Retaining Walls Types Details with Analysis and Design.

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Retaining walls are structures designed to restrain soil to unnatural slopes. They are used to bound soils between two different elevations often in areas of terrain possessing undesirable slopes or in areas where the landscape needs to be shaped severely and engineered for more specific purposes like hillside farming or roadway overpasses.

ئەم بابەتە کۆکراوەی چەند سەرچاوەیەکە بە مەبەستی بەرزکردنەوەی پلەی ئەندازیاری لە ڕێپێدراوەوە

بۆ ڕاوێژکار بە هیوای سوود گەیاندن بە گشت لایەك :

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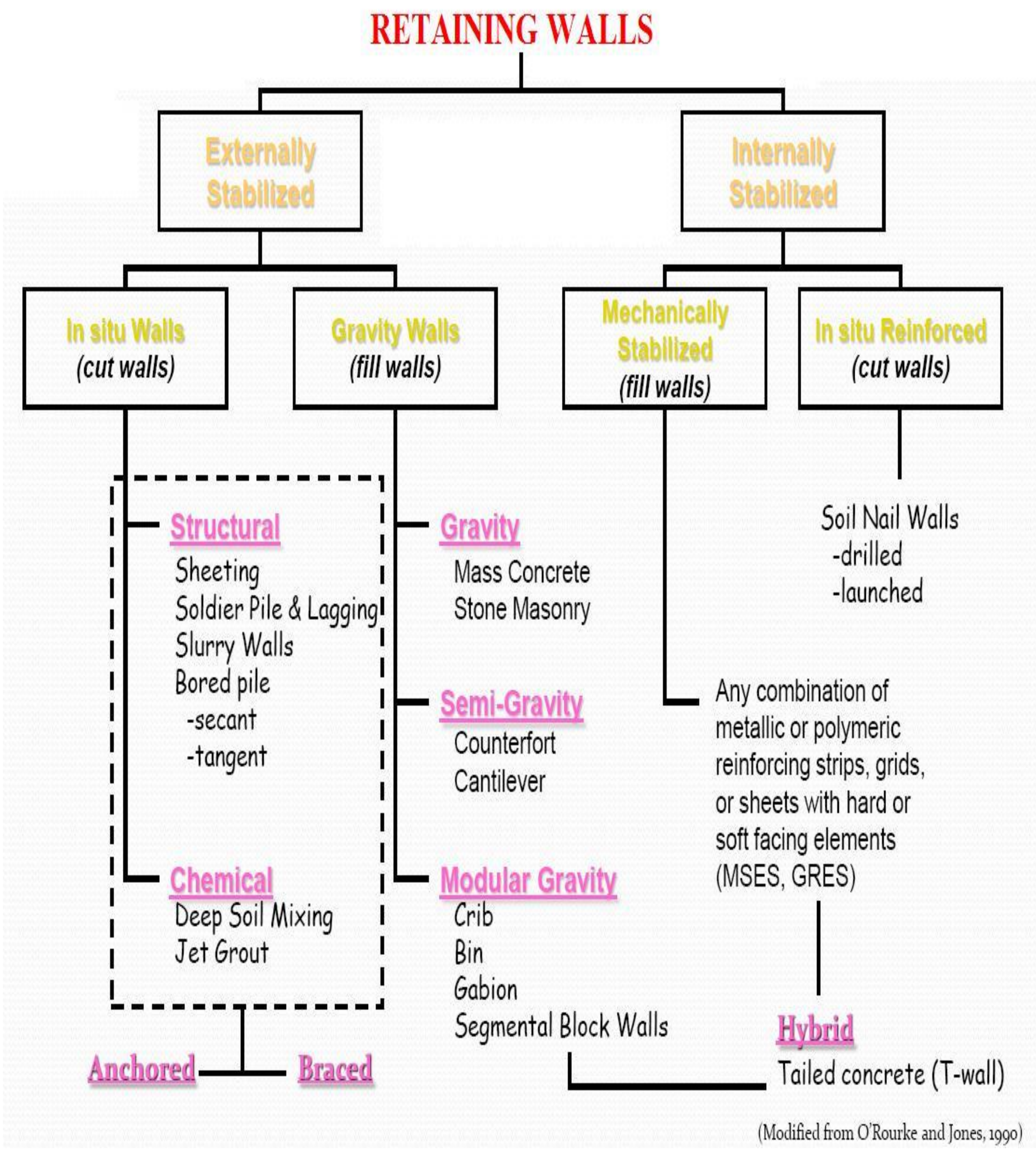
Evolution of Retaining Structures

In the year one-million BC, or thereabouts, an anonymous man, or woman, laid a row of stones A top another row to keep soil from sliding into their camp. Thus was constructed an early retaining wall, and we've been keeping soil in place ever since…… with increasingly better methods and understanding. The early engineers in the ancient cultures of Egypt, Greece, Rome and the Mayans were masters at invention and experimentation, learning primarily through intuition and trial-and-error what worked and what didn't We marvel at their achievements. Even the most casual observer looks in wonder at the magnificent structures they created and have stood for thousands of years – including countless retaining walls. With great skill they cut, shaped, and set stone with such precision that the joints were paper thin. Reinforced concrete would not be developed for a thousand years, but they used what they had, and learned how to do it better with each succeeding structure. Consider the Great Wall of China, for example, where transverse bamboo poles were used to tie the walls together – a forerunner of today's "mechanically stabilized earth". Those early engineers also discovered that by battering a wall so that it leaned slightly backward the lateral pressure was relieved and the height could be extended – an intuitive understanding of the soil wedge theory. Any student of ancient construction methods is awed by the ingenuity and accomplishments of those early engineers.

Major advances in understanding how retaining walls work and how soil generates forces against walls appeared in the 18th and 19th centuries with the work of French engineer Charles Coulomb 1776, who is better remembered for his work on electricity, and later by William Rankine in 1857. Today, their equations are familiar to most civil engineers. A significant body of work was the introduction of soil mechanics as a science through the pioneering work of Karl Terrzaghi in the 1920s. Indeed, soil mechanics and the design of retaining structures has advanced dramatically in recent decades giving us new design concepts, a better understanding of soil behavior, and hopefully safer and more economical designs.

What are Retaining Walls?

A retaining wall is any constructed wall that restrains soil or other material at locations having an abrupt change in elevation.   They are used to retain soil, rock or other materials in a vertical condition. Hence they provide a lateral support to vertical slopes of soil that would otherwise collapse into a more natural shape.


RETAINING WALLS
Externally Stabilized
Internally Stabilized
In situ Walls (cut walls)
Gravity Walls (fill walls)
Mechanically Stabilized (fill walls)
In situ Reinforced (cut walls)
Structural
Sheeting
Soldier Pile & Lagging
Slurry Walls
Bored pile
-secant
-tangent
Chemical
Deep Soil Mixing
Jet Grout
Anchored
Braced
Gravity
Mass Concrete
Stone Masonry
Semi-Gravity
Counterfort
Cantilever
Modular Gravity
Crib
Bin
Gabion
Segmental Block Walls
Any combination of metallic or polymeric reinforcing strips, grids, or sheets with hard or soft facing elements (MSES, GRES)
Soil Nail Walls
-drilled
-launched
Hybrid
Tailed concrete (T-wall)
(Modified from O'Rourke and Jones, 1990)

In general, retaining walls can be divided into two major categories:

- **(a) Conventional retaining walls.**
- **(b) Mechanically stabilized earth walls.**

  Conventional retaining walls can generally be classified as

  1- Gravity retaining walls.
  2- Semi-gravity retaining walls.
  3- Cantilever retaining walls.
  4- Counterfort retaining walls

Gravity retaining walls:

Gravity retaining walls (figure –a-) are constructed with plain concrete or stone masonry. They depend on their own weight and any soil resting on the masonry for stability. This type of construction is not economical for high walls. It is used for height up to 3.00m. It is economical for height up to 1.80m.

Semi-gravity retaining walls:

In many cases, a small amount of steel may be used for the construction of gravity walls, thereby minimizing the size of wall sections. Such walls are generally referred to as semi-gravity walls.

Cantilever retaining walls:

Cantilever retaining walls (figure –b-) are made of reinforced concrete that consists of a thin stem and a base slab. This type of wall is economical to a height of about (7.0 m).

Counterfort retaining walls:

Counterfort retaining walls (figure –c-) are similar to cantilever walls. At regular intervals, however, they have thin vertical concrete slabs known as counterforts that tie the wall and the base slab together.

Sheet Piling walls:

Sheet pile walls are constructed by **driving prefabricated sections into the ground**. Soil conditions may allow for the sections to be vibrated into ground instead of it being hammer driven. The full wall is formed by connecting the joints of adjacent sheet pile sections in sequential installation.

Concrete pile wall :

a- **Secant pile**

Secant pile walls are formed by constructing intersecting reinforced concrete piles. The secant piles are reinforced with either steel rebar or with steel beams and are constructed by either drilling under mud or augering.

b- **Contiguous pile**

A contiguous bored pile wall is formed by constructing a series of individual vertical RC piles. The diameter of each pile in a contiguous piled wall is usually not less than 300mm diameter. Contiguous piles are suitable where the groundwater table is below excavation level.

Restrained (Non-yielding) retaining walls

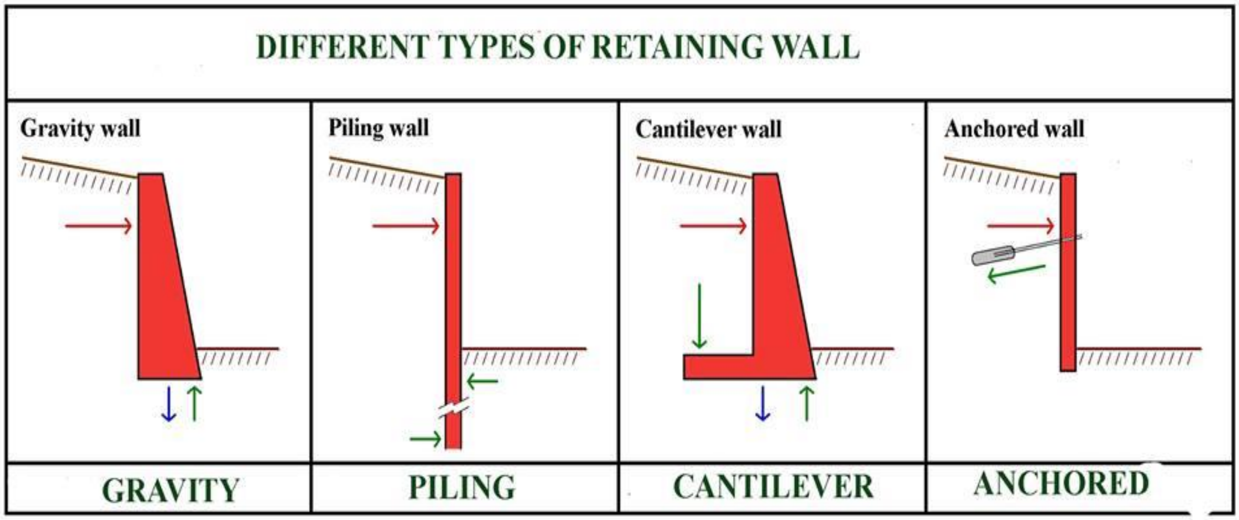
Also called “basement walls” (for residential and light commercial conditions) or “tie-back” Walls. These walls are distinguished by having lateral support at or near the top, thereby with less or no dependence for fixity at the foundation. Technically, these walls are classified as “nonyielding” walls because the walls cannot move laterally at the top, as opposed to cantilevered (yielding) walls. Such walls are usually designed as “pin connected” both at the top and bottom.

The earth pressure creates a positive moment in the wall, which requires reinforcing on the front of the wall, that is, the side opposite the retained soil. . In some cases it may be cost effective to fix the base of the wall to the footing to reduce both the bending in the wall and restraining force required at the top support.

Footings for these walls are usually designed for vertical loads only. However, it is often desirable to design the lower portion of a basement wall as a cantilevered retaining wall with fixity at the footing so that backfill can be safely placed to avoid bracing the wall, or waiting until the lateral restraint at the top is in place, such as a floor diaphragm. Note that conventional wood floors framed into the top of a basement wall may not provide a sufficient stiffness to allow for the restrained case,

Anchored retaining walls:

Anchor Piles are required to resist lateral loads with or without being braced depending on circumstances and an ordinary or standard house pile is required to carry a vertical load.



