E702.4

Designing Concrete Structures:

Buried Concrete Basement Wall Design

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Example Problem: Buried Concrete Basement Wall Design

Problem Statement

Provide a detailed strength design (durability and other considerations not included) for a new buried concrete basement wall in a single-story masonry building using the given information.

Given Information

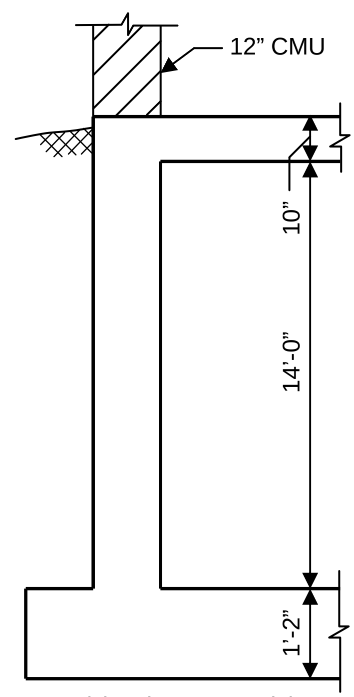
See Figure 1 for general layout and dimensions of wall section.

Given Information	Additional Guidance
Applicable Design Code is ACI 318-11	*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 (including slab self-weight)	*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 and pinned top (*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, relative wall/slab thicknesses and reinf detailing. A design check would also be needed at the building corners where the wall will attempt to span both vertically and horizontally.*



(sketch not to scale)

Figure 1

Version 2.0; Updated April 2013

Calculations | References

NOTE: All references are for ACI 318-11 (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

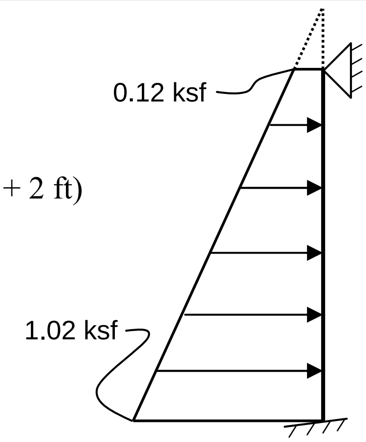
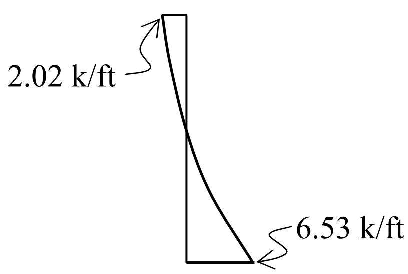


Figure 2

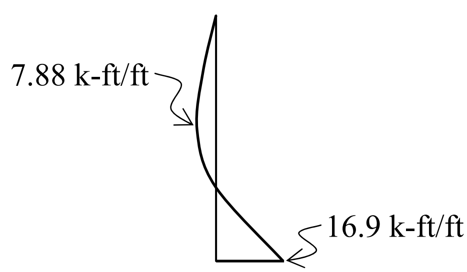
Service-Level Shear and Moment from Soil Pressure at base of wall:

- 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:

➢ Using Eq. 9-2 with 1.6H added per Section 9.2.5 (a). Dead, live, roof, snow & rain loads have zero lateral component in this example.
- 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 9.2.5; Sec 9.2.1; Eq 9-2 w/ 1.6H added

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

➢ For example purposes only, assume total live load given accounts for roof, snow, rain, etc. A one foot thick wall is initially assumed here, verify later in calculation.
- 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	
• Design shall satisfy Sections 14.2 and 14.3, plus 14.4, 14.5 or 14.8. o Section 14.2 described the general requirements o Section 14.3 provides minimum reinforcement requirements o Sections 14.4, 14.5 and 14.8 provide three different design methods (only one of which is used for any given design)	Sec 14.2.2
• Shear design shall be in accordance with Section 11.9 (Provisions for walls).	Sec 14.2.3
• Minimum reinforcement requirements are: o Vertical reinf – assume 0.15% of horizontal gross concrete area (assumption is valid if bar size is #6 or larger, conservative if not) o Horizontal reinf – assume 0.25% of vertical gross concrete area (assumption is valid if bar size is #6 or larger, conservative if not) o Vertical shear reinf – larger of Equation 11-30 and 0.25% of horizontal gross concrete area o Horizontal shear reinf – 0.25% of vertical gross concrete area	Sec 14.3.2; 14.3.3; 11.9.9.2; 11.9.9.4; Eq 11-30
• Use a maximum spacing of 18 inches (valid for walls thicker than 6 inches and longer than 7'-6")	Sec 14.3.5; 11.9.9.3; 11.9.9.5
• Section 14.5 (Empirical design method) is not considered appropriate for this example due to the higher lateral load and potential for the load resultant to have an eccentricity greater than h/6. A more appropriate example for this method can be found in PCA Notes, see the 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 the Additional Reading section below.	
• Section 14.4 (Walls designed as compression members) appears to be the most appropriate method for this example. This method uses the flexure and axial requirements in Chapter 10 (10.2, 10.3, 10.10, 10.11 and 10.14) and Chapter 14 (14.2 and 14.3).	Sec 14.4
Shear Design	
• Assume a 12 inch thick wall	
• Concrete shear strength: $V_C = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f_C'}\,b_0 d$ (controls over Eq 11-32 and 11-33 by inspection) • for a longer wall, β approaches ∞ (conservative) • λ is 1.0 for normal weight concrete (Sec 8.6.1) • using 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 11.9.1; 11.11.2.1; Eq 11-31
Note: this equation now matches Eq. 11-3 in Section 11.2.1.1, which is the basic shear equation for non-prestressed members. $V_C = \frac{2(1.0)\sqrt{4000}(12)(9.5)}{1000} = 14.4\text{ k/ft}$	

