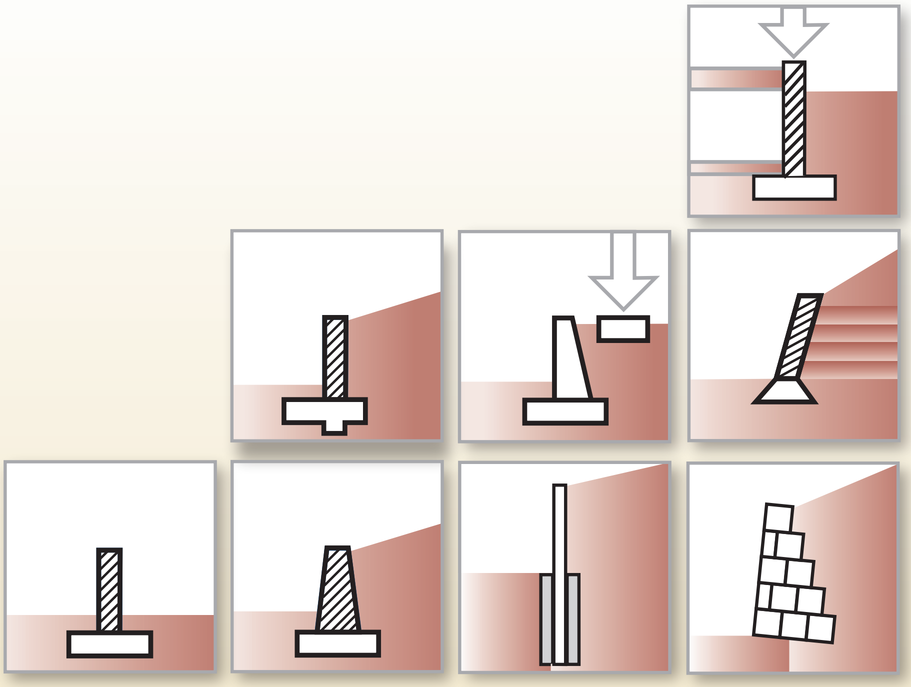
Basics of Retaining Wall Design

10th Edition



A Design Guide for Earth Retaining Structures

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Contents at a glance:

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Note !!

This document contains only selected pages from the full document.

The intent is to give you an idea of the content, quality, and illustrations of the full book.

Thank you for taking look inside!

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WHY THIS BOOK?

For The User

This book is intended to cover and explain design practices and building code requirements for the design of earth retaining structures. It is for both the practicing engineer who has become a bit rusty on this complex subject, and for the engineering student who has already acquired a basic knowledge of statics, soil mechanics, and the design of simple masonry and concrete structures. Design review agencies will also find it a useful reference.

Why It Was Written

The design of retaining walls is not an every-day design task. During my many years of providing technical support for Retain Pro software it became increasingly apparent that many engineers infrequently design retaining walls and need some brushing-up, particularly on code requirements. It is also for the civil/structural engineering student to supplement other texts with current design practices and building code requirements.

Although there is a considerable amount of technical information available, it is widely scattered in numerous textbooks and technical papers on soil mechanics, foundation engineering, concrete design, masonry design, and in all sorts of related topics. Despite the many references, a single volume on retaining wall design for the professional practitioner and civil engineering student could not be found. Hence this attempt to condense, simplify and compile information from many sources, including my own experience, into this book. Hopefully, it will ease your comfort level when designing retaining walls and give you a good overview of the process. For those who want to dig deeper into particular topics a comprehensive bibliography is attached in Appendix H.

Scope of This Book

This book treats design practices for most types of earth retaining structures: conventional cantilevered retaining walls: restrained (basement) walls, gravity walls, and segmental retaining walls both gravity and with geogrids. Other topics include sheet pile walls, tilt-up retaining walls, soldier pile walls, gabion walls, counterfort walls, pilaster walls and walls with pile or pier foundations. A review of basic soil mechanics is also included in this book and the appendix offers useful information including a glossary of terminlogy.

We'll welcome your feedback. You can contact us at hbrooks@hbapublications.com.

Hugh Brooks, PE, SE
October, 2013

PREFACE TO THE TENTH EDITION

This tenth edition, like its predecessors, become necessary because building codes change – seismic design requirements for example – and as we receive suggestions for topics to be added or expanded. To respond, successive editions have been updated, corrections made and additional materials added. All design examples have been reviewed, corrections made where needed and explanations added. A glossary of terminology has been added to this edition.

Another effort for this edition was to explain many of the topics in more detail so it will be helpful to the engineering student who may be using it as a text. We believe civil engineering students can benefit by learning building code requirements and design practices used by experienced engineers.

This series of editions began in 1996 as a modest companion manual to accompany my work developing Retain Pro software. In successive editions it has grown from 93 pages to 250 in this tenth edition, with many thousands of copies in print.

My objective in each edition was to cover basic design principles and practices in a concise, readable, manner. This book is not an in-depth treatment of the design of retaining structures. Earth retaining structures and soil mechanics are far too complex a subject to treat in a single concise volume. There are dozens of foundation engineering texts and countless technical papers available for review, and of course there is the Internet. However, finding what you need is time consuming; hence this compendium. The challenge was to decide what to put in and what to leave out and to put in the most helpful things a designer – and an advanced engineering student -- need to know to design most types of earth retaining structures. Surely there will be omissions and errors, but the intent is that you will find this book helpful in your practice.

We express appreciation to the many of you who have offered valuable suggestions, corrected errors, read portions of the draft, and faxed informative articles and excerpts from technical papers. My thanks to each of you. We will all benefit from your input.

And as for the Ninth Edition it is my good fortune and privilege to be joined by co-author John P. Nielsen, Ph.D, a civil and geotechnical engineer with a distinguished career in academia and geotechnical consulting practice. He brings many years experience to enhance and expand the scope of this and the previous edition.

We hope this edition will be helpful in your practice and informative for the engineering student.

And as always, your comments and suggestions will be welcome.

Hugh Brooks, P.E., S.E.
John P. Nielsen, P.E., G.E., Phd

1. ABOUT RETAINING WALLS: TERMINOLOGY

Evolution of Retaining Structures

In the year one-million BC, or thereabouts, an anonymous man, or woman, laid a row of stones atop 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 18^{th} and 19^{th} 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.

A Definition:

A retaining wall is any constructed wall that restrains soil or other material at locations having an abrupt change in elevation.

Types of Retaining Structures

There are many types of structures used to retain soil and other materials. Listed below are the types of earth retaining structures generally used today. The design of these will be discussed in later chapters.

Cantilevered retaining walls

These walls which retain earth by a wall cantilevering up from a footing are the most common type of retaining walls in use today. These walls are classified as "yielding" as they are free to rotate (about the foundation) because of the lack of any lateral restraint. Cantilevered retaining walls are generally made of masonry or concrete, or both, but can also take other forms as will be described.

Types of Cantilevered Retaining Walls Include:

Masonry or concrete walls

The stem of a masonry wall is usually constructed of either 8" or 12" deep concrete masonry block units. The cells are partially or solid grouted, and are vertically reinforced. An eight-inch block is generally adequate to retain up to about six feet, and a twelve-inch block up to ten to twelve feet.

The stems of a concrete wall must be formed, and can be tapered for economy, usually with the taper on the inside (earth side) to present a vertical exposed face.

Hybrid walls, with both concrete and masonry, can also be constructed using formed concrete at the base, where higher strength is required, then changing to masonry higher up the wall.

A variation for masonry cantilever walls uses spaced vertical pilasters (usually of square masonry units) with in-filled walls of lesser thickness, usually 6" masonry. The pilasters cantilever up from the footing and are usually spaced from four to eight feet on center. These walls are usually used where lower walls are needed – under about six feet high.

Counterfort retaining walls

Counterfort cantilevered retaining walls incorporate wing walls projecting upward from the heel of the footing into the stem. The thickness of the stem between counterforts is thinner (than for cantilevered walls) and spans horizontally, as a beam, between the counterfort (wing) walls. The counterforts act as cantilevered elements and are structurally efficient because the counterforts are tapered down to a wider (deeper) base at the heel where moments are higher. The high cost of forming the counterforts and the infill stem walls make such walls usually not practical for walls less than about 16 feet high. See Figure 1.1.

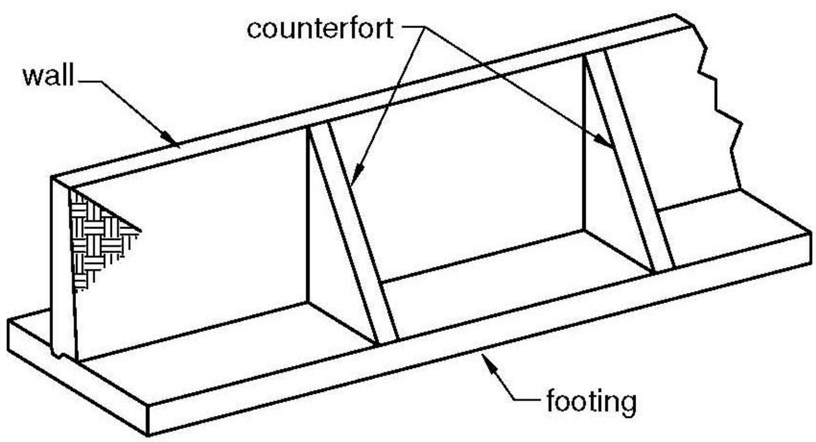

Figure 1.1 Counterfort retaining wall

Buttress retaining walls

These are similar to counterfort walls, but the wings project from the outside face of the wall. Such walls are generally used in those cases where property line limitations on the earth retention side do not allow space for the large heel of a traditional cantilevered retaining wall.

Gravity retaining walls

This type of wall depends upon the dead load mass of the wall for stability rather than cantilevering from a foundation.

Stacked and mortar-bonded stone, rubble, or rock walls

These are usually gravity walls relegated to landscaping features with retaining less than about four feet high. Engineering for such walls is limited, or none at all, and rules-of-thumb prevail (such as a retained height not more than two or three times the base width). Higher walls need engineering to evaluate overturning, sliding, soil bearing and to verify that flexural tension does not exist within the wall (or only as allowed by code for material used) because these walls are generally unreinforced.