Gabion or crib walls

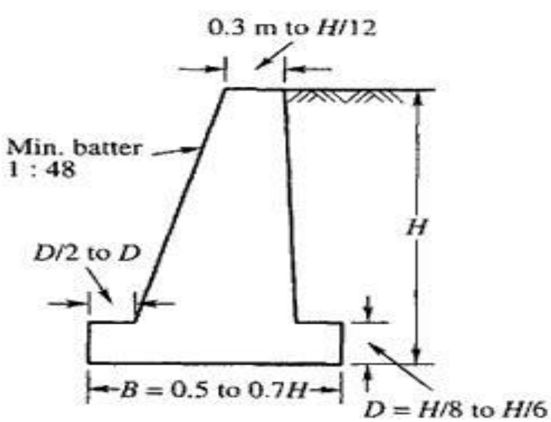
A gabion wall is a type of gravity wall whereby stones or rubble are placed within wire fabric baskets. Crib walls are a variation of the gabion method whereby mostly steel bins are filled with stone or rubble. Another variation is to stack a grillage of timbers and fill the interior with earth or rubble. Precast concrete crib walls are also widely used.

Wood retaining walls

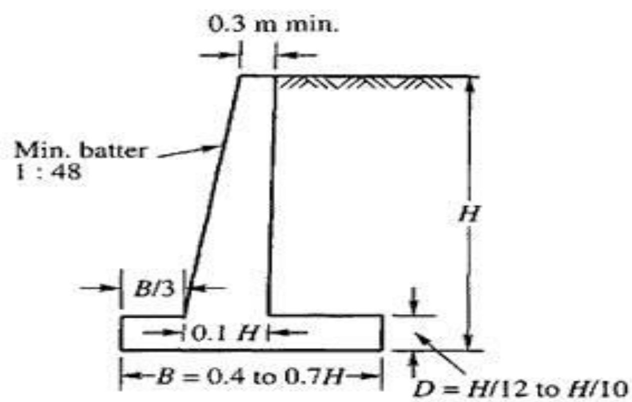
Wood is commonly used for low height retaining walls. Wood retaining walls usually consist of laterally spaced wood posts embedded into the soil, preferably into a drilled hole with the posts encased in lean concrete. Horizontal planks span between the upward cantilevering posts. Pressure treated wood is used, but even with treatment deterioration is a disadvantage, and wood walls are generally limited to low walls because height is limited by size and strength of the posts. Railroad ties are also commonly used for both posts and lagging.

Bridge abutments

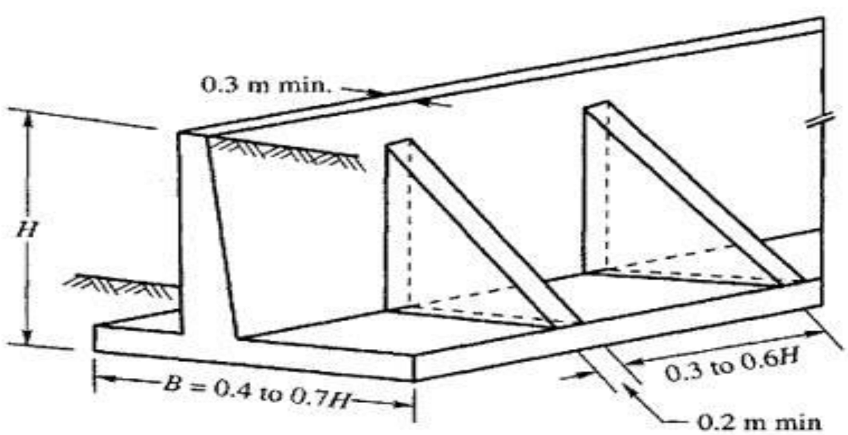
These support the end of a bridge and retain the earth embankment leading to the bridge. Bridge abutments usually have angled wing walls of descending height to accommodate the side slope of the embankment. Abutments are designed as cantilever walls, with girder bearing support free to slide at one end to accommodate horizontal expansion movement of the bridge deck. Design requirements for bridge structures are usually governed by the code requirements of the American Association of State Highway and Transportation Officials (AASHTO) and state Departments of Transportation (DOTs) such as California's CalTrans.



(a) Gravity wall



(b) Cantilever wall



(c) Counterfort wall

Sheet Piles Walls

What the Terms Mean:

Backfill: The soil placed behind a wall.
Backfill slope: Often the backfill slopes upward from the back face of the wall. The slope is usually expressed as a ratio of horizontal to vertical (e.g. 2:1).
Batter: The slope of the face of the stem from a vertical plane, usually on the inside (earth) face.
Dowels: Reinforcing steel placed in the footing and bent up into the stem a distance at least equal to the required development length.
Footing (or foundation): That part of the structure below the stem that supports and transmits vertical and horizontal forces into the soil below.
Footing key: A deepened portion of the footing to provide greater sliding resistance.
Grade: The surface of the soil or paving; can refer to either side of the wall.
Heel: That portion of the footing extending behind the wall (under the retained soil).
Horizontal temperature/shrinkage reinforcing: Longitudinal horizontal reinforcing usually placed in both faces of the stem and used primarily to control cracking from shrinkage or temperature changes.
Keyway: A horizontal slot located at the base of the stem and cast into the footing for greater shear resistance.
Principal reinforcing: Reinforcing used to resist bending in the stem.
Retained height: The height of the earth to be retained, generally measured upward from the top of the footing.
Stem: The vertical wall above the foundation.
Surcharge: Any load placed in or on top of the soil, either in front or behind the wall.
Toe: That portion of footing which extends in front of the front face of the stem (away from the retained earth).
Weep holes: Holes provided at the base of the stem for drainage. Weep holes usually have gravel or crushed rock behind the openings to act as a sieve and prevent clogging. Poor drainage of weep holes is the result of weep holes becoming clogged with weeds, thereby increasing the lateral pressure against the wall. Unless properly designed and maintained, weep holes seldom “weep”. Alternatively, perforated pipe surrounded with gravel and encased within a geotextile can be used to provide drainage of the backfill.

Design Criteria Checklist

Before establishing specific design criteria, the following checklist should be used before starting your design:

- What building codes are applicable?
- Do I have the correct retained height for all of my wall conditions?
- Is there a property line condition I need to know about?
- Is there a fence on top of the wall, or does the wall extend above the retained height? (exposure to wind)
- How deep must the bottom of the footing be (frost considerations?)?
- How will I assure that the backfill will be drained?
- Will there be any axial loads on top of the wall? If so, the eccentricity?
- What about surcharges behind the wall, such as parking, trucks, etc.
- If the wall extends above the higher grade, and is a parking area, is there an impact bumper load?
- What is the slope of the backfill? Level?
- Is there a water table I need to consider?
- Is a seismic design required?
- Are there any adjacent footing loads affecting my design?
- Should the stem be concrete or masonry, or a combination of the two?
- How high is the grade on the toe side, above the top of the footing?
- Is there a slab in front of the wall to restrain sliding or provisions to prevent erosion of soil?
- Is there lateral restraint at the top of the wall (if so, it’s not truly a cantilevered wall and requires a different design)?
- Do I have a soil investigation report or other substantiation for soil properties: active pressure, passive pressure, allowable bearing pressure, sliding coefficient, soil density, and other items I need to consider?
- Also consider whether a cantilevered retaining wall is the right solution. If the height of the wall is over about 16 feet, perhaps a tieback wall would be more economical (caution: be sure your client has the right to install tiebacks. If the wall is on a property line, there is obviously a problem). Perhaps a buttressed or counterfort wall would be better for high walls, or using Basics of Retaining Wall Design precast panels, or tilt-up to overcome construction constraints imposed by a restrictive rear property line.
- Lastly, determine how many conditions for which you will need a design. Perhaps the same retained height has several different backfill slopes, say, from level to 2:1. Here you need to use a little judgment in determining the number of cases. Usually you don’t design for less than two-foot height increments, unless there are different surcharges or other conditions. To design for one-foot height increments is not only tedious, but doesn’t save that much material cost. On the other hand, if the retained height along the length of a wall varies from, say, four feet to 12 feet, you would not want to specify the 12-foot design throughout. In this case, you would probably design for 12', 10', 8', 6' and 4'. You rarely “design” a wall less than 4 feet high, just use a little judgment unless

there is a steep backfill slope or large surcharges, in which case it should be properly designed.

Establish Design Criteria

The following information will be needed before starting your design. The values shown in parentheses are only given to illustrate those values frequently used.
Retained heights
Embedment depth of footing required below grade – See geotechnical report
* Allowable soil pressure.
* Passive pressure.
* Active earth pressure.
* Coefficient of friction.
Backfill slope (don't exceed about 2:1 horizontal: vertical unless approved by the geotechnical engineer)
Axial loads on stem
Surcharge loads
Wind, if applicable
* Seismic criteria if applicable
* Soil density.
Concrete and masonry allowable stresses (usually used values in parentheses
f 'c ,fy, fs, f'm , fr

* These values are usually given in the geotechnical report.
When you have gathered this information, you're ready to start.

Step-by-Step Design of a Cantilevered Retaining Wall

The design usually follows this order:
1. Establish all design criteria based upon applicable building codes. (See checklist above).
2. Compute all applied loads, soil pressures, seismic, wind, axial, surcharges, impact, or any others.
3. Design the stem. This is usually an iterative procedure. Start at the bottom of the stem where moments and shears are maximum. Then, for economy, check several feet up the stem (such as at the top of the development length of the dowels projecting from the footing) to determine if the bar size can be reduced or alternate bars dropped. Check dowel embedment depth into the footing assuming a 90° bend (hooked bar). The thickness of the stem may Basics of Retaining Wall Design vary, top to bottom. The minimum top thickness for reinforced concrete walls is usually 6- inches to properly place the concrete, 8-inches at the bottom.
4. Compute overturning moments, calculated about the front (toe) bottom edge of the footing. For a trial, assume the footing width, to be about 1/2 to 2/3's the height of the wall, with 1/3 being at the toe.
5. Compute resisting moments based upon the assumed footing width, calculated about the front edge of the footing.

6. Check sliding. A factor of safety with respect to sliding of 1.5 or more is standard. A key or adjusting the footing depth may be required to achieve an accepted factor of safety with respect to sliding.
7. An overturning factor of safety of at least 1.5 is considered standard of practice.
8. Based upon an acceptable factor of safety against overturning, calculate the eccentricity of the total vertical load. Is it within or outside the middle-third of the footing width?
9. Calculate the soil pressure at the toe and heel. If the eccentricity, e, is > B/6 (B = width of footing) it will be outside the middle third of the footing width (not recommended!), and because there cannot be tension between the footing and soil, a triangular pressure distribution will be the result. Consult with the project geotechnical engineer if this condition cannot be avoided, as it will result in a substantially lowered allowable soil bearing pressure.
10. Design footing for moments and shears. Select reinforcing.
11. Check and review. Have all geotechnical report requirements been met?
12. Place a note on the structural sheets and on the structural calculations indicating that the backfill is to be placed and compacted in accordance with the geotechnical report.
13. Review the construction drawings and specifications for conformance with the design.

Step-By-Step Design of a Restrained Retaining Wall

Similar to the above with some additional steps (italicized):
1. Establish all design criteria based upon applicable building codes. (See checklist above).
2. Compute all applied loads (at-rest earth pressures, seismic, wind, axial, surcharges, impact, or any others. *Select "height" to lateral restraint.*
3. *Select restraint – level and base of stem design assumptions: pinned - pinned; pinned fixed; or fixed - fixed. Then based on statics determine the reactions at the top and at the base of the wall.*
4. *If a floor slab is present at the top of the footing, check its adequacy to sustain this lateral sliding force.*
5. Design the stem. *If the stem is assumed pinned at the base and at the top, the maximum moment will be a positive moment near mid-height—select stem material, design thickness, and reinforcing for that location. Usually the same material (concrete or masonry) and thickness will be used for the full height. Some degree of "fixity" is likely at the top of the wall even with a pinned "design".*
6. Design the footing. *If the stem is assumed fixed at the base check the soil pressure (check Items 8 and 9 as above) and design for the moments and shears, and select reinforcing. If the*
Basics of Retaining Wall Design
stem is assumed pinned at the footing interface, try to center the footing under the wall to prevent eccentricity. If there is eccentricity check reinforcing at stem-footing interface to resist that moment because if it exceeds the moment due to eccentricity the soil pressure will not be uniform Check embedment depth into the footing assuming a 90° bend (hooked bar).
7. Check sliding. *If a restraining floor slab is not present, a key or adjusting the footing width or depth may be required.*
8. Check and review. *Have all soil report requirements been met?*

9. Review the construction drawing for conformance with your design.

SOIL MECHANICS SIMPLIFIED

A Soil Primer

The mantle of our earth is composed of water, rock and soil. It is the soil or rock that supports our structures. We need to understand what soil is, how it behaves, and the properties we need for design. Soil is a collective term for any mixture of sand, silt, or clay. Soil is not “dirt”, which we sweep off the floor and wash from our clothes. Dirt is a colloquial term contractors often use, such as “We underestimated the fill quantity and need to import 200 more yards of dirt (a “yard” in that terminology means one cubic yard).

