moment = 0 (pinned end)
M_2 = larger factored end moment = 27 k-ft/ft (fixed end)

$\therefore$ Slenderness effects may be ignored

Note: this result is somewhat unusual and is a result of having a slightly thicker wall than required for the given height. It is common for walls to need to be designed for slenderness. PCA Notes provides an example to check walls using Section 10.11.

Calculations	References
Vertical reinforcement at base of wall (continued)	
Perform strain compatibility analysis:	
• Assume wall section is tension controlled so $\varepsilon_t \geq 0.005$ and $\phi = 0.90$	Sec 10.3.4; Sec 9.3.2
Note: this is a common assumption and is usually correct for walls similar to the example wall. The assumption is verified below.	
$$Pn = 0.85\left(f_c^{'}\right)(b)(a) - (As)(fy)$$ where: Pn= nominal axial strength of cross section	Sec 10.3.1; Sec 10.2.1; Conc text books
Note: this version of the equation neglects the compression reinf contribution for simplicity. By using fy versus fs, failure will be initiated by yielding the tension steel.	
Set $Pn = Pu/\phi = (7.9 \text{ k/ft})/0.90 = 8.8 \text{ k/ft}$	
$$8.8 = 0.85(4)(12)(a) - (0.66)(60)$$	
Solving for a: a = 1.19 inches	
$$c = \frac{a}{\beta_1} = \frac{1.19}{0.85} = 1.40 \text{ inches}$$ where: $\beta_1 = 0.85$	Sec 10.2.7.1; 10.2.7.3
$$\varepsilon_t = \frac{0.003}{c}(d - c)$$	Sec 10.2.2
$$\varepsilon_t = \frac{0.003}{1.4}(9.5 - 1.4) = 0.0174$$	
∴ 0.0174>0.005 so the wall section is tension controlled	Sec 10.3.4

Calculations	References
Vertical reinforcement at base of wall (continued)	

Calculate design strength:

$$\varphi Mn = 0.90\left[0.85(f_c')(b)(a)\left(\frac{h}{2}-\frac{a}{2}\right)-(As)(fy)\left(\frac{h}{2}-d_t\right)\right]$$

Sec 10.3.1; 10.2.1; Conc text books

where: d_t = distance from compression fiber to extreme tension reinf layer = 9.5 inches (approx, conservative)

$$\varphi Mn = 0.90\left[0.85(4)(12)(1.19)\left(\frac{12}{2}-\frac{1.19}{2}\right)-(0.66)(60)\left(\frac{12}{2}-9.5\right)\right]$$

φMn = 360.9 k-in/ft = 30.1 k-ft/ft > 27 k-ft/ft = Mu **(OK)**

∴ #6@8” vertical reinf in each face is adequate

- As a graphical verification, pcaColumn was used to show the P-M diagram, see Figure 5 below. A unit length section of 1 ft was used.