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.

Tilt-up concrete retaining walls

Tilt-up concrete construction has been successfully used for retaining walls, either cantilevered or restrained at the top. These site-cast panels are set on concrete pads at panel ends, with the reinforcing projecting out from the bottom. A continuous concrete footing is then cast under the wall to complete the construction. Tilt-up walls are economical for higher walls, but sufficient space is needed to cast the panels.

Segmental retaining walls (SRWs)

Many manufacturers offer various systems of stacked segmental concrete units, steel bins, or other devices that retain soil by stacking individual components. Most are patented systems that are typically battered (sloped backward), primarily to reduce lateral soil pressure, thus requiring a minimal foundation. Reinforced concrete footings, steel reinforcing, or mortar are not used. Stability of SRW gravity walls depends solely upon the dead weight resisting moment exceeding the lateral soil pressure overturning moment. To attain greater heights – up to 40 feet and more – SRW's also utilize mechanically stabilized earth (MSE), also called reinforced earth, whereby geosynthetic fabric layers are placed in successive horizontal layers of the backfill to achieve an integral soil mass that increases resistance to overturning and horizontal sliding. A variety of facing block configurations and surface colors and textures are available from many manufacturers.

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.

Sheet pile and bulkhead walls

These are generally waterfront structures such as at docks and wharves, but steel sheet piling is also used for temporary shoring on construction sites. Steel sheet units configured for stiffness or concrete panels are driven into the soil to provide lateral support below the base of the excavation or the dredge line. Sheet pile walls cantilever upward to retain earth or are restrained at or near the top by either a slab-on-grade or tiebacks.

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 "non-yielding" 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 (tieback) walls

Anchors or tiebacks are often used for higher walls where a cantilevered wall may not be economical. Restraint is achieved by drilling holes and grouting inclined steel rods as anchors into the zone of earth behind the wall beyond the theoretical failure plane in the backfill. The anchors can be placed at several tiers for higher walls, and can be post-tensioned rods grouted into drilled holes, or non-tensioned rods grouted into the drilled holes. The latter are also known as soil nails.

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.

Cantilevered Retaining Wall Terminology

Cantilevered retaining walls have unique descriptive terminology as illustrated below:

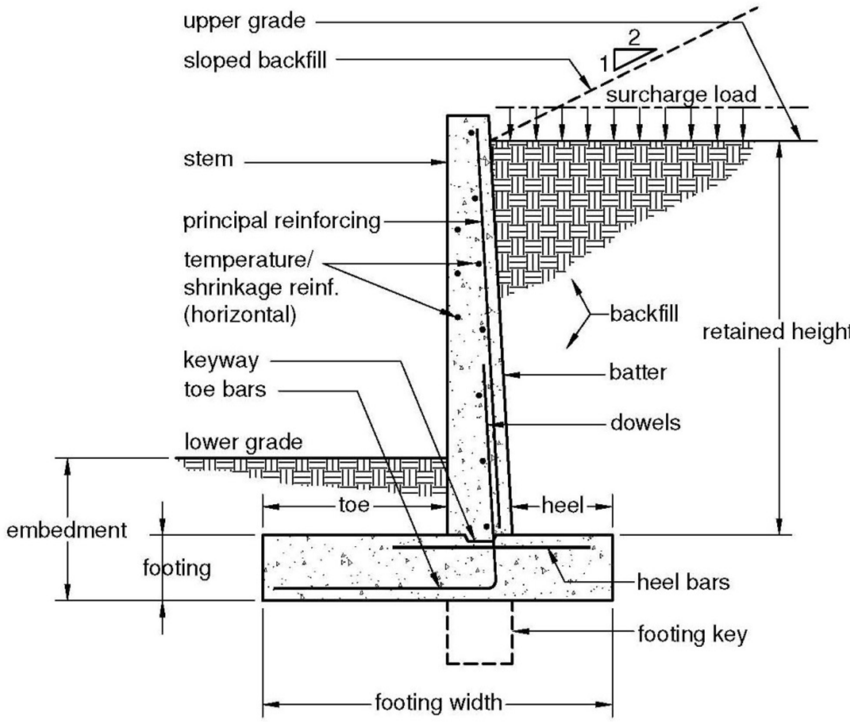

Figure 1.2 Retaining wall terminology

2. DESIGN PROCEDURE OVERVIEW

The Four Primary Concerns for the Design of Nearly any Retaining Wall are:

1. That it has an acceptable Factor of Safety with respect to overturning.
2. That it has an acceptable Factor of Safety with respect to sliding.
3. That the allowable soil bearing pressures are not exceeded.
4. That the stresses within the components (stem and footing) are within code allowable limits to adequately resist imposed vertical and lateral loads. It is equally important that it is constructed according to the design.

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

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 (1,000 psf to 3,000 psf)
* Passive pressure (150 to 350 pcf)
* Active earth pressure (30 pcf to 55 pcf)
* Coefficient of friction (.25 to .40)
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 (110 to 130 pcf)
Concrete and masonry allowable stresses (usually used values in parentheses – also see Appendix I – Notations & Symbols).

f'_c (2,000 psi to 4,000 psi)
f_y (60,000 psi)
f_s (24,000 psi)
f'_m (1,500 psi)
f_r (145 psi to 178 psi, strength design)

* These values are usually given in the geotechnical report.

When you have gathered this information, you're ready to start.

Basic Design Principals for Cantilevered Walls

Stability requires that a cantilevered retaining wall resists both overturning and sliding, and material stresses including the allowable soil bearing that must be within acceptable values.

To resist forces tending to overturn the wall (primarily the lateral earth pressure against the back of the wall), the wall must have sufficient weight, including the soil above the footing, such that the resisting moments are greater than the overturning moments. The safety factor for overturning should be at least 1.5 – some codes require more.

To resist sliding, the weight of the wall plus the weight of the soil above the footing plus vertical loads on the wall and any permanent surcharges multiplied by the coefficient of friction between the foundation soil and the bottom of the footing, plus the passive pressure resistance force at the front of the wall, must be sufficient to resist the lateral force pushing on the wall. The recommenced safety factor against sliding is 1.5. If the soil is cohesive, the coefficient of friction is replaced by the adhesive bond (see page 20) of the cohesion between the footing and soil, in psf.

The stem must be designed to resist the bending caused by earth pressures, including the effect of surcharges placed behind the wall, seismic or wind if applicable, impact loads, or axial loads acting eccentrically on the wall. The maximum bending and shear stresses in a cantilevered wall will be at the bottom of the stem. Each of these subjects will be discussed later.

Figure 2.1 is a free-body force diagram illustrating stability forces on a cantilevered wall.

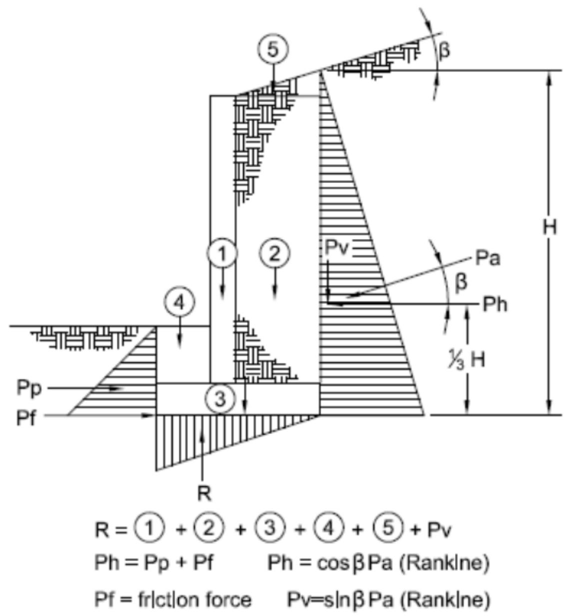

Figure 2.1 Free-body of cantilevered retaining wall

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

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. See Figure 8.4.
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*

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.

All these topics will be discussed later.

3. 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 interparticle 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, ϕ, 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:

$$s = c + p \tan \phi$$

s = shear strength; p = effective normal stress and c = cohesion, both usually expressed in psf; and ϕ = effective angle of internal friction.

4. 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.

5. FORCES AND LOADS ON RETAINING WALLS

Determination of Loads and Forces

The design of retaining walls may include any or all of the following (each will be discussed in the text that follows):

- Lateral earth pressure
- Axial loads
- Adjacent footing loads
- Surcharge loads
- Impact forces
- Wind on projecting stem
- *Seismic wall self-weight forces and seismic earth pressure force
  *Discussed in Chapter 6

Lateral Earth Pressures

The purpose of a retaining wall is to retain soil and to resist the lateral pressure of the soil against the wall. Most lateral pressure theories are based upon the sliding soil wedge theory. This, in simple terms, is based upon the assumption that if the wall is suddenly removed, a triangular wedge of soil will slide down along a rupture plane, and it is this wedge of soil that the wall must retain. The development of the soil wedge theory was discussed in Chapter 3. There are two basic equations for computing lateral earth pressures: the Coulomb equation and the Rankine equation.

The Coulomb Equation

The Coulomb Equation where K_a is the coefficient of active pressure, which takes into account backfill slope, friction angle at wall face, angle of rupture plane and angle of internal friction:

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2\alpha \,\sin(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\,\sin(\phi - \beta)}{\sin(\alpha - \delta)\,\sin(\alpha + \beta)}}\right]^2}$$

$$K_a\,(\text{horiz.}) = K_a \cos\delta \quad \text{if } \alpha = 90°$$

β = Angle of backfill slope
ϕ = Angle of internal friction of the soil
α = Wall slope angle from horizontal (90° for vertical face, α = < 90° if the back of wall is battered outward or > 90° if wall battered inward)
δ = Angle of friction between soil and wall (usually assumed to be 2/3ϕ to 1/2/ϕ)

Figure 5.1 The Coulomb equation

The Coulomb equation should only be used for gravity, segmental, gabion, and cantilevered walls having a short heel dimension. The reason is that the Coulomb equation includes a soil-to-wall friction angle, designated δ, which assumes the moving soil mass contacts the wall face and activates a shear resistance as the wall deflects. This friction angle δ is generally assumed to be between 0.5 and 0.7 times the phi (ϕ) angle. For the case of a cantilevered wall with a larger heel dimension the soil between the stem and the failure plane can be considered a rigid mass, then δ in the Coulomb equation or Mononobe-Okabe equation (discussed in the seismic chapter) can be taken a equal to ϕ because with a cantilevered wall the soil above the heel will move in mass with the wall so that wall friction cannot develop, the failure plane being through the heel of the footing.