Soil is the result of the decomposition of rock. Rocks decompose by weathering, freezing and thawing, by crushing and grinding along earthquake faults, along planes of failure in landslides, by the overland movement of glaciers, the tumbling action of rivers and streams, and from the corrosive inorganic acids present in the atmosphere and derived from plants. Additionally, we must add heat, temperature changes and pressures within the earth.

Before the mid-1920s, determining how large a footing was needed to support a structure was rudimentary. It consisted primarily of driving rods into the soil and observing the resistance, auger borings, test pits, and usually load testing a small area and observing tolerable deformations from which a footing could be safely sized. Recommended bearing capacities were published in the handbooks of the day. For instance, the 1916 *New York Building Code* listed capacity of various soils. An example: “Sand and clay mixed or in layers” allowed “2 tons per square foot”.

A pioneer to advance soil behavior to a science was Karl Terzaghi (1883-1963) who in 1925 published *Erdbaumechanik*, which loosely translates to mechanics of soil in construction, followed in 1926 by *Principles of Soil Mechanics*. Later, in 1948, he and Peck published the classic *Soil Mechanics in Engineering Practice*. His studies were based upon application of the theory of elasticity to mass materials. From his work, and that of others, the term *soil mechanics* evolved into *geotechnical engineering*.

Moving ahead to today, types of soil – sand, silt, or clay, primarily – are classified by particle size and the composition of the mixture. The distribution of grain size in a soil sample is determined by a grain size analysis. For example, in a sieve test a sample is passed through successively smaller sieves, and the amount by weight retained on each sieve is noted as a percent of the total.

With this information the geotechnical engineer can classify the soil per the most-used *Uniform System for Classification of Soil* (USCS) that is reproduced in Appendix B. Sieve sizes use a numbering system where the number indicates the number of openings per inch. For example, a #4 sieve has four openings per inch, or ¼” each, and a #200 sieve has 200 openings per inch, and so forth.

Some common designations of soil are:

Boulders > 12”
Cobbles > 3” < 12”
Gravel > #4 sieve < 3”
Sand > #200 sieve < #4 sieve
Silt < #200 sieve
Clay < 0.005 to 0.002 mm

There are other classifications systems, such as the AASHTO system (American Association of State and Highway Transportation Officials), but the USCS classification is most often referred to in the foundation investigation reports you will read.

Soil is further classified as being cohesive, non-cohesive, or somewhere in between.
Cohesive soil derives its strength primarily from the cohesive bond between particles. Examples include fine-grained silts and clays.

Non-cohesive, or granular soil, derives its strength from inter-particle friction between grains. Sand and gravel are examples of non-cohesive soil. Non-cohesive soil is the type usually assumed for analysis of pressures against a retaining wall.

Expansive soils usually consist of clay, but some silt is also expansive. Expansive soil can lift footings if water is present or shrink upon drying. Some clays are highly expansive and change in volume with changes in water content. Such swelling can cause considerable pressure on retaining structures. It is for this reason that clay backfill should be avoided, and if the site contains expansive soil, the geotechnical engineer will recommend measures to minimize its effect, mainly by removal and replacement with suitable material. It is important that water not be allowed to penetrate expansive soil.

Frost line is a term used in colder climates in the northern US, whereby upper portions of the ground may freeze seasonally or permanently, with depths ranging from a few inches to eight feet or more. To prevent the added pressure of swelling because of freezing, foundations should be placed below the frost line. The geotechnical engineer and applicable building codes will address this local concern. In areas where the ground is permanently frozen to a great depth, such as
Alaska, local expertise and experience will apply.
Bearing capacity of a soil is an estimate of its capability to support a vertical load in compression.
The shearing strength of the soil is the controlling factor for determining its bearing capacity. The shear between particles can be either frictional resistance (sliding friction between particles) or in the case of a clayey soil, cohesion and perhaps inter particle friction. Sandy soil requires confinement to develop shear strength, as for example a lack of confinement is illustrated when you step on sand at the beach you will notice the sand displaces sideways under your feet. This illustrates the lack of frictional forces at work.

When soil samples (cores retrieved from drilling) are taken to the laboratory for testing, the geotechnical engineer will calculate the bearing capacity of the particular soil by determining its angle of internal friction, , its unit cohesion, c, and its unit weight.

Most soil mechanics texts will thoroughly cover the several types of shear tests available to the geotechnical engineer.

The basic equation for shear resistance developed along a plane of rupture is:

$s = c + p \tan \varnothing$
s = shear strength; p = effective normal stress and c = cohesion, both usually expressed in psf; and $\varnothing$ = effective angle of internal friction.

BUILDING CODES AND RETAINING WALLS

What Building Code(s) Apply To This Project?

Always check with the Building Department having jurisdiction over the project to determine the code(s) adopted by the jurisdiction and if any local amendments apply. The following codes are most often adopted or cited.

Building Codes

International Building Code (IBC)

This standard building code has been adopted by most jurisdictions, some with local modifications (California Building Code, for example). The IBC was a culmination of efforts to merge into one national building code the Uniform Building Code, Southern Building Code, and Standard Building Code. The IBC is compiled and published by the International Code Council (ICC), County Club Hills, Illinois. The series of International Building Codes (e.g. plumbing, electrical, etc.) are collectively referred to as the "I-Codes". The IBC Website is www.iccsafe.org. The current edition is 2012. IBC 2012 references or modifies other standard codes, principally ASCE 7-10 Minimum Design Loads for Buildings and Other Structures.

Uniform Building Code (UBC), '97

This now defunct code, the last in a series first published in 1927 by the International Conference of Building Officials, was the dominant code in the Western states until replaced by the International Building Code and California Building Code.

California Building Code (CBC)

This California code was first published in 2001 to replace the ’97 Uniform Building Code. It is an adaptation of the IBC with minor modifications and is essentially the same as the IBC. The current edition is 2013. See www.bsc.ca.gov.

NFPA 5000: Building Construction and Safety Code (National Fire Prevention Association)

NFPA 5000 has been promoted in some States. It addresses construction protection and occupancy features necessary to minimize danger to life and property. The current edition is NFPA 5000: Building Construction and Safety Code, 2012 Edition. The NFPA web address is www.nfpa.org. This code references ACI 318, ASCE 7 and ACI 530 for structural design issues.

Referenced Publications

IBC 2012, CBC ’10, and other regional codes, often refer to the following standards for structural issues:

Minimum Design Loads for Buildings and Other Structures, ASCE 7-10 Published by American Society of Civil Engineers (ASCE), Reston, VA. This often referenced publication covers loads and seismic design. See www.asce.org.

Building Code Requirements for Reinforced Concrete (ACI 318-11), American Concrete Institute (ACI), Detroit, MI. The standard for concrete design. See **www.concrete.org**

Building Code Requirements for Masonry Structures (ACI 530.1-11) Also known as MSJC, this masonry code is published jointly by ACI, SEI, and The Masonry Society.

National Earthquake Hazard Reduction Program (NEHRP), developed by the Building Seismic Safety Council for FEMA (Federal Emergency Management Agency). This is not a code, per se, but is referenced by IBC and NFPA as guidelines for seismic design. The 2009 Edition NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures contains often referenced information on seismic design of retaining walls, particularly information in the Commentary, which is discussed in Chapter 6 of this book. See **www.nehrp.gov.**

Annual Book of ASTM Standards. This is the standard of reference on materials and processes cited in most codes and specifications. Its 70+ volumes cover over 11,000 specifications. Published by ASTM International, West Conshocken, PA. See **www.astm.org.**

National Design Standards for Wood Construction (NDS), 2012. Published by American Wood Council **www.awc.org.**

Other codes as applicable:

AASHTO LRFD Bridge and Highway Design Specifications, 5th. Edition, 2010, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C. **www.aashto.org.** Adopted by most states, some with amendments (California's Caltrans for example.)

Naval Facilities Engineering Command (NAVFAC). Foundations and Earth Structures, NAVFAC Design Manual 7.02. This design manual contains information on many aspects of retaining structures. Refer to www.navfacnavy.mil for more information. Many Navy documents are available for download.

U.S. Army Corps of Engineers Design Manuals. Comprehensive design procedures, Standards, and sample calculations: The web address is: **www.usace.army.mil.** Many Corps documents are available for download.

PROPORTIONING RETAINING WALLS:

When designing retaining walls, an engineer must assume some of the dimensions, called proportioning, which allows the engineer to check trial sections for stability. If the stability checks yield undesirable results, the sections can be changed and re-checked. The figure a,b,and c above shows the general proportions of various retaining walls components that can be used for initial checks.

[Note: minimum dimension of D is 2 ft (≈0.6 m)]

For counterfort retaining walls, the general proportion of the stem and the base slab is the same as for cantilever walls. However, the counterfort slabs may be about 12 in.(≈0.3 m) thick and spaced at center-to-center distances of (0.3 H to 0.7 H).

General Consideration for design:

To design retaining walls properly, an engineer must know the basic soil parameters-that is, the unit weight (γ), angle of friction (ϕ), and cohesion-for the soil retained behind the wall and the soil below the base slab (C). Knowing the properties of the soil behind the wall enables the engineer to determine the lateral pressure distribution that has to be designed for. There are two phases in the design of conventional retaining walls:

First: With the lateral earth pressure known, the structure as a whole is checked for stability. That includes checking for possible overturning, sliding, and bearing capacity failures. [Geotechnical Part]

Second: Each component of the structure is checked for adequate strength, and the steel reinforcement of each component is determined. [Structural Part].

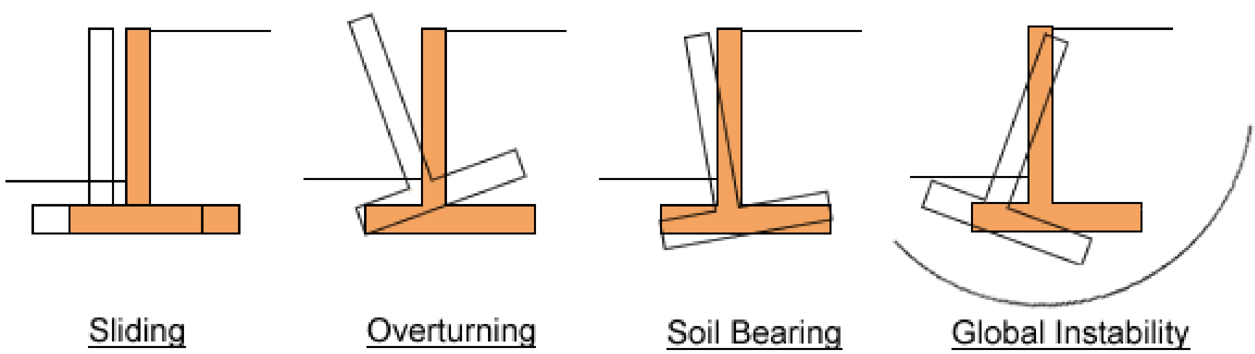
A retaining wall must have sufficient weight and width or be otherwise supported so that it does not overturn or slide forward due to external forces being exerted upon it. For this reason, the retaining wall must be stable against overturning, sliding and bearing capacity.