Calculations	References

Shear Design Continued

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

where: Vn = nominal shear strength provided
Vu = factored shear force
= 10.4 k/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 and avoids special detailing for easier constructability), simplifies the equation to:

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

∴ 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 or additional minimum steel requirements
  - Reduce Vu by reducing the span to match the critical section
- A designer should be aware that by reducing the wall thickness, d will be reduced as well. This leads to increased steel requirements for flexure and axial forces. Additional slenderness concerns and second order effects need to be addressed with thinner walls. Steel congestion can also be a concern if the steel to concrete ratio is higher (See 1% limit in Section 14.3.6).
- As long as the design is not too conservative, maintaining a straightforward detailing layout at the expense of a slightly thicker wall often produces a more economical design for the Owner due to savings in construction labor cost.

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 walls which have shorter plan lengths (i.e. high vertical to horizontal aspect ratios), especially those in higher seismic areas, require additional design and often special detailing. Section 11.9 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 at base but use on both faces for simplification.

Area of a #6 bar = 0.44 in^2

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

Appendix E

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

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

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

Sec 14.3.2(b)

Note: Shear reinforcement would also need to be checked, see Section 11.9.8 and 11.9.9. This reinf meets the minimum for vertical shear reinf (0.0092 > 0.0025).

Sec 11.9.9 & 11.9.9.4

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)
  - lu = unbraced length = 14 ft = 168 inches
  - r = radius of gyration = 0.3(12 in) = 3.6 inches

Fig R10.10.1.1(a)

Sec 10.10.1.2

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

$$32.7 < 34 - 12\left(\frac{M_1}{M_2}\right) \le 40 = 34 - 12\left(-\frac{0}{27}\right) = 34$$

Sec 10.10.1(b), Eq 10-7

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

∴ Slenderness effects may be ignored

Note: this result would be considered unusual for commercial construction or other industries which optimize wall thickness. It is a result of having a slightly thicker wall than required for the given height. Thinner and taller walls often need to be designed for slenderness. PCA Notes provides an example to check walls using Section 10.10.

Calculations	References
<u>Vertical reinforcement at base of wall (continued)</u>	
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 9.3.2.1
Note: this is a common assumption and is usually correct for walls similar to the example wall. The assumption is verified below.	
$$Pn = 0.85(f_c')(b)(a) - (As)(fy)$$ 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 k/ft)/0.90 = 8.8 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: β_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, the assumption is verified	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 at base is adequate