If the backfill is level, the inside wall face is vertical, and if zero friction is assumed between the soil and wall, then the Coulomb equation reduces to the familiar Rankine equation:

$$K_a = (1 - \sin \phi) / (1 + \sin \phi)$$

The Rankine Equation

The Rankine equation is a simplified version of the Coulomb equation that does not take into account wall batter or friction at the wall-soil interface. As such, it is a conservative approach to the design of retaining walls. An example of its use will be described later for both the Coulomb and Rankine equations. For the case for vertical walls with a level backfill and zero wall friction, the lateral pressure factor K_a will be the same by either approach

Rankine's approach was to evaluate the stress at a point in the backfill by using Mohr's circle concepts to obtain the minimum lateral stress at a point in the backfill. The minimum lateral stress corresponds to the "active" case. Integration of that stress with respect to depth leads to a second-order equation (the well-known triangular distribution) for the total lateral force against the wall.

The use of the Rankine approach is recommended for most cantilevered retaining wall designs. It is conservative because it predicts a larger active force than that of Coulomb. It's also simpler to calculate for most walls, and easily handles sloping backfills and surcharge loads.

The Rankine Equation for active pressure:

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

$$K_a = (\text{horiz.}) = K_a \cos\beta$$

β = Angle of backfill slope
ϕ = Angle of internal friction of the backfill soil

Figure 5.2 The Rankine equation

If the backfill is level the Rankine equation can be written as: $K_a = \tan^2\left(45 - \frac{\phi}{2}\right)$ or $= \frac{1 - \sin\phi}{1 + \sin\phi}$

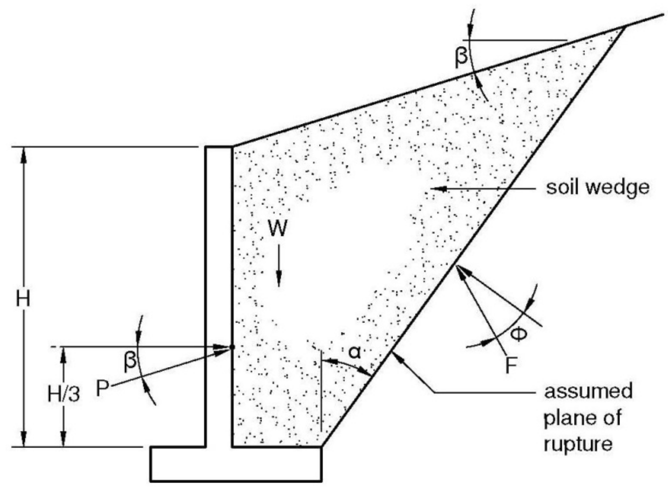

Figure 5.3 Rankine free-body of lateral forces on stem

Example: Assume: $\phi = 34°$, $\beta = 26.6°$ (2:1 slope)

Then $K_a = \dfrac{\cos 26.6 - \sqrt{\cos^2 26.6 - \cos^2 34}}{\cos 26.6 + \sqrt{\cos^2 26.6 - \cos^2 34}} \cos 26.6$

$= 0.41$

If the backfill is sloped you need to convert K_a to its horizontal component for computing stem moments and overturning.

Therefore K_a horiz. = 0.41 x cos 26.6 = 0.37 and corresponding horizontal equivalent fluid weight of the soil = 0.37 x say 110 pcf = 40 pcf for a 2:1 backfill slope (= 26.6°).

6. EARTHQUAKE (SEISMIC) DESIGN

Earthquakes – An Overview

Although our planet Earth is an incomprehensible 4.5 billion years old it is still cooling and adjusting. The tectonic plates (tectonic from the Latin: "building") that wrap our earth continue to float, move, and rotate, as in past eons, to shape our topography and build mountains. And cause earthquakes as they lurch along their boundaries.

Earthquakes can occur anywhere. However, in the US the West Coast is most vulnerable as the Pacific tectonic plate, which covers the entire Pacific Rim, rotates counter-clockwise, northward along the West Coast, moving about an inch per year as it grinds past its boundary with the easterly North American plate. This movement is primarily along California's infamous San Andreas Fault (so named for the community it passes near San Francisco) and is responsible for the numerous stress-relieving earthquake jolts occurring daily on the many associated faults.

In California there are over 400 measurable earthquakes each week Many are never felt (those under magnitude 3.0 are rarely felt). Fortunately, few cause damage. Some larger earthquakes in California include:

1908, San Francisco, 7.2 (estimated)
1933, Long Beach, 6.4 (estimated)
1940, Imperial Valley, 7.0
1952, Kern County, 7.3
1971, San Fernando, 6.7
1987, Whittier Narrows, 5.9
1989, Loma Prieta, 6.9
1992, Landers, 7.3
1994, Northridge, 6.7
2010, Borrego Springs, 5.4

20xx, The California "Big One": who knows when or where?

Note: Reports of earthquakes prior to 1935 use estimated Richter magnitudes.

Ironically, however, one of the largest earthquake events occurred mid-continent, near the town of Madrid on the Mississippi River midway between St. Louis and Memphis. Known as the New Madrid faults, there were a series of earthquakes in 1811-12 with estimated magnitudes of ≈ 7.7. They were felt as far away as NYC and reportedly rang church bells as far away as Boston.

The largest recorded in North America: 1964, Alaska, 9.2

The largest earthquake ever recorded worldwide was in Chile in 1960 with a magnitude of 9.5.

More recently on 3/11/2011 off-shore Japan, magnitude 8.9.

The term "magnitude" as used in the above list, and in the media, was developed in 1935 by Caltech professor Charles Richter and colleagues and bears his name "Richter Scale". They used data from a seismograph to describe a specific earthquake in terms of seismic energy released. It is a logarithmic scale (to the base 10) whereby a magnitude 5 earthquake releases about ten times the energy of that of a magnitude 4 ($5^{10} / 4^{10} \approx 10$). This measure is popular with the media but does not have a direct correlation to ground acceleration that is used to determine the seismic force for the design of structures.

Prior to using the Richter Scale, the Mercalli Intensity Scale was developed, which classified earthquakes based upon their effect at the earth's surface. It was developed by Guiseppe Mercalli in 1902 and described in a USGS pamphlet as shown below – the higher the number the more severe the damage:

- (I) - Not felt except by a very few under especially favorable conditions.
- (II) - Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
- (III) - Felt quite noticeably by persons indoors, especially on the upper floors of buildings. Many do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration similar to the passing of a truck. Duration estimated.
- (IV) - Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
- (V) - Felt by nearly everyone; many awakened. Some dishes and windows broken. Unstable objects overturned. Clocks may stop.
- (VI) - Felt by all; many frightened and run outdoors, walk unsteadily. Windows, dishes, glassware broken... books off shelves... some heavy furniture moved or overturned; a few instances of fallen plaster. Damage slight.
- (VII) - Difficult to stand... furniture broken… damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
- (VIII) - Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture moved.
- (IX) - General panic... damage considerable in specially designed structures, well designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.
- (X) - Some well built wooden structures destroyed; most masonry and framed structures destroyed with foundation. Rails bent.
- (XI) - Few, if any masonry structures remain standing. Bridges destroyed. Rails bent greatly.
- (XII) - Damage total. Lines of sight and level distorted. Objects thrown into the air.

The Richter magnitude scale, used mostly by the media and for general intensity comparisons, is now replaced by site-specific ground accelerations as explained in following sections. An excellent source of information on earthquakes, including hazard maps, is http://www.usgs.gov.

When is Seismic Design Required for Retaining Walls?

It depends upon what guides you. The evidence of earthquake damage to properly designed retaining walls is nearly non-existent, excluding waterfront walls where liquefaction occurred, and walls poorly designed for static loads. Based on the senior author's observations and reviews of inspection reports from both the Northridge and Loma Prieta earthquakes, incidents of damage were not noted for walls properly designed for static loads. Building code changes are usually prompted by failures observed, such as that of wall-to-roof diaphragm connections on tilt-up buildings during the San Fernando earthquake of 1971 which prompted corrective code changes. However, this does not seem to be the sequence for retaining walls because of the lack of failure evidence. It can, however, be argued that we have not yet experienced "the big one", and more may be learned from the Japan magnitude 8.9 earthquake of 3/10/2011. Whether the evidence supports it or not, (and the authors are not aware of results from seismic simulated tests on retaining walls) we are guided by IBC '12 and ASCI 7-10 (or as modified or adopted by jurisdictional codes):

Current Code Requirements in IBC '12

Here is IBC '12, 1613.1:

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14 and Appendix l1A. The seismic design category for a structure is permitted to be determined in accordance with Chapter 1613 or ASCE 7.

This clearly requires all "structures" to be designed for seismic loads. The question is whether a retaining wall which is unoccupied and not a risk to life safety (unless supporting a building), is considered such a "structure"? Or is it exempt as permitted by Exception 3: *Agricultural storage structures intended only for incidental human occupancy*?

However, the ASCE 7 cited by IBC '12, 1613.1, further states in the 2010 edition:

ASCE 7-10 chapter 15.6.1, *Earth-Retaining Structures*: *"This chapter applies to all earth-retaining structures assigned to Seismic Design Category D, E, or F* (Note: these preclude Seismic Design Categories A, B, and C which are exempt from seismic design because S_{DS} is less than 0.50 -- see following section for definition of S_{DS}) *The lateral pressure due to earthquake ground motion shall be determined in accordance with Chapter 11.8.3".* This latter Chapter states that if a geotechnical investigation report is required (often at the discretion of the building official) the report shall include "*The determination of dynamic seismic lateral earth pressures on basement and retaining walls due to design earthquake ground motions".* 15.6.1 continues: *"The risk category shall be determined by the proximity of the earth-retaining structure to other buildings and structures. If failure of the earth-retaining structure would affect the adjacent building or structure, the risk category shall not be less than that of the adjacent building or structure. Earth-retaining walls are permitted to be designed for seismic loads as either yielding or nonyielding walls. Cantilevered reinforced concrete or masonry retaining walls shall be assumed to be designed as simple flexural wall elements."*

If a geotechnical investigation is *required per* IBC '12, 1803 such a report shall comply with IBC 1803.5.12. Note: that seismic design is not required for wall supporting less than six feet.