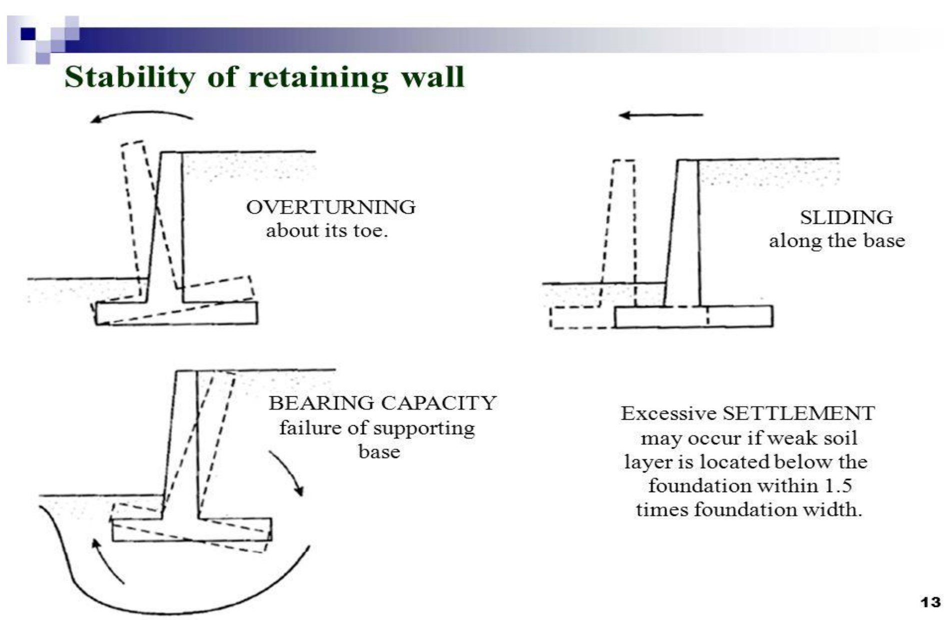
To check the stability of a retaining wall, the following steps are necessary:

1. Check for overturning about its toe.

2. Check for sliding along its base.

3. Check for bearing capacity failure of the base.





For each of these considerations, the resisting or stabilizing forces must exceed the forces that would cause failure by a predetermined Factor of Safety (FOS) for each of these considerations. The selected factors of safety should reflect the consequences of failure and the designer's confidence in the accuracy of the input parameters. The following factors of safety are normally used in the design of retaining walls:

FOS - Overturning ≥ 2.0 to 3.0

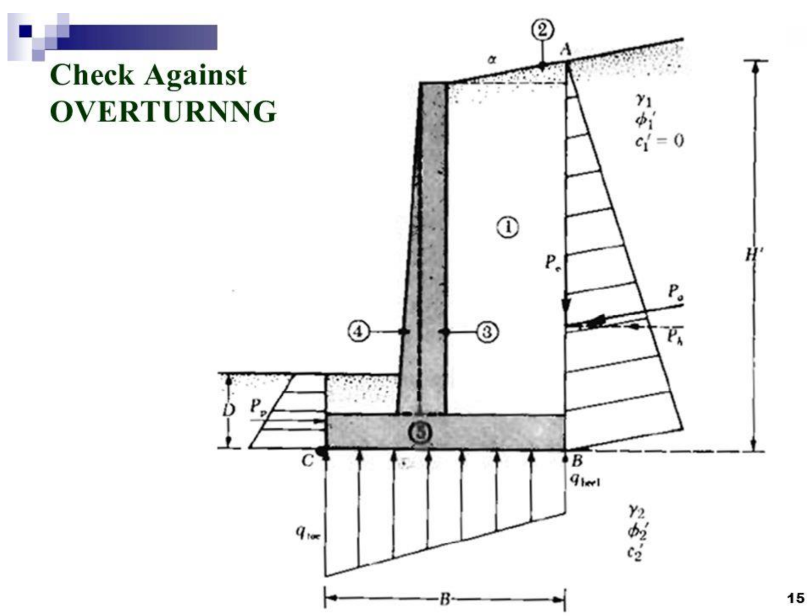
FOS - Base Sliding ≥ 1.5

FOS - Bearing Capacity ≥ 2.0 to 3.0

FOS Overturning:

Forces at Work Overturning

- **Active Earth Pressure Using H'**
- **Any load along top – q Resisting**
- **Weight of Soil (1 & 2) • Weight of Structure (3, 4 & 5)**
- **Bearing Capacity of Base**
- **Passive Pressure against Base.**



Note:

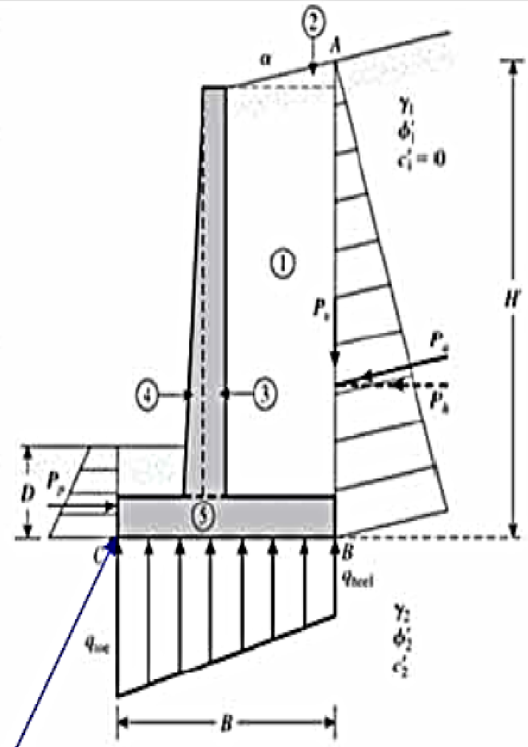
C: Point of Rotation

Moment Table:

Procedure for Calculating ΣM_R

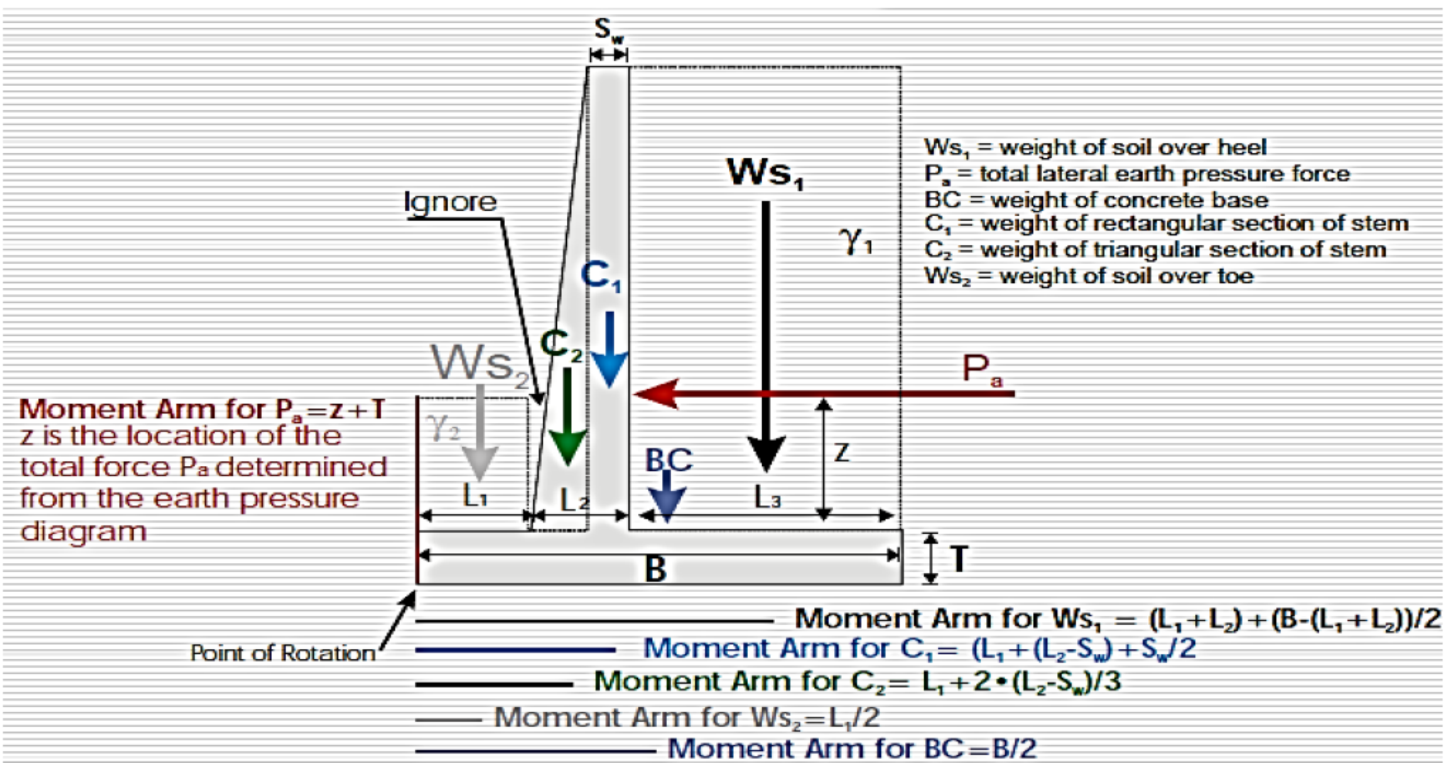
Section	Area	Weight per unit length of wall	Moment arm measured from C	Moment about C
1	2	3	4	5
1	A1	$W_1 = \gamma_1 \cdot A_1$	X_1	M_1
2	A2	$W_2 = \gamma_2 \cdot A_2$	X_2	M_2
3	A3	$W_3 = \gamma_3 \cdot A_3$	X_3	M_3
4	A4	$W_4 = \gamma_4 \cdot A_4$	X_4	M_4
5	A5	$W_5 = \gamma_5 \cdot A_5$	X_5	M_5
6	A6	$W_6 = \gamma_6 \cdot A_6$	X_6	M_6
		P_v	B	M_v
		ΣV		ΣM_R

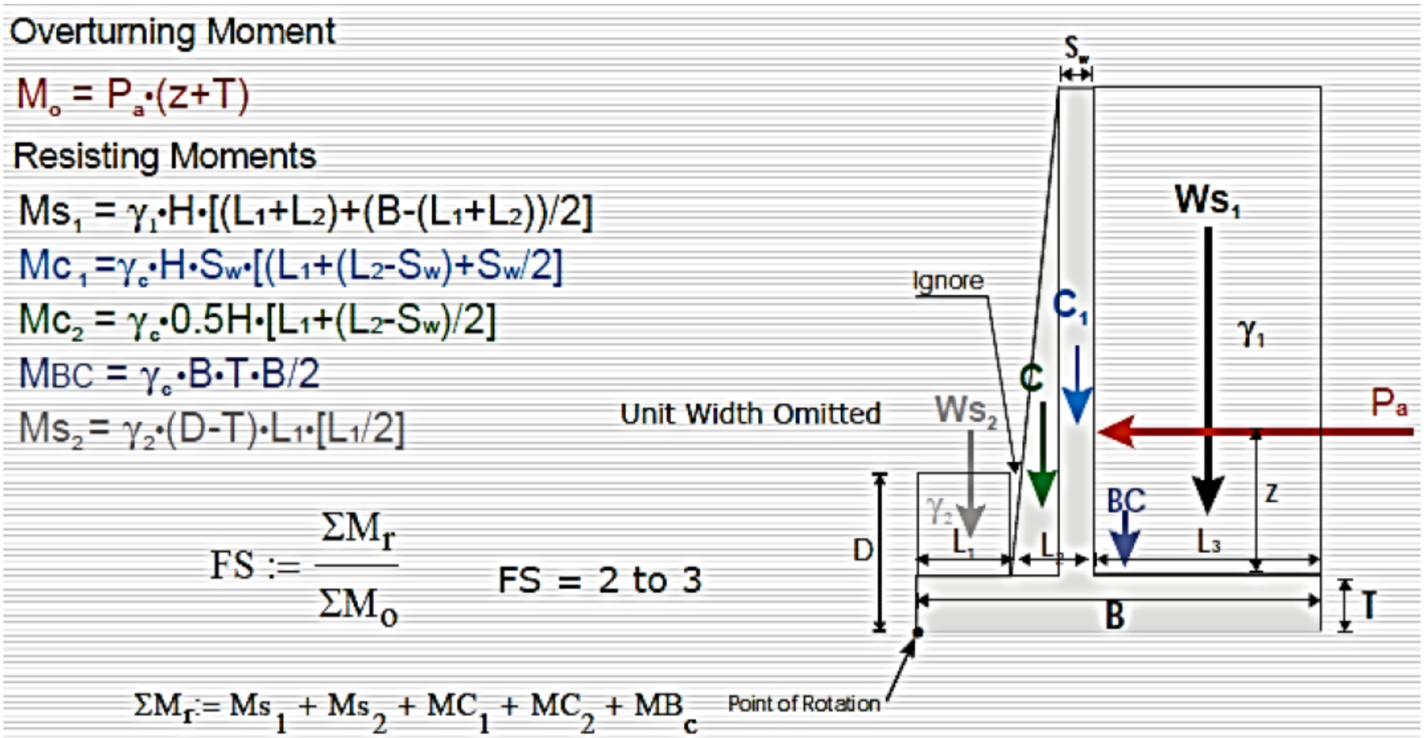
P_v = Vertical Component of P_a if sloped ground