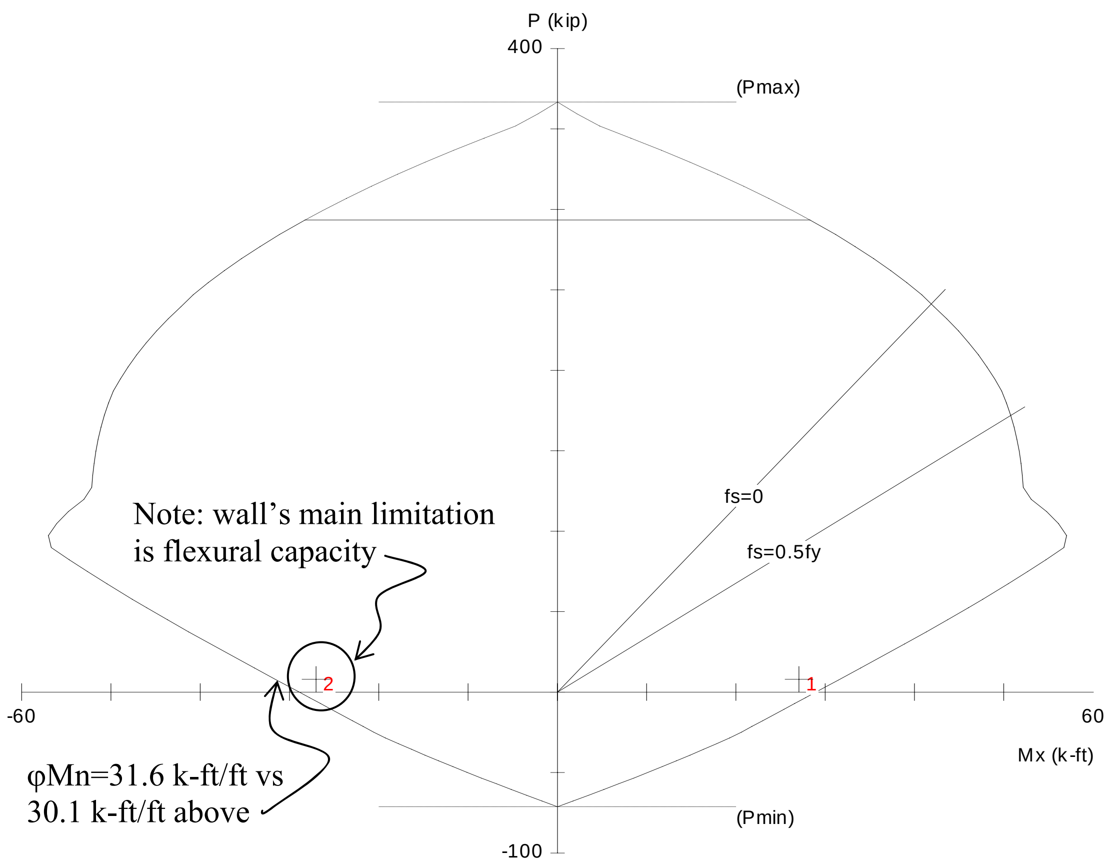


Figure 5

Calculations	References

Vertical reinforcement at base of wall (continued)

Check maximum spacing:

- Maximum spacing is the lesser of 3(h) and 18 inches (Sec 14.3.5)

$$3(12) = 36 \text{ in} > 18 \text{ in} > 8 \text{ in provided} \qquad \textbf{(OK)}$$

Discussion: *In practice, a designer could calculate the required steel for the wall above the base and use a lighter reinforcement layout in the main wall and the #6@8" as dowels. If a different spacing is used for the main wall, the designer should avoid complicated spacing layouts to simplify construction. A designer could also use less reinf on the inside face depending on other loading conditions.*

- The designer also needs to check the lap length between dowel and main reinf, Chapter 12 provides the necessary requirements.

Horizontal reinforcement

- Minimum reinf required is 0.25% of cross section; try #4 bars at 12 inches. Use on both faces. Note: in final design, corner regions will probably require a higher percent of steel due to moments from 2-way action.

$$\text{Area of a \#4 bar} = 0.20 \text{ in}^2$$

(Appendix E)

$$As = (0.20\,\text{in}^2)\left(\frac{12\,\text{in/ft}}{12\,\text{in}}\right) = 0.20\,\text{in}^2/\text{ft} \quad \text{per face}$$

$$\rho_t = \frac{As}{bh} = \frac{0.20(2)}{(12)(12)} = 0.0028 > 0.0025 \qquad \textbf{(OK)}$$

(Sec 14.3.3(b))

$\therefore$ #4@12" horizontal reinf in each face is adequate

Design Summary

Use a 12 inch thick wall with #6@8" vertical reinforcement on each face and #4@12" horizontal reinforcement on each face.

What Ifs

- Structure was in a high seismic region?
  - Designer would need to consider the additional load combinations, redundancy, select appropriate R values, etc. Chapter 21 would need to be followed, which contains several additional requirements for design and detailing.
  - Optimizing the wall thickness may help reduce the self-weight component of the seismic forces. A stepped wall approach might be beneficial with a deep wall.
  - Consult geotech report for seismic soil pressures, a common technique is to use an inverted triangle.
- No geotech report is available or report is old and not considered reliable?
  - Depending on the size of the project and importance of accurate information, a designer could request a new report however schedule and cost impacts should be considered.
  - If accurate information is less critical (i.e. during preliminary design) a designer may obtain common values based on soil type from geotechnical books or geotechnical engineers.
  - Soil properties, including equivalent fluid pressure, vary widely. It is not uncommon for soil conditions at two project sites close to each other to be different. Use caution if using a report from a different project site.

Additional Reading

- ACI 318-05, *Building Code and Commentary for Structural Concrete*, American Concrete Institute, Farmington Hills, MI, 2005.
  - Example is based on the use of this code. A designer should always be familiar with all code requirements as most examples do not cover all circumstances of a specific problem, this example included.
- *Notes on ACI 318-05 Building Code Requirements for Structural Concrete*, Portland Cement Association (PCA), Skokie, IL, 2005.
  - Contains numerous example problems and explanation for ACI 318-05. Explanation is provided with a more plain language approach, helping the designer better understand the code. Several design examples are provided for each chapter of ACI 318.
- Iyad M. Alsamsam and Mahmoud E. Kamara. *Simplified Design: Reinforced Concrete Buildings of Moderate Size and Height*, Portland Cement Association, EB104, 2004.
  - Provides examples and discussion for low rise buildings such as the one in this example. A method for performing in-plane shear design mentioned in the discussion above is presented in chapter 6.
- ACI 530-05, *Building Code Requirements for Masonry Structures and Commentary – ASCE 5-05/TMS 402-05*, Farmington Hills, MI, 2005.
  - Code to design masonry wall being supported by the concrete basement wall in this example.
- American Concrete Institute website, www.concrete.org.
  - Provides numerous additional references available for purchase in the bookstore.
  - Several design examples on many different topics
  - View abstracts for papers covering the latest in research and design methods.