1803.5.12 Seismic Design Categories D through F. For structures assigned to Seismic Design Category *D. E or F the geotechnical investigation required by Secai 1803.5.11 shall also include all of the following as applicable:*

1. The determination of dynamic seismic lateral pressures on foundation walls and retaining supporting more than 6 feet (1.83 m) of backfill height due to design earthquake ground motions

2. The potential for liquefaction and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the maximum considered earthquake ground motions. Peak ground acceleration shall be determined based on:

2.1 A site-specific study in accordance with Section 21.5 of ASCE 7;or

2.2 In accordance with Section 11.8.3 of ASCE 7.

3. An assessment of potential consequences of liquefaction and soil strength loss, including, but not limited to:

3.1. Estimation of total and differential settlement:

3.2. Lateral soil movement;

3.3. Lateral soil loads on foundations;

3.4. Reduction in foundation soil-bearing capacity and lateral soil reaction;

3.5. Soil downdrag and reduction in axial and lateral soil reaction for pile foundations;

3.6. Increases in soil lateral pressures on retaining walls; and

3.7. Flotation of buried structures.

4. Discussion of mitigation measures such as. but not limited to:

4.1. Selection of appropriate foundation type and depths;

4.2. Selection of appropriate structural systems to accommodate anticipated displacements and forces;

4.3. Ground stabilization; or

4.4. Any combination of these measures and how they shall be considered in the design of the structure.

Bottom line: Check with the local building authority for code applicability and interpretation, and with the geotechnical engineer for their recommendations applicable to retaining walls.

Seismic Design Background

Determining with some rationale how seismic forces act on retaining walls is complex and impeded by diverse opinions, differing theoretical assumptions, and in-situ tests that don't match theoretical approaches. Researchers acknowledge the complexity of this task as code-writers try to mandate minimum design guidelines for public safety.

This effort is difficult for two reasons. As stated earlier, unlike buildings where we can learn from failures, reports of damage to reasonably well designed retaining walls (that were not designed, considering seismic forces) are nearly non-existent (waterfront walls and liquefaction conditions excluded) therefore there is little to observe and analyze to suggest design remedies. And as opinioned above, many question the need for adding seismic forces to static-designed retaining walls, considering both performance history and factors of safety incorporated into the design of walls. Secondly, and compounding the dilemma, as stated above many of the theoretical approaches to determine seismic forces on retaining walls each relies upon differing assumptions that yield differing results, and to in-situ and laboratory tests that didn't perform as theory predicted.

In past years "pseudo-static" (that is, using a static force to simulate a dynamic force) analyses were conducted for which the inertial effects of ground shaking were represented by a lateral *force*, which then made the problem solvable using statics. That force was usually set equal to 0.15W, where 0.15 was assumed to be the effective horizontal ground acceleration and W the "rigid body mass" portion of the backfill. The line of action of the force was assumed to act through the center of gravity the rigid soil mass. Factors greater than 0.15 might have been used based upon a consideration of the "importance" of the wall (now codified as the *Importance Factor).*

The practice of assuming a static equivalent horizontal ground acceleration factor is now largely replaced with a "site acceleration" based upon site specific spectral analyses. The concept of spectral analysis, whereby the design acceleration is based upon the characteristics and period of a structure, was introduced in the 30's and codified in the 40's. Accelerations for retaining walls, which are generally considered "short period" structures (less than 0.2 seconds), use a design acceleration given for 2% damping with a 10% chance of being exceeded in 50 years. This site-specific Peak Ground Acceleration (PGA) derived from a Maximum Considered Earthquake (MCE), is given by the geotechnical consultant or can be obtained by maps in IBC '12 or ASCE 7-10 (they are the same), or directly from www.usgs.gov.

The pseudo-static approach is useful when analyzing a wall for stability – overturning, soil bearing, and sliding – but does not give the distribution of seismic lateral force incrementally on the stem. Resolving this deficiency is discussed in following sections.

Mononobe-Okabe Analysis

The well known and frequently cited Mononobe-Okabe equation (M-O) is an adaptation of the Coulomb equation to account for seismic forces. The Coulomb equation (discussed in Chapter 5, *Forces and Loads on Retaining Walls)* can be used to determine the lateral force on a retaining wall from earth pressure but does not include the inertial force that the soil backfill impacts on a retaining wall during an earthquake. One of several solutions to this problem, taking into account both horizontal and vertical ground accelerations, are the M-O equations that provide seismic coefficients for active and passive pressure (K_{AE} and K_{PE} respectively). The development of this widely accepted work is based on an original work by Japanese professors Mononobe and Okabe in 1926-29.

Many investigations of dynamic forces on retaining walls are reported in the technical literature. One of the most important and influential was an ASCE paper titled *Design of Earth Retaining Structures for Dynamic Loads*, by Seed and Whitman, the results from which were presented at a 1970 Cornell University conference. In that paper they also cite the pioneering studies by Mononobe and Okabe. Another contribution was a later ASCE paper by Robert Whitman titled, *Seismic Design and Behavior of Gravity Retaining Walls* in 1990. That paper considered the lateral force to be derived from an inverted triangular wedge of soil behind the wall. Seed-Whitman proposed a simplified equation, based upon the Mononobe and Okabe theory, for the combined static and seismic factor, which they termed K_{AE}, to be applied to this wedge acting against the wall. This adaptation of the Coulomb equation to calculate the total (seismic and static) pressure, introduced the variable θ, which is defined as the angle whose tangent is the ground acceleration ($\theta = \tan^{-1}\left[\frac{k_h}{1-k_v}\right]$), resulted in the M-O equation we now use (See Appendix I for definitions of k_h and k_v). We note that if the acceleration variable θ is excluded from the M-O equation it reverts to the familiar Coulomb equation.

7. DESIGNING THE CANTILEVER WALL STEM

Basics of Stem Design

Here are two very rough rules-of-thumb for assuming stem thickness: If a reinforced concrete stem, try one inch of thickness for each foot of retained height, but not less than eight inches. If a masonry stem, 8" is usually adequate for walls about six feet high, and 12" for walls up to 12 feet. Higher walls, those with sloping backfills, or when surcharge load are present will require thicker stems.

The controlling design condition for reinforcement occurs at the bottom of the stem (top of footing), where the maximum stem moment occurs. Reinforcing steel must be selected to resist that moment, however, it is not economical to use the same steel design higher up the wall where the moment is less (unless the wall is very low). Usually, after the base of the stem is designed, another design is performed several feet higher, usually at the top of the dowels projecting from the footing. At that point alternate bars can be dropped, or sizes reduced, for economy. The diagram in Figure 7.1 illustrates this concept. If the wall is very high, you may want three or four cut-off levels and perhaps a change in stem thickness, but carefully observe the influence of a battered wall on stem thickness or changes in (concrete to masonry blocks), material. See Figure 7.1.

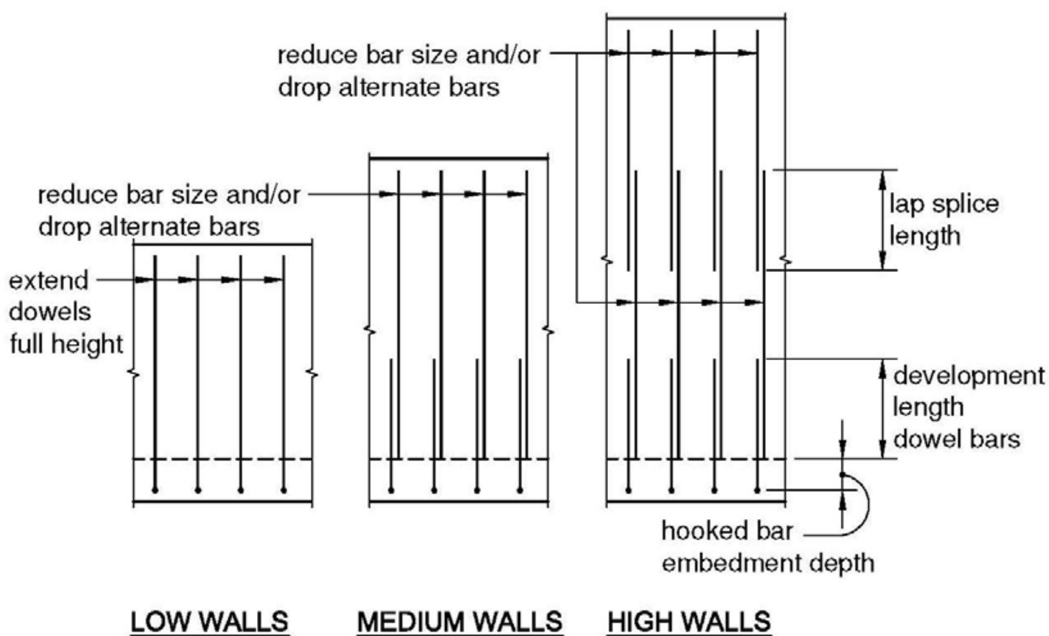

Figure 7.1. Reinforcing placement in stem

A useful rule to remember is that for a triangular lateral active pressure behind the wall, the moment at the base of the stem diminishes to one-half of that at 0.20H above the base. For example, for a 10 foot retained height the moment is one half its maximum at two feet above the base. In nearly all cases the moment at the top of the dowels is about one-half that at the base of the stem thereby halving the design requirement for continuing lapped reinforcing.

Often the stem projects above the retained height to provide a fence barrier, or a wood fence may be added to the top of the stem. In such cases, the wind load on that portion above the earth should be considered in the design, as it contributes to overturning. If the stem is essentially a yard wall and not a retaining wall and with very little earth retention, then remember that the wind can blow from either direction which will require that the wall and footing to be checked for both conditions.