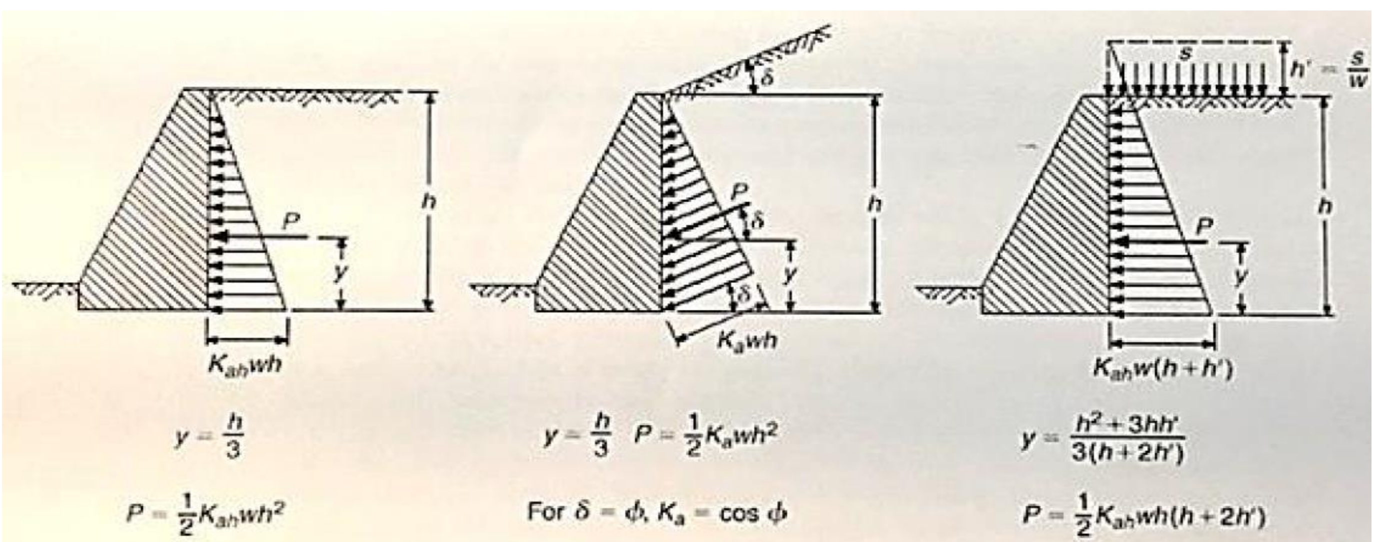
Always build the moment table

C = Point of Rotation





Determination of (P, Ph and Pv)



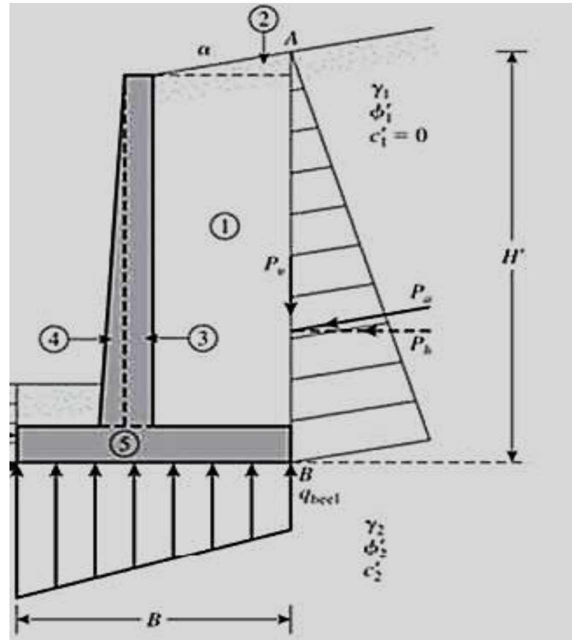
Determination of (P_a , P_h and P_v)

$P_a = \frac{1}{2} \gamma_1 H'^2 K_a$

Where:

H' ……….. Total height of the retaining wall

K_a ……….. Active Earth-Pressure Coefficient.



$$K_a = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}}$$

Rankine earth pressure for inclined backfill

$K_a = \tan^2(45 - \varphi/2)$ …..For horizontal Backfill

α ………Angle of backfill

slop φ………Angle of internal friction of the backfill soil Or,

K_a can be found according to the table below…….

K_a ……….. Active Earth-Pressure Coefficient.

[φ - Degree]

α degree	28	30	32	34	36	38	40
0	0.361	0.333	0.307	0.283	0.260	0.238	0.217
5	0.366	0.337	0.311	0.286	0.262	0.240	0.219
10	0.380	0.350	0.321	0.294	0.270	0.246	0.225
15	0.409	0.373	0.341	0.311	0.283	0.258	0.235
20	0.461	0.414	0.374	0.338	0.306	0.277	0.250
25	0.573	0.494	0.434	0.385	0.343	0.307	0.275

FOS Sliding:

Forces at Work

Driving Forces

• Σ of driving forces = Ph.

Resisting Forces

• R' = Σ V tan δ + B ca + Pp

Where:

δ ……. Angle of friction between the soil and the base slab.

δ = k1 φ2

Ca ……..Adhesion between the soil and the base slab.

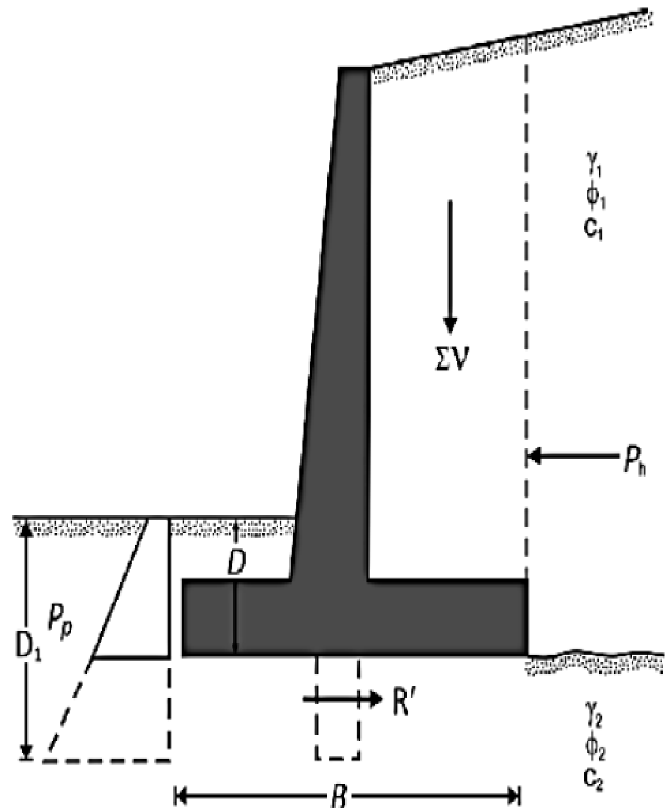
ca = k2 c2 K1 and k2 are ranged from (1/2 to 2/3)

B …….. Width of footing.

$Pp = ½ γ_2 D^2 Kp + 2 C2 D√Kp$

If Key is included…. Then

$Pp = ½ γ_2 D1^2 Kp + 2 C2 D1√Kp$



Where: D …… Depth of passive soil.

D1 ….. Depth of (passive soil and key).

Kp =tan2 (45 + ϕ2/2)

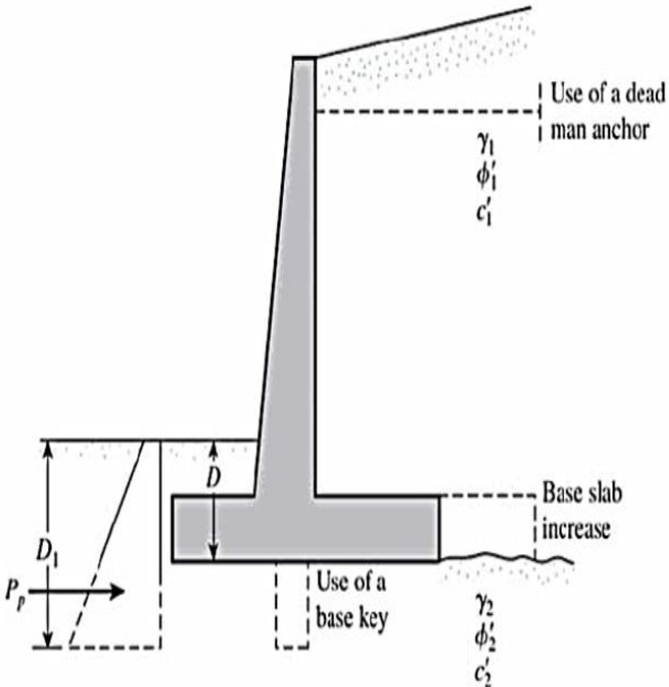
$$FOS_{Sliding} = \frac{[\sum V.\tan\delta + B.c_a + P_p]}{P_h}$$

FOS Sliding ≥ 1.5

Note:

If the factor of Safety (against sliding) was less than (1.50). In this case one of the following solutions can be adopted:

1- Increasing base slab.
2- Use of a base key.
3- Use of dead anchor.



FOS Bearing Capacity:

Resultant Force R acting on base

$$R := \Sigma V + P_a$$

Net Moment

$$M_{net} := \Sigma M_r - \Sigma M_o$$

Location of Resultant Force "R"

$$CE = \overline{X} := \frac{M_{net}}{\Sigma V}$$

So Eccentricity "e"

$$e := \frac{B}{2} - \overline{X} < B/6$$

Pressure on Base

$$q := \frac{\Sigma V}{A} := M_{net} \cdot \frac{y}{I} := \frac{(\Sigma V \cdot e)}{\frac{1}{12} \cdot 1 \cdot B^2}$$

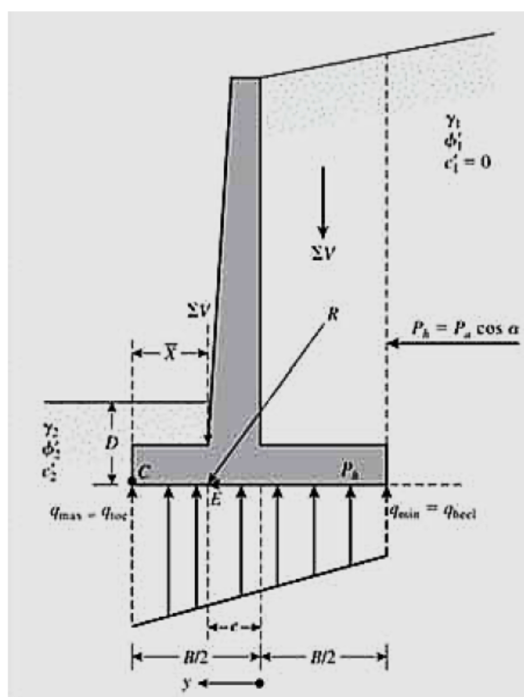
I=Moment of Inertia Per Unit Length

Max Pressure @ Toe

$$q_{max} := \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

Min Pressure @ Heel

$$q_{min} := \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$



In determination of (qu) ultimate bearing capacity of the supporting soil, two methods can be adopted:

1. Using the (q allowable), as mentioned and given by the soil investigation report.
2. Calculating the (qu) using the principle of soil mechanics, as follows…

Note you use only the lowest fill height over the footing.

$$q_u := \frac{\left(\gamma_2 \cdot B' \cdot N_\gamma\right)}{2} + c_2 \cdot N_c \cdot F_{cd} + \gamma_2 \cdot D \cdot N_q \cdot F_{qd}$$

$$B' := B - 2\,e$$

$$F_{cd} := 1 + 0.4 \cdot \frac{D}{B'}$$

$$F_{qd} := 1 + 2 \cdot \tan\phi_2 \cdot \left(1 - \sin\phi_2\right)^2 \cdot \frac{D}{B'}$$

$$F_{\gamma d} := 1$$

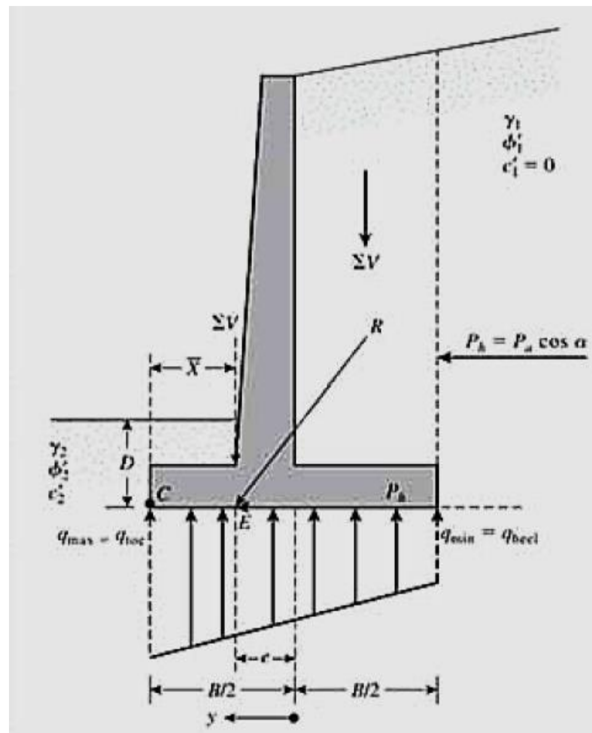
$$FS := \frac{q_u}{q_{max}}$$

$FS > 3$

For inclined surface, add inclination factors F_{ci}, F_{qi}, $F_{\gamma i}$

$$F_{\gamma i} := \left(1 - \frac{\psi^\circ}{\phi^\circ}\right)^2 \qquad F_{ci} := F_{qi} := \left(1 - \frac{\psi^\circ}{90^\circ}\right)^2$$

$$\psi := \tan^{-1}\left[\frac{P_a \cdot \cos(\alpha)}{\Sigma V}\right]$$



FOS = qu / q max ………………..≥ 3.0

FOS = q allowable / q max ………..≥1.0

CODE

COMMENTARY

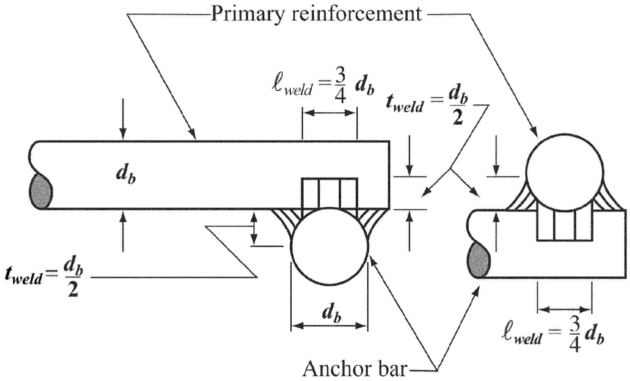


Fig. R11.9.6—Weld details used in tests of Reference 11.53.

forcement. The weld detail used successfully in the corbel tests reported in Reference 11.53 is shown in Fig. R11.9.6. The primary tension reinforcement should be anchored within the supporting column in accordance with the requirements of Chapter 12. See additional discussion on end anchorage in R12.10.6.