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

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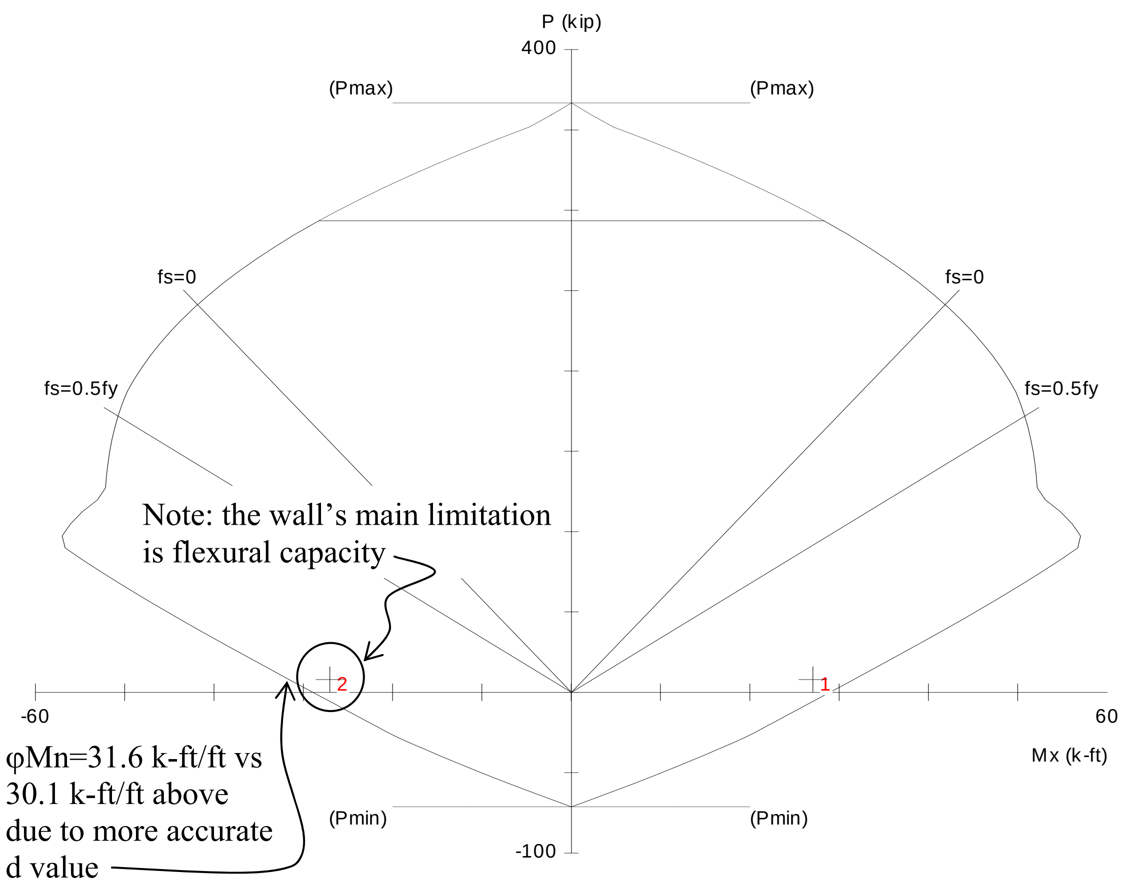

Figure 5

Calculations	References

Vertical reinforcement at base of wall (continued)

Check maximum spacing:

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

3(12) = 36 in > 18 in > 8 in provided **(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. For example, a designer could use #6@6" dowels and lap them with #6@12" main wall reinforcement at a location above the base where the moment is sufficiently reduced and the lap fully developed. A designer could also use less reinf on the inside face that still satisfies the positive moment, min reinf requirements and other loads.*

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

Horizontal reinforcement

- Minimum reinforcement required is 0.25% of gross cross section; try #4 bars at 12 inches. Use on both faces. Note: in final design, corner regions will probably require a higher ratio of reinforcement due to moments from 2-way action. Additional corner bars may be used to supplement the primary reinforcement or heavier primary reinforcement may be used throughout. This is often preferred for shorter length walls which approach length to height aspect ratios that would experienced more 2-way action, such as those with ratios of 3:1 or smaller.

Area of a #4 bar = 0.20 in^2 — Appendix E

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

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

Sec 14.3.3(b)

∴ #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 the geotechnical report for seismic soil pressures; a common technique is to use an inverted triangle.
- No geotechnical 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 might obtain common values based on soil type from other resources such as Chapter 18 of the International Building Code, geotechnical textbooks 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; it may not meet proper standard of care and may require permission from the author!

Additional Reading

- ACI 318-11, *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Farmington Hills, MI, 2011.
  - 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.
- ACI's Concrete Knowledge Center, www.concreteknowledge.org
  - Provides numerous design examples, online CEU program and other technical info. Some content is free to members (including 8 free CEU credits per year).
- American Concrete Institute website, www.concrete.org.
  - Bookstore contains numerous additional publications available for purchase.
- *Notes on ACI 318-11 Building Code Requirements for Structural Concrete, Companion to the Notes on ACI 318-08*, Portland Cement Association (PCA), Skokie, IL, 2012.
  - Contains numerous example problems and explanation for ACI 318-11. The explanation is provided using a plain language approach, helping the designer better understand the Code. Several design examples are provided for each chapter of ACI 318, including some referenced in this design example. This edition is a companion to the ACI 318-08 version, focusing on the changes only.
- Iyad M. Alsamsam and Mahmoud E. Kamara. *Simplified Design: Reinforced Concrete Buildings of Moderate Size and Height*, Portland Cement Association, EB204, 2012.
  - Presents simplified methods and design techniques that facilitate and speed the engineering of low-rise buildings within certain limitations. This design example uses similar simplifications which are commonly used in the industry.
- ACI 314R-11, *Guide to Simplified Design for Reinforced Concrete Buildings*, American Concrete Institute, Farmington Hills, MI, 2012.