Dowels from Footing into the Stem

The reinforcing at the bottom of the stem usually consists of footing bars bent up into the stem as dowel bars. Unless the wall is relatively low, say four or five feet, it is not economical to extend the dowel bars to the top of the wall, because the moment in the stem diminishes rapidly with height. Vertical dowels must only extend up to where they are not required, at which point either alternate bars can be dropped, or spliced (lapped) with lesser size bars. However, dowels must extend up into the stem a distance equal to the development length of the bar, or the required lap distance for the continuing bars, whichever is greater, provided however, that each bar extends at least 12 bar diameters beyond the point bars of that size and spacing are not needed for moment. The lap length required for the continuing bars nearly always governs. The required development length and lap lengths for both masonry and concrete are shown in the table below. Hooked bar embedments into the footing are also shown. Note the footnote assumptions at the bottom of the table.

Bar Size		Masonry(2) f'_m =1500 psi		Concrete (3)		
		Grade 60	Grade 40	2000 psi	3000 psi	4000 psi
#4	L	24	20	20.9	17.1	14.8
	H(4)			9.4	7.7	6.7
#5	L	30	25	26.2	21.4	18.5
	H(4)			11.8	9.6	8.3
#6	L	36	30	31.4	25.6	22.2
	H(4)			14.1	11.5	10.0
#7	L	42	35	45.8	37.4	32.4
	H(4)			16.5	13.4	11.6
#8	L	48	40	52.3	42.7	37.0
	H(4)			18.8	15.4	13.3

(1) Minimum lap for spliced bars, inches, assumes f_y = 60 ksi

(2) 40 bar diameters for f_y = 40 ksi and 48 bar diameters for f_y = 60 ksi (48 bar diameters shown)

(3) Minimum lap is development length x 1.3, assuming Class B splice. Cannot be reduced for stress level

(4) Assumes standard hook and not reduced by ratio A_s (required) / A_s (provided)

Note: IBC '12, 2107.3, deletes for ASD the following development length equation in MSJC '11, 3.3.3.3.

(5) "L" = lap length; "H" = hook bar embedment in inches.

Figure 7.2 Lap splice lengths and hooked bar embedments (inches)

Development length in masonry is given in MSJC 2011, 3.3.3.3:

$$\ell_d = \frac{0.13\, d_b^2\, f_y\, \gamma}{K\sqrt{f'_m}}$$

γ = 1.0 for #3,4,5 bars, 1.3 for #6, 7, and 1.5 for #8
K = Masonry cover but not less than 5 d_b
d_b = Bar diameter

This equation results in much longer lap lengths than 48 bar diameters and has met with considerable objection. IBC 2012 modified this requirement (only for Allowable Stress Design, ASD) to: $\ell_d = 0.002\, d_b f_s$ but not less than 12". This requires 48 bar diameters for Grade 60.

Horizontal Temperature / Shrinkage Reinforcing

Horizontal reinforcing is necessary to control cracks because of temperature changes and concrete shrinkage. Figure 7.3 shows minimum requirements for both concrete and masonry (CMU). There may be conditions (climate, aesthetics, and better crack control) where additional reinforcement would be required at designer's option.

Typical Horizontal Rebar Spacing .0007 A_g Masonry and .002 A_g for concrete						
Mat'l	**Thickness**	**#3**	**#4**	**#5**	**#6**	**#7**
Concrete	6	9	17	18	18	—
Concrete	7	8	14	18	18	—
Concrete	8	7	12	18	18	—
Concrete	9	6	11	17	18	—
Concrete	10	5.5	10	15	18	—
Concrete	12	9	17	18	18	—
Concrete	14	8	14	18	18	—
Concrete	16	7	12	18	18	—
CMU	6	24	48	48	48	—
CMU	8	16	32	48	48	—
CMU	10	16	24	32	48	—
CMU	12	12	24	32	48	—
CMU	16	8	16	24	40	48

Figure 7.3 Horizontal temperature/shrinkage reinforcement concrete and masonry walls (inches)

The ACI requirement for reinforcing in both faces of concrete walls over 10 inches thick is waived for retaining walls in contact with earth per interpretation of ACI – 14.3.4.

8. SOIL BEARING AND STABILITY – CANTILEVERED WALLS

Overturning and Resisting Moments

The easiest way to check stability, sliding, and soil pressure, is to set up a table showing each force and load element, together with the its moment arm measured from the lower front (toe) edge of the footing. An example of such a table is shown on Design Example #1 in Chapter 24. The tabular format provides an orderly summary of forces, moment arms and moments for easy checking of computations.

Proportioning Pointers

Here are a few pointers and guidelines to proportion the footing:

- The width of the footing for most conditions will be approximately 2/3 of the retained height.
- It is usually most advantageous to have more of the footing width on the heel side of the stem. This will put more soil weight on the heel to improve sliding and overturning resistance.
- If there is a property line on the heel side, try to get as much heel width as possible as to provide the additional soil weight. Otherwise, you will have a sliding problem.
- If you need a key for sliding resistance, try to keep its depth less than about one-fourth the retained height, but recommend not over about two feet.
- If there is a property line on the toe side, the heel of the footing may need to be wider because soil pressures are usually greater at the toe.

Overturning Moments

Overturning moments, as discussed in Chapter 5, are horizontally applied forces multiplied by the moment arm from the bottom of the footing to the line of action of the force. The primary force causing overturning is the lateral earth pressure against the wall. Derived from a triangular pressure diagram, its point of application is one-third the height above the bottom of the footing. The height used to compute over-turning is on the virtual plane at the back of the footing (i.e., where this plane intersects the ground surface). Lateral pressure from a surcharge is a uniform load applied to the back of the wall, therefore its point of application is one-half the height and the moment arm is from that point down to the bottom of the footing. See Figure 5.5 which illustrates both conditions. The overturning moment from the lateral earth pressure is acting against the virtual plane at the back of footing as illustrated in Figure 8.1.

Wind pressure on the stem projecting above the soil or on a fence sitting atop a wall can also cause overturning. Wind pressures are computed in accordance with the applicable building code, and generally range from 12 to 30 psf.

Seismic, if applicable, will also contribute to overturning. That was discussed in Chapter 6.

If there is significant depth of soil or ponded water above the toe of the footing, this lateral force could be viewed by some as being deductible from the heel-side active force for computing overturning and sliding. Our recommendation is to disregard this concept because it may not remain in place during the design life of the wall. Only consider the depth of soil on toe side below the top of the footing when computing passive resistance.

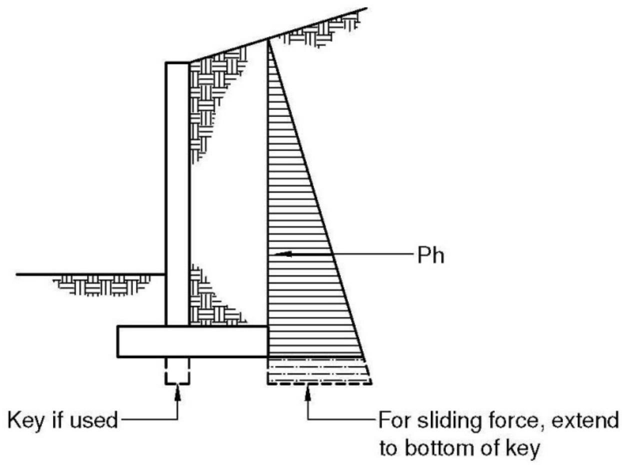

Figure 8.1 Overturning moments for a cantilevered retaining wall

Resisting Moments

By convention, resisting forces are all vertical loads applied to the footing. These forces include the stem weight, footing weight, the weight of the soil over the toe and heel, and a surcharge if applicable and any axial load applied to the top of the wall. The total resisting moment is the summation of these loads multiplied by the moment arm of each measured from the front bottom edge of the footing. See Figure 8.2.

The generally accepted factor of safety against overturning is 1.5, although some agencies require more. When seismic is included, a factor of 1.1 is permitted by IBC '12.

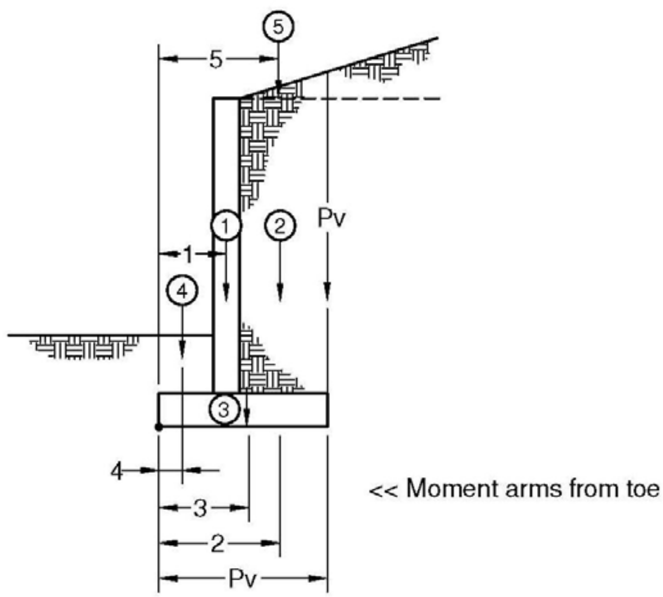

Figure 8.2 Resisting moments

To determine overturning and resisting moments, eccentricities and soil pressures, you should tabulate these values as illustrated on Design Example #1, Chapter 24.

Vertical Component of Active Pressure From a Sloped Backfill

If the backfill is sloped, there is a vertical component of the lateral pressure, which is assumed to act on a vertical plane at the back of the footing. This vertical component can act to resist overturning because when the wall starts to rotate there will be a frictional resistance along that plane. See Figure 8.3.