11.9.7 — Bearing area on bracket or corbel shall not project beyond straight portion of primary tension reinforcement, nor project beyond interior face of transverse anchor bar (if one is provided).

R11.9.7 — The restriction on the location of the bearing area is necessary to ensure development of the specified yield strength of the primary tension reinforcement near the load. When corbels are designed to resist horizontal forces, the bearing plate should be welded to the primary tension reinforcement.

11.10 — Special provisions for walls

11.10.1 — Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 11.12. Design for horizontal in-plane shear forces in a wall shall be in accordance with 11.10.2 through 11.10.9. Alternatively, it shall be permitted to design walls with a height not exceeding two times the length of the wall for horizontal shear forces in accordance with Appendix A and 11.10.9.2 through 11.10.9.5.

11.10.2 — Design of horizontal section for shear in plane of wall shall be based on Eq. (11-1) and (11-2), where V_c shall be in accordance with 11.10.5 or 11.10.6 and V_s shall be in accordance with 11.10.9.

11.10.3 — V_n at any horizontal section for shear in plane of wall shall not be taken greater than $0.83\sqrt{f_c'}\,hd$, where h is thickness of wall, and d is defined in 11.10.4.

R11.10 — Special provisions for walls

R11.10.1 — Shear in the plane of the wall is primarily of importance for shearwalls with a small height-to-length ratio. The design of higher walls, particularly walls with uniformly distributed reinforcement, will probably be controlled by flexural considerations.

R11.10.3 — Although the width-to-depth ratio of shearwalls is less than that for ordinary beams, tests[11.54] on shearwalls with a thickness equal to $\ell_w/25$ have indicated that ultimate shear stresses in excess of $0.83\sqrt{f_c'}$ can be obtained.

CODE

11.10.4 — For design for horizontal shear forces in plane of wall, d shall be taken equal to $0.8\ell_w$. A larger value of d, equal to the distance from extreme compression fiber to center of force of all reinforcement in tension, shall be permitted to be used when determined by a strain compatibility analysis.

11.10.5 — Unless a more detailed calculation is made in accordance with 11.10.6, V_c shall not be taken greater than $0.17\sqrt{f_c'}\,hd$ for walls subject to axial compression, or V_c shall not be taken greater than the value given in 11.3.2.3 for walls subject to axial tension.

11.10.6 — V_c shall be permitted to be the lesser of the values computed from Eq. (11-29) and (11-30).

$$V_c = 0.27\sqrt{f_c'}\,hd + \frac{N_u d}{4\ell_w} \qquad (11\text{-}29)$$

or

$$V_c = \left[0.05\sqrt{f_c'} + \frac{\ell_w\left(0.1\sqrt{f_c'} + 0.2\dfrac{N_u}{\ell_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}}\right]hd \qquad (11\text{-}30)$$

where ℓ_w is the overall length of the wall, and N_u is positive for compression and negative for tension. If $(M_u/V_u - \ell_w/2)$ is negative, Eq. (11-30) shall not apply.

11.10.7 — Sections located closer to wall base than a distance $\ell_w/2$ or one-half the wall height, whichever is less, shall be permitted to be designed for the same V_c as that computed at a distance $\ell_w/2$ or one-half the height.

11.10.8 — Where V_u is less than $0.5\phi V_c$, reinforcement shall be provided in accordance with 11.10.9 or in accordance with Chapter 14. Where V_u exceeds $0.5\phi V_c$, wall reinforcement for resisting shear shall be provided in accordance with 11.10.9.

11.10.9 — Design of shear reinforcement for walls

11.10.9.1 — Where V_u exceeds ϕV_c, horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s} \qquad (11\text{-}31)$$

COMMENTARY

R11.10.5 and R11.10.6 — Eq. (11-29) and (11-30) may be used to determine the inclined cracking strength at any section through a shearwall. Eq. (11-29) corresponds to the occurrence of a principal tensile stress of approximately $0.33\sqrt{f_c'}$ at the centroid of the shearwall cross section. Eq. (11-30) corresponds approximately to the occurrence of a flexural tensile stress of $0.5\sqrt{f_c'}$ at a section $\ell_w/2$ above the section being investigated. As the term

$$\left(\frac{M_u}{V_u} - \frac{\ell_w}{2}\right)$$

decreases, Eq. (11-29) will control before this term becomes negative. When this term becomes negative Eq. (11-29) should be used.

R11.10.7 — The values of V_c computed from Eq. (11-29) and (11-30) at a section located a lesser distance of $\ell_w/2$ or $h_w/2$ above the base apply to that and all sections between this section and the base. However, the maximum factored shear force V_u at any section, including the base of the wall, is limited to ϕV_n in accordance with 11.10.3.

R11.10.9 — Design of shear reinforcement for walls

Both horizontal and vertical shear reinforcement are required for all walls. The notation used to identify the direction of the disturbed shear reinforcement in walls was updated in 2005 to eliminate conflicts between the notation used for ordinary structural walls in Chapters 11 and 14 and the notation used for special structural walls in Chapter 21. The distributed reinforcement is now identified as being oriented parallel to either the longitudinal or transverse axis of the wall. Therefore, for vertical wall seg-

CHAPTER 14 — WALLS

CODE

14.1 — Scope

14.1.1 — Provisions of Chapter 14 shall apply for design of walls subjected to axial load, with or without flexure.

14.1.2 — Cantilever retaining walls are designed according to flexural design provisions of Chapter 10 with minimum horizontal reinforcement according to 14.3.3.

14.2 — General

14.2.1 — Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

14.2.2 — Walls subject to axial loads shall be designed in accordance with 14.2, 14.3, and either 14.4, 14.5, or 14.8.

14.2.3 — Design for shear shall be in accordance with 11.10.

14.2.4 — Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for each concentrated load shall not exceed center-to-center distance between loads, nor the bearing width plus four times the wall thickness.

14.2.5 — Compression members built integrally with walls shall conform to 10.8.2.

14.2.6 — Walls shall be anchored to intersecting elements, such as floors and roofs; or to columns, pilasters, buttresses, of intersecting walls; and to footings.

14.2.7 — Quantity of reinforcement and limits of thickness required by 14.3 and 14.5 shall be permitted to be waived where structural analysis shows adequate strength and stability.

14.2.8 — Transfer of force to footing at base of wall shall be in accordance with 15.8.

COMMENTARY

R14.1 — Scope

Chapter 14 applies generally to walls as vertical load carrying members. Cantilever retaining walls are designed according to the flexural design provisions of Chapter 10. Walls designed to resist shear forces, such as shearwalls, should be designed in accordance with Chapter 14 and 11.10 as applicable.

In the 1977 code, walls could be designed according to Chapter 14 or 10.15. In the 1983 code these two were combined in Chapter 14.

R14.2 — General

Walls should be designed to resist all loads to which they are subjected, including eccentric axial loads and lateral forces. Design is to be carried out in accordance with 14.4 unless the wall meets the requirements of 14.5.1.

CODE

14.3 — Minimum reinforcement

14.3.1 — Minimum vertical and horizontal reinforcement shall be in accordance with 14.3.2 and 14.3.3 unless a greater amount is required for shear by 11.10.8 and 11.10.9.

14.3.2 — Minimum ratio of vertical reinforcement area to gross concrete area, ρ_ℓ, shall be:

(a) 0.0012 for deformed bars not larger than No. 16 with f_y not less than 420 MPa; or

(b) 0.0015 for other deformed bars; or

(c) 0.0012 for welded wire reinforcement not larger than MW200 or MD200.

14.3.3 — Minimum ratio of horizontal reinforcement area to gross concrete area, ρ_t, shall be:

(a) 0.0020 for deformed bars not larger than No. 16 with f_y not less than 420 MPa; or

(b) 0.0025 for other deformed bars; or

(c) 0.0020 for welded wire reinforcement not larger than MW200 or MD200.

14.3.4 — Walls more than 250 mm thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

(a) One layer consisting of not less than one-half and not more than two-thirds of total reinforcement required for each direction shall be placed not less than 50 mm nor more than one-third the thickness of wall from the exterior surface;

(b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 20 mm nor more than one-third the thickness of wall from the interior surface.

14.3.5 — Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, nor farther apart than 450 mm.

14.3.6 — Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

COMMENTARY

R14.3 — Minimum reinforcement

The requirements of 14.3 are similar to those in previous codes. These apply to walls designed according to 14.4, 14.5, or 14.8. For walls resisting horizontal shear forces in the plane of the wall, reinforcement designed according to 11.10.9.2 and 11.10.9.4 may exceed the minimum reinforcement in 14.3.

The notation used to identify the direction of the distributed reinforcement in walls was updated in 2005 to eliminate conflicts between the notation used for ordinary structural walls in Chapters 11 and 14 and the notation used for special structural walls in Chapter 21. The distributed reinforcement is now identified as being oriented parallel to either the longitudinal or transverse axis of the wall. Therefore, for vertical wall segments, the notation used to describe the horizontal distributed reinforcement ratio is ρ_t, and the notation used to describe the vertical distributed reinforcement ratio is ρ_ℓ.

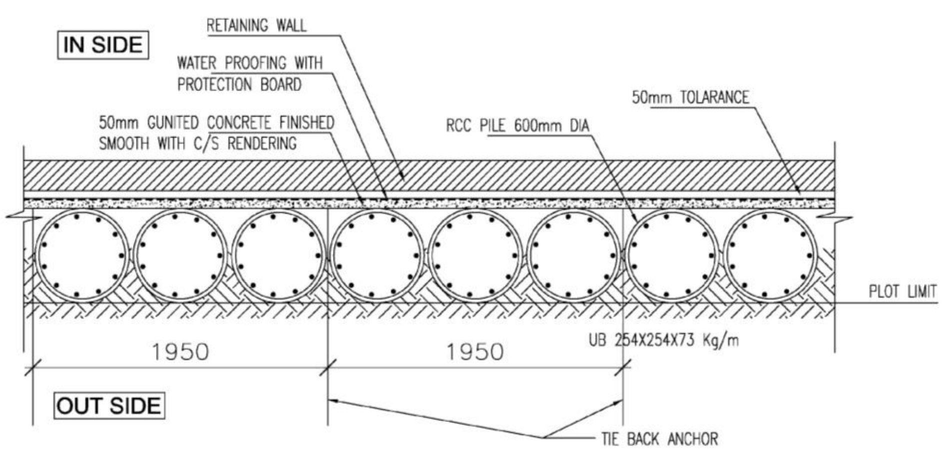
Shoring Piles :

Temporary Supporting of Excavation Sides of the construction site during foundation and Basement Works to prevent the failure of surrounding soil.