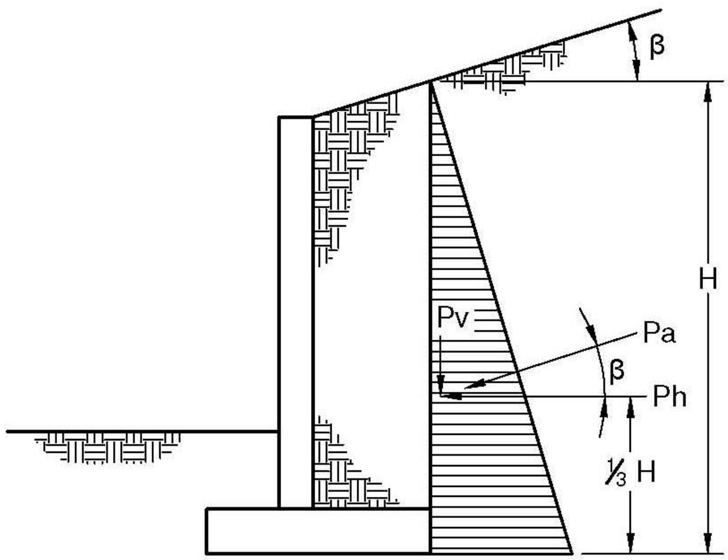

Figure 8.3 Vertical Component of Active Pressure

There is, however, controversy over whether to use this vertical component for soil pressure calculations because its use can significantly reduce soil bearing pressure and may not be justifiable if there is a large heel dimension. Similarly, it may not be justified to add vertical force to increase friction for sliding resistance. Most texts recommend using the vertical component only to resist overturning – not to reduce sliding or soil bearing. However, this judgment is left to the engineer.

Determining Soil Bearing Pressure

The allowable soil bearing value, q_{all}, is within the purview of the geotechnical engineer, and usually varies from 1000 psf for poorer soil (or without a substantiating soil investigation), to 4000 psf for dense soil.

After you have assumed a footing width, taking into account property lines or other conditions that may restrict the heel or toe distances, you can determine the applied soil pressure by determining the eccentricity of the total vertical force load with respect to the centerline of the footing. This is done as follows: first determine how far from the edge of the toe the resultant vertical force acts. This is simply the total overturning moment minus the resisting moment, divided by the total vertical force.

$$x = \frac{M_{resisting} - M_{overturning}}{W}$$

W = Total vertical force (weight of concrete, soil over the heel and toe, plus loads on the soil backfill)

Basics of Footing Design

The method of reinforced concrete design known as the Strength Design (SD) Method should be used to design retaining wall footings. Strength Design requires the soil pressure to be factored to compute shears and moments. See the Design Examples for procedures. Footing design based upon Strength Design requires factoring the upward soil pressure attributable to lateral earth pressure by 1.6, and pressure attributable to the weight of earth or other dead loads be factored by 1.2. Because these two components apply to footing factoring it may be reasonable to simplify by factoring the ASD soil pressure by their average: [(1.6 + 1.2) /2] = 1.4).

Embedment of Stem Reinforcing Steel into Footing

For an adequate moment connection from the stem into the footing it is customary to extend the stem reinforcing into the footing a depth sufficient to form a 90° bar hooked toward the toe (or heel if the distance is insufficient). In practice, the footing bars are placed first and extend as dowels up into the stem to lap with continuing stem reinforcing. See Figure 9.1.

This stem/dowel reinforcement must be hooked into the footing and can be bent 90° and extended to reinforce the toe. The required embedment length is specified by the following equation (see ACI 318-11, 12.5):

$$\ell_{dh} = \frac{0.02\, d_b f_y}{\sqrt{f_c}} (0.7) \left(\frac{A_s \text{required}}{A_s \text{provided}} \right)$$

where d_b = bar diameter

ℓ_{dh} = required hooked bar embedment, $8d_b$ or 6" but not less than 6 inches

Embedment depth can be reduced by the stress level in the reinforcing depending upon the application of ACI 318-11,12.5.3 (d) which states that excess reinforcement can be credited except where "*...anchorage or development is not specifically required...*"

Required dimensions and radii of hooked bars are shown on Figure 9.1. Embedment requirements plus the three inches of protective concrete cover determine the minimum total depth of the footing.

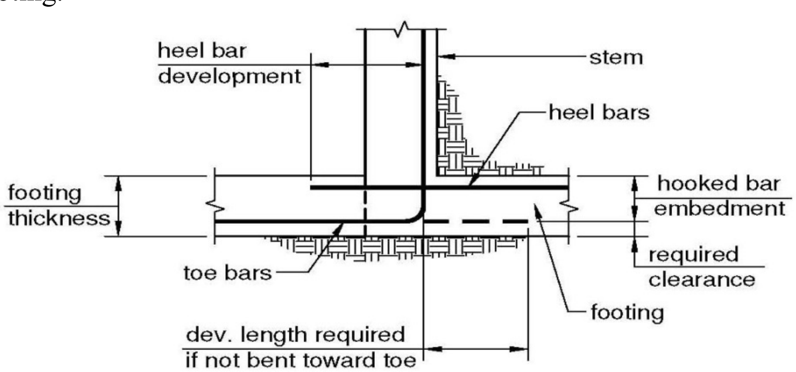

Figure 9.1 Hooked Bar Bend Requirements

10. PIER AND PILE FOUNDATIONS

Piles, Piers, and Caissons

Returning wall support options perform essentially the same function: to penetrate soil to a depth sufficient to achieve greater load bearing capacity than would be provided by a spread footing. This is achieved either by end bearing or frictional resistance along the lateral area of the shaft, or both.

PILES accomplish this by being driven (steel, concrete, or timber) to either bear on hard strata or develop sufficient skin-friction through the depth of penetration. Concrete piles are usually the choice for retaining walls and abutments, and are either driven precast concrete, or cast-in-place in drilled bores.

CAISSONS is a term often used interchangeably with piers. Caissons are usually large diameter piers, but can have narrow shafts with a flared (bell) bottom for greater bearing area. Neither type is often used for retaining walls.

PIERS is a term used to describe a relatively short cast-in-place concrete shaft foundation. Some codes define a pier (as opposed to a pile or caisson) as having a depth-to-diameter ratio less than twelve. A pier's supporting capacity is achieved by a combination of lateral surface friction and end bearing but some codes do not allow both combined. If a masonry retaining wall has spaced pilasters, the pilasters can be cantilevered up from an embedded pier (see Pilaster Masonry Wall, Chapter 18).

When to Use Piles or Piers?

The recommendation to use piles or piers to support a retaining wall will usually come from the geotechnical engineer. Conditions which would suggest using piles include poor or compressible underlying soil, the need for greater lateral resistance, space limitations when a conventional footing may be too large, or other site-specific concerns. Single-row drilled cast-in-place piers, aligned under a retaining wall, are probably more commonly used. Single rows of piers are relatively easy to install, penetrate to better soil, and resist both the vertical and lateral loads imposed from the wall above. With higher walls a double row of staggered piers is common practice. The staggering provides from greater overturning resistance and use of smaller diameter piers. Small implies diameters less than 24", as opposed to large diameter piers that might be needed for overturning moment or high retaining walls.

Design Criteria

Design criteria for piers and piles is usually provided by the geotechnical engineer because IBC '12, 1803.5.5 requires a foundation investigation for deep *foundations "unless sufficient data upon which to base the design and installation is available"*. This investigation generally includes: recommended type of piles or piers suitable for the site; allowable capacity curves for the various alternates, including lateral design criteria; minimum spacing; driving and installation requirements; testing requirements and related recommendation that include site-specific precautions.

To aid the geotechnical engineer, the designer should provide the total vertical load imposed by the retaining wall (weight of stem, footing, soil, surcharges, and any additional axial loads) and the total base shear (lateral force imposed by the retaining wall). Using the recommendations of the foundation investigation report the designer can then select the proper size and penetration of the pier or pile, and provide the appropriate specifications, referencing the foundation investigation report. It is important that the owner retain the geotechnical engineer to observe all aspects of the installation for conformance with the recommendations of the geotechnical report.

Pile Design

The structural design requirements for piles are covered in IBC ‘12 Chapters 1808 through 1812.

Lateral stability is an essential consideration for any retaining wall. To resist a lateral force piles may be either battered or the lateral force can be resisted by bending in vertically aligned piles. In the latter case, passive and active pressures can be used to determine pile/pier depth.

Consider possible site clearance problems and consult the installing subcontractor for suitability of your design when using battered piles. Generally, a batter flatter than 1(H): 4 (V) should be avoided. Combining lateral pile bending with battered pile resistance is not recommended.

Where multiple piles are used the code requires interconnected lateral restraint at the top of the piles. However for retaining walls this is achieved by the footing which also serves as the pile cap.

Pile Design Example

This example assumes the same vertical load and horizontal force as Design Example #1.

Use two rows of piles, space 4 ft. apart laterally, centered under footing, and, say, 8 ft. on center longitudinally.

Reduce footing width to 7 ft. and increase thickness to 24", therefore footing weight about the same.

V_{base} = 4,360 # $P_{vert.}$ = 9,034 #. e (eccentricity from C.L. ftg.) = 1.54 ft.

Convert to 8 ft tributary length: V_{base} = 34,380 # P_{vert} = 72,272 #

Vert. load per pile = $P = \frac{\sum V}{n} \pm \frac{\sum Md}{\sum d^2}$

n = number of piles (= 2).

d = distance from c.g. of piles to specific pile (= 2).

M_{ecc} = (9034 x 8)(1.54) = 111,299 ft-lbs.

$\therefore\ P = \frac{72{,}272}{2} \pm \frac{111{,}299 \text{ x } 2}{4} = 91{,}785 \text{ ft} - \text{lbs. max.}$

V to each pile = (4,360 x 8)/2) = 17,440 lbs.

11. COUNTERFORT RETAINING WALLS

Description

The decision to use either a buttress or counterfort depends up site restraints, such as property line locations, and aesthetics. A “counterfort” wall should not be confused with a “buttressed” wall. The two are different. A counterfort wall has the stiffening element on the inside of the wall, within the retained earth. See Figure 11.1. A buttress wall has the counterforts on the outside exposed side of the wall. Although most counterfort walls are cast-in-place concrete, masonry can also be used. The design procedures are essentially the same.