(Concrete pile wall):

Medium Excavation 2 or 3 Basement or more with Multi- Anchors

Contiguous Piles

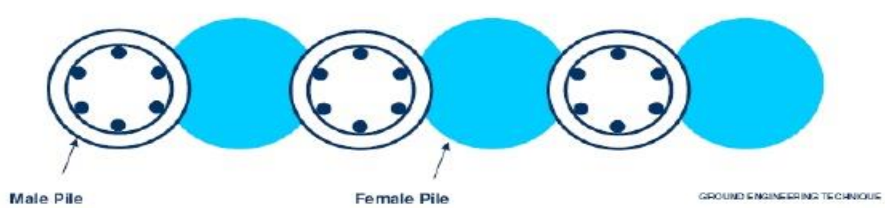


CONTIGUOUS PILES

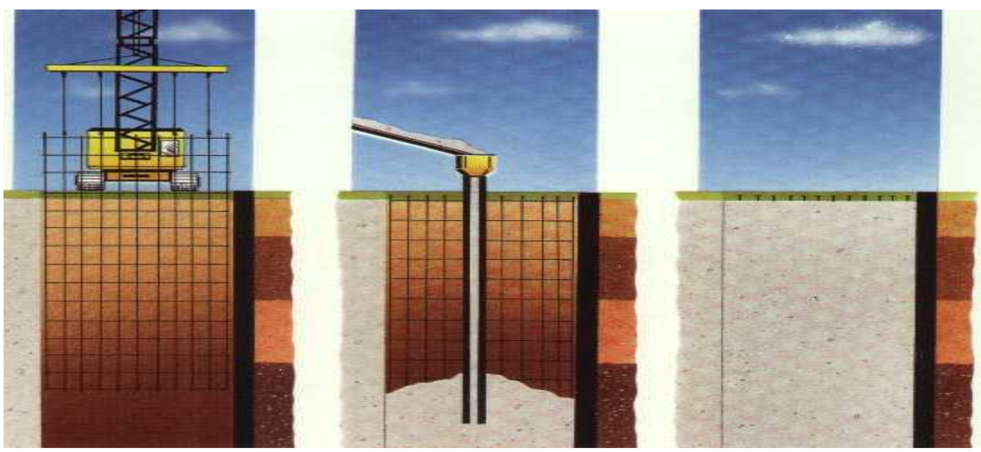
- In this technique the reinforced concrete unconnected piles are installed in the periphery of the proposed site.
- The contiguous piles wall is designed in such a way that it resists the lateral earth pressure of retained soil.
- The width of gap between piles varies between 50 and 500mm according to ground conditions.
- The dia. of these piles varies from 300 to 1200mm normally.
- The soil between two piles is stabilized by shotcreting.

SECANT PILES

- In this technique primary piles (PCC) are installed first with secondary piles (RCC) constructed in b/w primary piles when it gained some strength as shown.



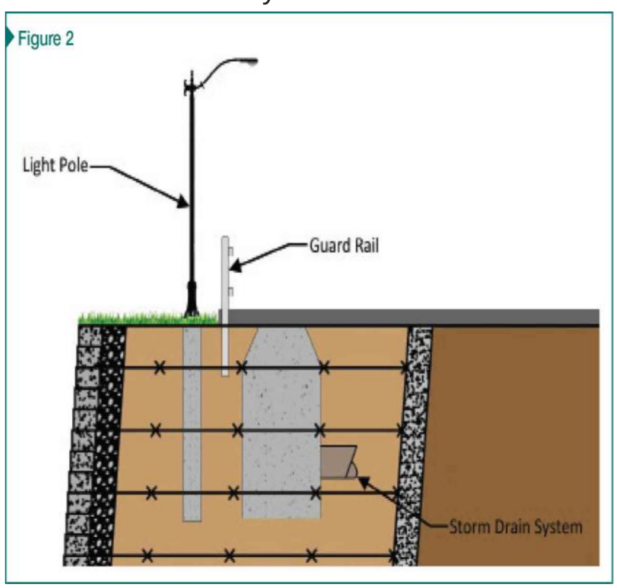
Diaphragm Walls

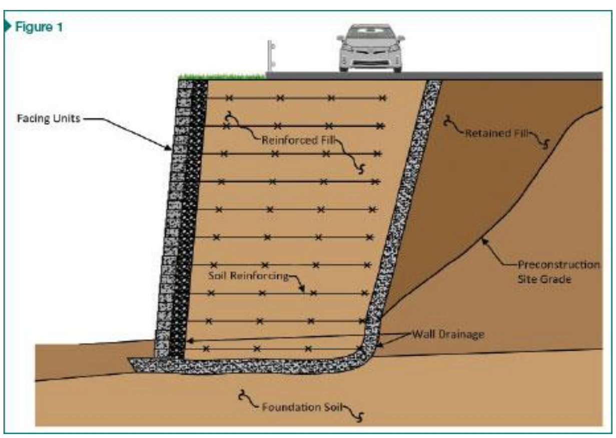


(b) Mechanically stabilized earth walls.

Mechanically stabilized earth (MSE or reinforced soil) is soil constructed with artificial reinforcing. It can be used for retaining walls, bridge abutments, seawalls, and dikes although the basic principles of MSE have been used throughout history; MSE was developed in its current form in the 1960s. The reinforcing elements used can vary but include steel and geosynthetics .

An MSE wall employs facing elements, soil, and reinforcement that work together to form a gravity-retaining structure (Figure 1). Compacted soil backfill is interlayered with soil reinforcement, together forming a reinforced-soil mass. In most systems, reinforcement is connected to facing elements. The MSE wall system relies on self-weight of the reinforced-soil mass to resist lateral pressures from earth, surcharges (*e.g.* vehicles, buildings), seismic events, and water.





Reference

- Foundation Analysis and Design – Fifth Edition Joseph E. Bowles
- Principles of Foundation Engineering – Seventh Edition Braja Das
- Basics of Retaining Wall Design – 10^{th} Edition Hugh Brooks & John p.Nielsen
- Design of Reinforced Concrete – Ninth Edition, Chapter 13 by JACK C.McCORMAC. RUSSELL H. BROWN
- ACI 318 M-05.
- Internet Sources.

Thank you

If instead of dowels the vertical stem bars are embedded into the footing, they should not extend up into the wall more than 8 ft or 10 ft before they are spliced because they are difficult to handle in construction and may easily be bent out of place or even broken. Actually, after examining Figure 13.21(a), you can see that such an arrangement of stem steel can sometimes be very advantageous economically.

The bending moment in the stem decreases rapidly above the base; as a result, the amount of reinforcing can be similarly reduced. It is to be remembered that these bars can be cut off only in accordance with the ACI Code development length requirements.

Example 13.3 illustrates the detailed design of a cantilever retaining wall. Several important descriptive remarks are presented in the solution, and these should be carefully read.

Example 13.3

Complete the design of the cantilever retaining wall whose dimensions were estimated in Example 13.2 and are shown in Figure 13.22 if $f'_c = 3000$ psi, $f_y = 60{,}000$ psi, $q_a = 4000$ psf, and the coefficient of sliding friction equals 0.50 for concrete on soil. Use ρ approximately equal to $0.18f'_c/f_y$ to maintain reasonable deflection control.

SOLUTION

The safety factors against overturning and sliding and the soil pressures under the heel and toe are computed using the actual unfactored loads.

$$\text{Safety factor against overturning} = \frac{149{,}456 \text{ ft-lb}}{70{,}560 \text{ ft-lb}} = 2.12 > 2.00 \quad \underline{\underline{\text{OK}}}$$

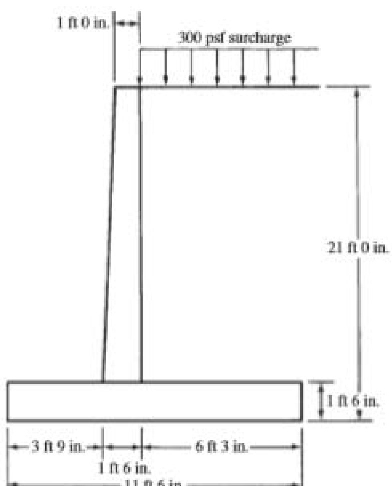


FIGURE 13.22 Dimensions of retaining wall for Example 13.3.

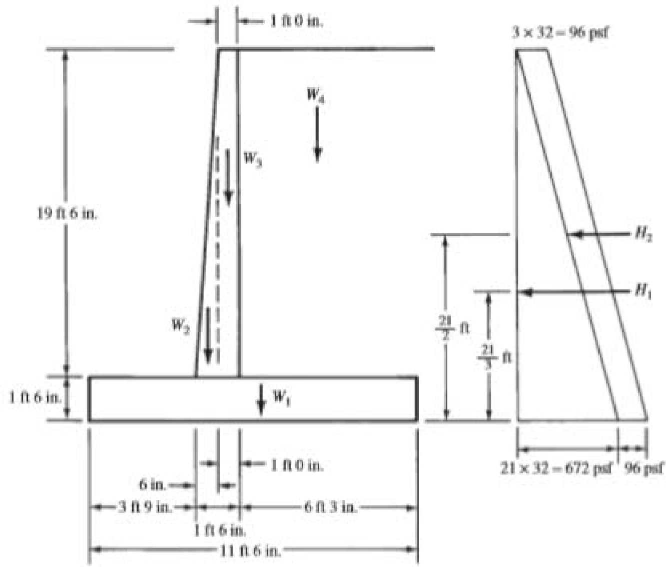


FIGURE 13.23 Forces acting on retaining wall for Example 13.3.

Safety Factor against Overturning (with Reference to Figure 13.23)

Overturning Moment		
Force	**Moment Arm**	**Moment**
$H_1 = (\frac{1}{2})(21\text{ ft})(672\text{ psf})$	$= 7056\text{ lb} \times 7.00\text{ ft}$	$= 49{,}392$ ft-lb
$H_2 = (21\text{ ft})(96\text{ psf})$	$= 2016\text{ lb} \times 10.50\text{ ft}$	$= \underline{21{,}168}$ ft-lb
Total		70,560 ft-lb

Righting Moment		
Force	**Moment Arm**	**Moment**
$W_1 = (1.5\text{ ft})(11.5\text{ ft})(150\text{ pcf})$	$= 2{,}588\text{ lb} \times 5.75\text{ft}$	$= 14{,}881$ ft-lb
$W_2 = (\frac{1}{2})(19.5\text{ ft})(\frac{6}{12}\text{ ft})(150\text{ pcf})$	$= 731\text{ lb} \times 4.08\text{ft}$	$= 2{,}982$ ft-lb
$W_3 = (19.5\text{ ft})(\frac{12}{12}\text{ ft})(150\text{ pcf})$	$= 2{,}925\text{ lb} \times 4.75\text{ft}$	$= 13{,}894$ ft-lb
$W_4 = (22.5\text{ ft})(6.25\text{ ft})(100\text{ pcf})$	$= \underline{14{,}062\text{ lb}} \times 8.37\text{ft}$	$= \underline{117{,}699}$ ft-lb*
	$R_v = 20{,}306$ lb	$M = 149{,}456$ ft-lb

* Includes surcharge.

Factor of Safety Against Sliding

Here the passive pressure against the wall is neglected. Normally it is felt that the factor of safety should be at least 1.5. If it is not satisfactory, a little wider footing on the heel side will easily take care of the situation. In addition to or instead of this solution, a key, perhaps 1 ft 6 in. × 1 ft 6 in. (size selected to provide sufficient development length for the dowels selected later in this design) can be used. Space is not taken here to improve this safety factor.

$$\text{Force causing sliding} = H_1 + H_2 = 9072 \text{ lb}$$

$$\text{Resisting force} = \mu R_v = (0.50)(20{,}306 \text{ lb}) = 10{,}153 \text{ lb}$$

$$\text{Safety factor} = \frac{10{,}153 \text{ lb}}{9072 \text{ lb}} = 1.12 < 1.50 \quad \underline{\underline{\text{No good}}}$$

Footing Soil Pressures

$R_v = 20{,}306$ lb and is located a distance $\bar{x}$ from the toe of the footing

$$\bar{x} = \frac{149{,}456 \text{ ft-lb} - 70{,}560 \text{ ft-lb}}{20{,}306 \text{ lb}} = \frac{78{,}896 \text{ ft-lb}}{20{,}306 \text{ lb}} = 3.89 \text{ ft} \qquad \underline{\underline{\text{Just inside middle third}}}$$

$$\text{Soil pressure} = -\frac{R_v}{A} \pm \frac{Mc}{I}$$

$$A = (1 \text{ ft})(11.5 \text{ ft}) = 11.5 \text{ ft}^2$$

$$I = \left(\frac{1}{12}\right)(1 \text{ ft})(11.5 \text{ ft})^3 = 126.74 \text{ ft}^4$$

$$f_{\text{toe}} = -\frac{20{,}306 \text{ lb}}{11.5 \text{ ft}^2} - \frac{(20{,}306 \text{ lb})(5.75 \text{ ft} - 3.89 \text{ ft})(5.75 \text{ ft})}{126.74 \text{ ft}^4}$$

$$= -1766 \text{ psf} - 1714 \text{ psf} = -3480 \text{ psf}$$

$$f_{\text{heel}} = -1766 \text{ psf} + 1714 \text{ psf} = -52 \text{ psf}$$

Design of Stem

The lateral forces applied to the stem are calculated using a load factor of 1.6, as shown in Figure 13.24.