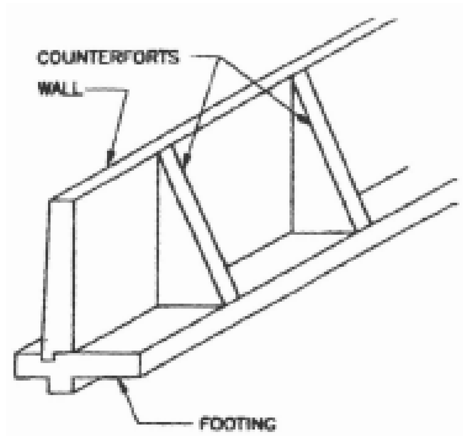

Figure 11.1 Typical counterfort wall

Proportioning

The spacing between counterforts for economical design is usually one-half to two-thirds the wall height. The width of the footing will usually be about two-thirds the wall height, or larger for surcharges or sloped backfill.

Design Overview

The design of a counterfort wall can be somewhat complex because the number of components which must be designed differently than for a conventional cantilevered wall. The steps in the design of a reinforced concrete counterfort wall are as follows (each step will be discussed later):

1. After establishing the retained height, select a spacing for the counterforts, usually one-half to two-thirds of the retained height. Determine the footing width required and soil bearing at both the toe and heel because you will need these dimensions to establish the counterfort dimensions, and for stability calculations design as if the wall is a continuous cantilevered wall. You can add an estimated weight of the counterforts prorated as a uniform longitudinal axial load.
2. Design the wall as described in the following section as a two-way slab, fixed at the base and at the counterfort crossings and free at the top.
3. Design the footing toe as a cantilever from the wall.
4. Design the heel as a longitudinal beam spanning between counterforts.
5. Design the counterfort. It will be a tapered trapezoidal shaped tension member.
6. Check the final design for stability, overturning, sliding, and soil pressures.

12. CANTILEVERED TILT-UP WALLS

Description

Tilt-up concrete construction is a growing segment of the concrete industry and now accounts for over 50% of all low-rise commercial buildings and about 90% of industrial and warehouse buildings. Tilt-up yard walls, trash area enclosures, dock walls, and retaining walls are now commonplace and the use of this technique can be advantageous for retaining walls in general. This method is particularly advantageous for long walls allowing repetitive use of panels.

The primary advantage of the use of tilt-up concrete is speed of construction and the elimination of expensive formwork necessary for cast-in-place walls. However, because a crane is necessary during erection, and because a casting bed is required, provision must be made for stacking panels on the site. Connections must also be made for joints between panels.

Construction Sequence

After preparing a 3” to 4” thick concrete casting slab (later discarded), edge forms are set, a bond breaker is sprayed on the bed to prevent bonding of the wet concrete to the casting bed, reinforcing is placed, and the concrete for the wall is placed. To save casting area, panels can be stacked on top of each other, separated by a bond breaker, up to five or six slabs high as desired.

Unique to using tilt-up panels for free-standing or retaining walls, a trench for the foundation is first excavated and the panels set on temporary concrete setting blocks and the panel is temporarily braced. Dowels project from the bottom of the panels into the footing excavation to provide a moment connection after the concrete is placed.

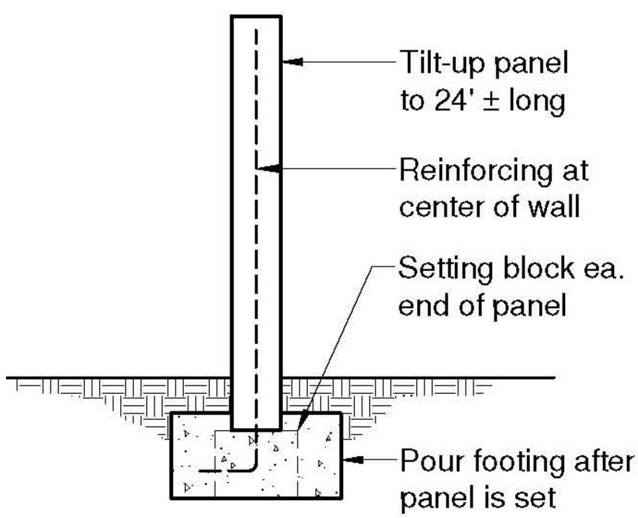

Figure 12-1 Tilt-up freestanding panel

13. WOOD RETAINING WALLS

Overview

These retaining walls are usually constructed by the homeowner or landscape contractor and rarely exceed three or four feet soil retention. These consist of wood posts embedded into the soil a sufficient depth to restrain the lateral soil pressure imposed by wood lagging spanning between the posts. The design is often based upon do-it-yourself books for wood retaining walls. Wood retaining walls are advantageous for economical construction of low walls (about five feet maximum earth retention). Such walls do require excavation into the uphill side and the low side of the wall can be used for planting. An illustration of a wood retaining wall is shown in Figure 13.1.

Figure 13.1 Typical wood retaining wall

Calculating Lateral Pressures

To design the horizontal lagging, then the cantilevered support posts, the lateral soil pressure can be determined using the Rankine equation as described in Chapter 5. For example, the lateral pressure at a depth of three feet, with a soil density of 110 pcf, phi angle of 34°, and a level backfill, would be 94 psf acting horizontally at the lower most lagging. The lagging at that depth would be designed for that lateral force along the entire span between posts. The code prescribed minimum lateral earth pressure for a level backfill is 30 psf/ft, but increases if a sloped backfill and for different soil types. See IBC '12, Table 1610.1.

Lagging Design

Lagging usually consists of planks with a nominal thickness of 2", 3", 4", or 6". Thicker planks are generally not economical but may be necessary for higher walls. Allowable stresses are based upon the species selection and given in National Design Standards (NDS), 2005 Edition. All stresses should be based upon long-term loading and wet conditions of use. Spacing between posts is usually determined by how far a 3" plank for a given depth will span – or 6" if necessary. For the above example, a 3" x 12" plank (dressed dimensions 2-1/2" x 11-1/4") has a section modulus (weak axis) of 11.7. in^3. Assuming an allowable stress of 900 psi this plank could safely

14. GRAVITY WALLS

Overview

Gravity walls depend upon bulk weight for stability, as opposed to a cantilevered retaining wall fixed to a foundation. Some of the many types of gravity retaining walls were described in Chapter 1. Most gravity retaining walls are relatively low, such as used in landscaping, and do not require engineering per se – the design is intuitive to the astute builder. Most landscaping walls do not have a footing, rather are founded on a gravel base.

Note that retaining walls not over four feet from bottom of footing to retained height, and if without a surcharge, do not require a building permit per IBC ‘12, 105.2(4).

Gabion walls, crib walls, and large-block gravity walls are discussed in Chapter 15.

The design of the more common types of gravity walls composed of rubble, stones, and mass concrete is discussed in this chapter.

Design Procedure

The design of a gravity retaining wall of concrete or bonded (mortar/grout) stone involves seven basic steps:

1. Calculate the dead weight of the wall, including all components and any superimposed surcharge or axial load, plus tributary earth weight over the base.

2. Based upon (1) compute the resisting moment about the front edge of the base.

3. Determine the lateral soil pressure and its line of action. The Coulomb Equation (see Chapter 5) should be used because it includes backfill slope, batter of the wall, and the soil friction angle at the wall interface. You may consider the use of the vertical component of the active pressure, which is assumed to act vertically at the back edge of the wall footing. The line of action for the resultant lateral force is assumed to be the wall friction angle plus the inclination angle of the wall batter. Alternatively use the Rankine equation with the force diagram in Figure 14.1.

4. Check stability by computing overturning moment, resisting moment (per above), and determine factor of safety (1.5 minimum).

5. Check that soil bearing is within code allowable.

6. Check sliding. Coefficient of friction is generally 0.25 to 0.40. If soil is clay, cohesion would control.

7. Verify that little or no flexural tension exists in the wall. Check at several locations by calculating the section modulus of the wall and lateral moment at each selected height.

15. GABION AND MULTI-WYTHE LARGE- BLOCK WALLS

Descriptions

Gabion walls consist of steel wire baskets filled with rock and stacked as units to form gravity retaining walls. Similar baskets have been used since ancient times and the word "gabion" does not refer to an inventor but rather to Italian and Latin words meaning "cage". Today, the cages are manufactured, generally, in three foot by three foot by three foot steel wire panel sides which at the job site are unfolded to form a cage, which are filled with rock, tied together, and assembled into the retaining walls. Since mesh openings are generally 3 inches square, the rock infill should be 3 inch to 8 inch clean hard stone. Perpendicular to the plane of the wall the wythes can be 1, 2, 3 or more units deep and can be stacked in successive courses to a height usually not more than about 15 feet.

Note: The masonry term "wythe" means one vertical section of wall one unit in thickness.

Similar in concept, precast large concrete blocks, which are commercially available from a number of vendors and concrete plants, can be laid one or more blocks deep (wythes) and stacked to retain soil to 12 feet or more. Such blocks can be laid with the front exposed side flush or with successive blocks stepped back.

If the front face is flush, it is customarily tilted into the soil about 6° for aesthetics.

Design Methodology

The cages are wired together and due to their mass they are considered one rigid cohesive mass for design purposes. Gabion walls are designed or analyzed in the same manner as gravity walls. Resisting moments are taken about the front lower corner of the first row and overturning moments are applied to the back face using the Coulomb method for calculating K_a. Density of the gabion units is usually taken as 120 pcf. Refer to Figure 15.1 for conceptual example of a flush-face wall.

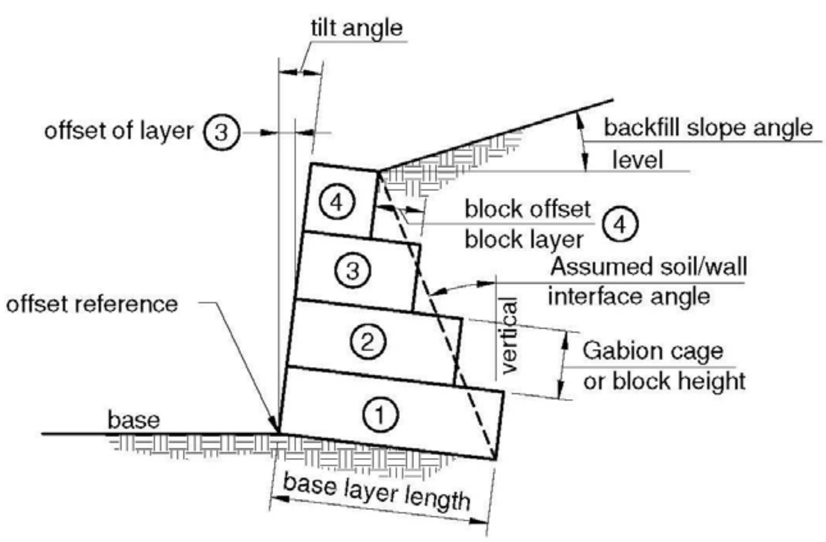

Figure 15.1 Example of gabion wall analysis section

Lateral pressures are computed by the Coulomb equation shown in Figure 15.2.