Design of Stem for Moment

$$M_u = (H_1)(6.50 \text{ ft}) + (H_2)(9.75 \text{ ft}) = (9734 \text{ lb})(6.50 \text{ ft}) + (2995 \text{ lb})(9.75 \text{ ft})$$

$$M_u = 92{,}472 \text{ ft-lb}$$

Use

$$\rho = \text{approximately } \frac{0.18 f'_c}{f_y} = \frac{(0.18)(3000 \text{ psi})}{60{,}000 \text{ psi}} = 0.009$$

$$\frac{M_u}{\phi b d^2} = 482.6 \text{ psi (from Appendix A, Table A.12)}$$

$$bd^2 = \frac{(12 \text{ in/ft})(92{,}472 \text{ ft-lb})}{(0.9)(482.6 \text{ psi})} = 2555 \text{ in.}^3$$

$$d = \sqrt{\frac{2555 \text{ in.}^3}{12 \text{ in.}}} = 14.59 \text{ in.}$$

$$h = 14.59 \text{ in.} + 2 \text{ in.} + \frac{1 \text{ in.}}{2} = 17.09 \text{ in.} \quad \underline{\underline{\text{Say 18 in. } (d = 15.50 \text{ in.})}}$$

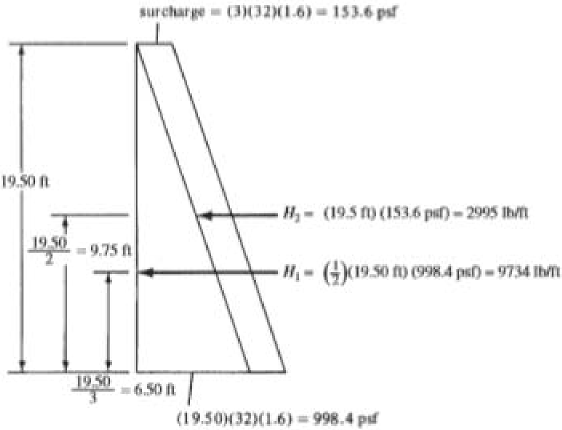


FIGURE 13.24 Lateral forces from backfill and surcharge.

$$\frac{M_u}{\phi bd^2} = \frac{(12 \text{ in/ft})(92{,}472 \text{ ft-lb})}{(0.90)(12 \text{ in.})(15.5 \text{ in.})^2} = 427.7 \text{ psi}$$

$$\rho = 0.00786 \text{ (from Appendix A, Table A.12)}$$

$$A_s = (0.00786)(12 \text{ in.})(15.5 \text{ in.}) = 1.46 \text{ in.}^2 \qquad \underline{\underline{\text{Use \#8 @ 6 in. (1.57 in.}^2)}}$$

$$\text{Minimum vertical } \rho \text{ by ACI Section 14.3} = 0.0015 < \frac{1.57 \text{ in.}^2}{(12 \text{ in.})(15.5 \text{ in.})} = 0.0084 \qquad \underline{\underline{\text{OK}}}$$

$$\text{Minimum horizontal } A_s = (0.0025)(12 \text{ in.})(\text{average stem } t)$$

$$= (0.0025)(12 \text{ in.})\left(\frac{12 \text{ in.} + 18 \text{ in.}}{2}\right) = 0.450 \text{ in.}^2$$

(say one-third inside face and two-thirds outside face)

$$\underline{\underline{\text{Use \#4 @ } 7\tfrac{1}{2} \text{ in. outside face and \#4 @ 15 in. inside face}}}$$

Checking Shear Stress in Stem

Actually, V_u at a distance d from the top of the footing can be used, but for simplicity:

$$V_u = H_1 + H_2 = 9734 \text{ lb} + 2995 \text{ lb} = 12{,}729 \text{ lb}$$

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd = (0.75)(2)(1.0)(\sqrt{3000} \text{ psi})(12 \text{ in.})(15.5 \text{ in.})$$

$$= 15{,}281 \text{ lb} > 12{,}729 \text{ lb} \qquad \underline{\underline{\text{OK}}}$$

Design of Heel

The upward soil pressure is conservatively neglected, and a load factor of 1.2 is used for calculating the shear and moment because soil and concrete make up the load.

$$V_u = (22.5 \text{ ft})(6.25 \text{ ft})(100 \text{ pcf})(1.2) + (1.5 \text{ ft})(6.25 \text{ ft})(150 \text{ pcf})(1.2) = 18{,}563 \text{ lb/ft}$$

$$\phi V_c = (0.75)(2)(1.0)(\sqrt{3000} \text{ psi})(12 \text{ in.})(14.5 \text{ in.}) = 14{,}295 \text{ lb} < 18{,}563 \text{ lb} \qquad \underline{\underline{\text{No good}}}$$

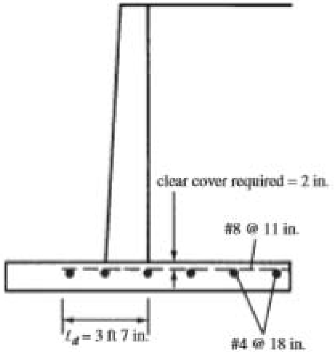


FIGURE 13.25 Heel reinforcing.

Try 24-in. Depth (d = 20.5 in.)

Neglecting slight change in V_u with different depth

$$\phi V_c = (0.75)(2)(1.0)(\sqrt{3000}\text{ psi})(12\text{ in.})(20.5\text{ in.})$$

$$= 20{,}211\text{ lb} > 18{,}563\text{ lb} \quad \underline{\underline{\text{OK}}}$$

$$M_u \text{ at face of stem} = (18{,}563\text{ lb})\left(\frac{6.25\text{ ft}}{2}\right) = 58{,}009\text{ ft-lb}$$

$$\frac{M_u}{\phi b d^2} = \frac{(12\text{ in/ft})(58{,}009\text{ ft-lb})}{(0.9)(12\text{ in.})(20.5\text{ in.})^2} = 153\text{ psi}$$

$$\rho = \rho_{\min}$$

Using $\rho = 0.00333$,

$$A_s = (0.00333)(12\text{ in.})(20.5\text{ in.}) = 0.82\text{ in}^2/\text{ft} \qquad \underline{\underline{\text{Use \#8 @ 11 in.}}}$$

ℓ_d required calculated with ACI Equation 12-1 for #8 top bars with $c = 2.50$ in. and $K_{tr} = 0$ is 43 in. < 72 in. available. $\underline{\underline{\text{OK}}}$

Heel reinforcing is shown in Figure 13.25.

Note: Temperature and shrinkage steel is normally considered unnecessary in the heel and toe. However, the authors have placed #4 bars at 18 in. on center in the long direction, as shown in Figures 13.25 and 13.27, to serve as spacers for the flexural steel and to form mats out of the reinforcing.

Design of Toe

For service loads, the soil pressures previously determined are multiplied by a load factor of 1.6 because they are primarily caused by the lateral forces, as shown in Figure 13.26.

$$V_u = 10{,}440\text{ lb} + 7086\text{ lb} = 17{,}526\text{ lb}$$

(The shear can be calculated a distance d from the face of the stem because the reaction in the direction of the shear does introduce compression into the toe of the slab, but this advantage

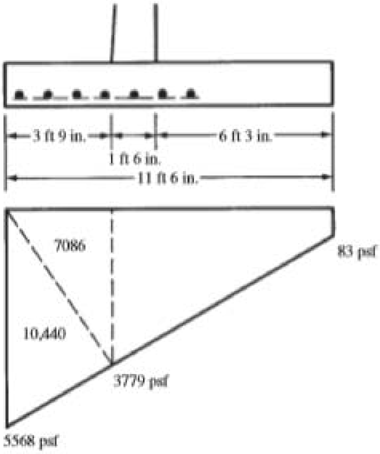


FIGURE 13.26 Soil reactions.

is neglected because 17,526 lb is already less than the 19,125 lb shear in the heel, which was satisfactory.)

$$M_u \text{ at face of stem} = (7086 \text{ lb})\left(\frac{3.75 \text{ ft}}{3}\right) + (10{,}440 \text{ lb})\left(\frac{2}{3} \times 3.75 \text{ ft}\right) = 34{,}958 \text{ ft-lb}$$

$$\frac{M_u}{\phi b d^2} = \frac{(12 \text{ in/ft})(34{,}958 \text{ ft-lb})}{(0.9)(12 \text{ in.})(20.5 \text{ in.})^2} = 92 \text{ psi}$$

$$\rho = \text{less than } \rho_{min}$$

Therefore, use

$$\frac{200}{60{,}000 \text{ psi}} = 0.00333$$

$$A_s = (0.00333)(12 \text{ in.})(20.5 \text{ in.}) = 0.82 \text{ in}^2/\text{ft}$$ Use #8 @ 11 in.

ℓ_d required calculated with ACI Equation 12-1 for #8 bottom bars with $c = 2.50$ in. and $K_{tr} = 0$ equals 33 in. < 42 in. available. OK

Toe reinforcing is shown in Figure 13.27.

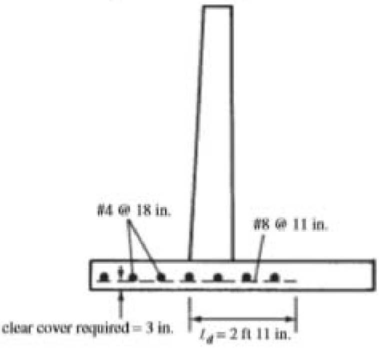


FIGURE 13.27 Toe reinforcing.

TABLE 13.1 Stem Design for Example 13.3

Distance from Top of Stem (ft)	M_u (ft-lb)	Effective Stem d (in.)	ρ	$A_{s\,required}$ (in^2/ft)	Bars Needed
5	2,987	11.04	Use $\rho_{min} = 0.00333$	0.44	#8 @ 18 in.
10	16,213	12.58	Use $\rho_{min} = 0.00333$	0.50	#8 @ 18 in.
15	46,080	14.12	0.00452	0.77	#8 @ 12 in.
19.5	92,472	15.50	0.00786	1.46	#8 @ 6 in.

Selection of Dowels and Lengths of Vertical Stem Reinforcing

The detailed selection of vertical bar lengths in the stem is omitted here to save space, and only a few general comments are presented. Table 13.1 shows the reduced bending moments up in the stem and the corresponding reductions in reinforcing required.

After considering the possible arrangements of the steel in Figure 13.21 and the required areas of steel at different elevations in Table 13.1, the authors decided to use dowels for load transfer at the stem base.

Use #8 dowels at 6 in. extending 33 in. down into footing and key.

If these dowels are spliced to the vertical stem reinforcing with no more than one-half the bars being spliced within the required lap length, the splices will fall into the class B category (ACI Code 12.15), and their lap length should at least equal $1.3\ell_d = (1.3)(33) = 43$ in. Therefore, two dowel lengths are used—half 3 ft 7 in. up into the stem and the other half 7 ft 2 in.—and the #7 bars are lapped over them, half running to the top of the wall and the other half to middepth. Actually, a much more refined design can be made that involves more cutting of bars. For such a design, a diagram comparing the theoretical steel area required at various elevations in the stem and the actual steel furnished is very useful. It is to be remembered (ACI Code 12.10.3) that the bars cut off must run at least a distance d or 12 diameters beyond their theoretical cutoff points and must also meet the necessary development length requirements.

13.11 Cracks and Wall Joints

Objectionable horizontal cracks are rare in retaining walls because the compression faces are the ones that are visible. When they do occur, it is usually a sign of an unsatisfactory structural design and not shrinkage. In Chapter 6 of this book, the ACI procedure (Section 10.6) for limiting crack sizes in tensile zones of one-way beams and slabs was presented. These provisions may be applied to vertical retaining wall steel. However, they are usually thought unnecessary because the vertical steel is on the earth side of the wall.

Vertical cracks in walls, however, are quite common unless sufficient construction joints are used. Vertical cracks are related to the relief of tension stresses because of shrinkage, with the resulting tensile forces exceeding the longitudinal steel capacity.

Construction joints may be used both horizontally and vertically between successive pours of concrete. The surface of the hardened concrete can be cleaned and roughened, or keys can be used as shown in Figure 13.28(a) to form horizontal construction joints.

If concrete is restrained from free movement when shrinking—for example, by being attached to more rigid parts of the structure—it will crack at points of weakness. Contraction joints are weakened places constructed so that shrinkage failures will occur at prepared locations. When the shrinkage tensile stresses become too large, they will pull these contraction