16. SEGMENTAL RETAINING WALLS (SRWs)

Overview

Segmental block retaining walls (SRWs) are composed of dry-stacked masonry blocks usually manufactured as proprietary products. They have gained wide acceptance for high earth retention condition and are seen everywhere: leaning against hillsides alongside highways, behind shopping centers, providing tiered grade changes for developments, highways, railroads, bridge abutments and other applications. See Figure 16.1.

Figure 16.1 Example segmental block wall

Advantages include relatively fast construction; at the footing consists of just a gravel levelling pad, and the units are dry-stacked without mortar, steel reinforcing, or grouting. The designer has a choice of block sizes, textures, colors and configurations, from a variety of vendors. Retained heights of 40 feet or more can be achieved (using geogrids) far exceeding economical limits of conventional masonry or concrete retaining walls. These do, however, have limitations. If a segmental retaining wall requires geogrids for stability, this requires an available space behind the wall of approximately 70% or greater of the wall height for the placement of the geogrid reinforcement layers. If space is unavailable, a segmental wall is not an option. Buried utility lines or drain lines in the backfill zone may also be constraints for a segmented wall. However, using the segmental blocks as a fascade is also feasible for soil not walls or soldier beam and wood-logging walls.

Segmental walls are of two types: pure gravity walls where stability depends solely upon the resisting moment of the stacked blocks to exceed the overturning moment of the lateral soil pressure. This stability problem limits the height to four or five feet, although some vendors offer larger blocks enabling greater retained heights.

Higher walls, the more common type of segmental walls use layers of geogrids placed in the backfill for soil reinforcement as the wall is constructed. This results in a mass of reinforced soil (also termed Mechanically Stabilized Earth, MSE) which can be used en masse to improve resistance to overturning and sliding. To be effective, each layer must be properly connected to the block facing by engaging the geogrid within block joints, and extending behind the wall and beyond the failure plane a distance sufficient for anchorage. The vertical separation between geogrid layers is usually two- to three blocks, but varies with design requirements. The length of the reinforced zone is usually a minimum of 60% to 70% of the wall height.

For many engineers, designing segmental retaining walls is a niche market. Their design can be quite complex, particularly for higher walls using geogrids, and other improvements that are to be accommodated in the geogrid reinforced zone, such as caissons, drain liners, shallow foundations for light weight structures. Consultation with a selected block vendor is recommended and many offer design software.

Segmental Blocks

Segmental Blocks are concrete blocks with compressive strength of 3,000 psi or greater, and, in the US, they are manufactured per proprietary designs at licensed local plants. The blocks come in many choices of texture, color, sizes, and configurations. The blocks vary in size, with the most commonly used blocks being 8-inch high with depths varying from 10" to 24". The block width for the most commonly used blocks is 18 inches. Blocks with dimensions smaller than these are available for non-engineered landscape applications for retaining heights of about three feet or less. All of these blocks weigh between 30 and 110 lbs each. So called "big blocks" are also available from some vendors, weighing two tons or more and placed by small cranes.

The blocks are designed to allow construction of walls with vertical batter -- angle of the wall face to the vertical -- to as much as over 15 degrees from vertical. To control batter most segmental blocks have offset lips or other means, such as pins between units, to control the offsets as successive courses of blocks are placed.

Angle of wall batter = $\tan^{-1}$ [(offset per block) / (block height)]

Most blocks have interior voids which are infilled with granular backfill material. Weight per square foot of wall surface is often assumed to be 130 pcf for both block weight and infill.

All vendors have web sites for more information and technical data. Best source: a Google search for "segmental retaining walls".

Segmental Gravity Wall Design

For segmental gravity walls to be stable, the resisting moment should exceed the overturning moment by a factor of safety of at least 1.5. This limits the height of gravity segmental walls to about four feet, depending upon the batter of the wall and block type. For larger blocks that are in the market, the gravity wall height can be greater.

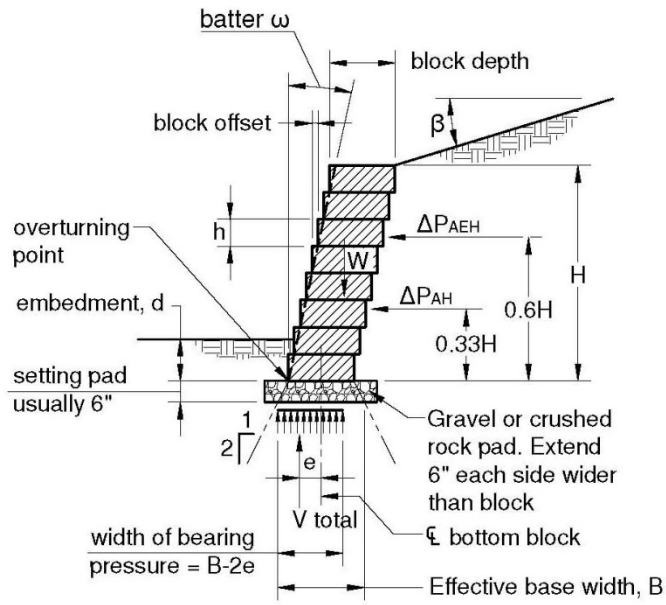

Figure 16.2 Forces on gravity SRW walls

The design procedure for gravity walls follows these steps:

Select the block vendor for texture, color, size and configuration desired. This is often dependent upon proximity to distributors.

1. Determine the retained height required and embedment depth below grade. Embedment depth is usually one block course or one foot. Total wall height is equal to the full retaining height, plus the embedment.

2. Determine surcharges, if applicable, from backfill slope or roadway traffic. If seismic design is required see below for seismic design.

3. Check "hinge height", which is the height to which blocks can be stacked, with offsets, before tipping over. The equation for this is:

   Hinge height = (block depth) / [(tan (batter angle)]

   Don't stack higher than this or the wall will collapse!

4. Determine soil properties: Soil unit weight and internal friction angle (phi) for both internal (backfill soil) and external (in-situ, or natural) soil. Backfill should be granular soils, with phi angle greater than 30 º. Ideally it would be USCS Group GW (well-graded gravels, gravel-sand mixtures, little or no fines, per Unified Soil Classification System – see Appendix B). However, for economical reasons, on-soils such as, clayey sand, silty sand, and sand are also used, provided their phi angle is considered in the design and compacted property under montering and testing by Project Geotechnical Engineer.

17. SWIMMING POOL WALL DESIGN

Swimming pools are constructed in a wide variety of shapes, sizes, curvatures and designed to fit a specific terrain and soil conditions. One thing nearly all have in common is shotcrete or Gunite walls and bottoms sprayed over a shaped excavation, and encasing the reinforcing. Plaster or tile is used to provide a smooth, aesthetic finish.

The terms “shotcrete” and “Gunite” are used interchangeably, but the former refers to wet-mix spraying whereas the material is mixed in a hopper before exiting the nozzle, whereas the latter is a “dry-mix” where the material reaches the nozzle dry where water is injected. Shot Crete (we’ll use the generic term) sticks to the earth and self compacts because of the velocity of application, thereby permitting it to be used against vertical surfaces. Shot Crete is covered in IBC ‘12, section 1910.

Designing the walls of a pool is unique because not only does the wall usually curve as it descends, but the strength of the cantilevered wall must resist the greater of earth pressure acting inward with the pool empty, or the water pressure outward if the exterior grade is lower or of poor soil. The design task is made further tedious because of the number of cross sections which must be checked (shallow end, deep end, and intermediate points).

The typical controlling condition is when the pool is empty and earth pressure from the outside governs the design. However, the condition is often reversed, such as for “infinity pools” or architectural features where the outside grade is substantially lower, or slopes downward lessening its lateral support value. There also may be lateral support from of a surrounding deck at or near the top of the wall. All these conditions must be considered and the walls of the pool designed for the most critical combination of conditions that may occur. Lateral loading from a surcharge or increased soil pressures because of expansive soil must also be considered.

Design of swimming pools is a specialty for some engineers and they have developed software (usually spreadsheets) to make the task less tedious.

The walls and bottom are generally at least 4” thick, generally 5” for floors, and may be more depending upon design requirements. Typically, #3 bars are used because of the relative ease in bending and securing to curved surfaces. Number 4 bars can also be used, but #5 bars are difficult to bend and place. Shot Crete strength is typically 2500 psi minimum, and a low slump suitable for pumping and spraying. Minimum reinforcing for flexural members is 200 / f_y, = 0.0033 for Grade 60 reinforcing. Thus, for a 4" wall the minimum would be #3 at 9", however, the typical practice pattern is 12" on center each way. Under slab drainage is recommended on sites with expansive soil with special reinforcement and/or thickened slab required for sites with expansive soil to protect from uplift along the bottom of the shallow end.

The classic method of designing swimming pool walls has been to draw to scale (or CAD generated) a cross section at each location to be investigated. Then divide the wall into segments, usually 12" high. You can then determine the bending moment and shear at the bottom of each segment by constructing a table (spreadsheet) showing the active pressure from either earth or water acting at the bottom of each segment, and the additive (or deductive) moment due to the vertical weight of the segments above acting at their eccentricity from a reference point. This is illustrated in Figure 17.1. This is a tedious process but yields satisfactory results for design. Reinforcing is usually placed in the center of the wall, but for higher moments thicker walls may be needed and off-center reinforcing as the design may require.

18. PILASTER MASONRY WALLS

Description

Shown in Figure 18.1 is a retaining wall with spaced pilasters and masonry filler walls. Such walls can be economical for low retaining or freestanding walls. The filler walls, usually 6" or 8" masonry, span horizontally between pilasters and the pilasters cantilever up from the footing.

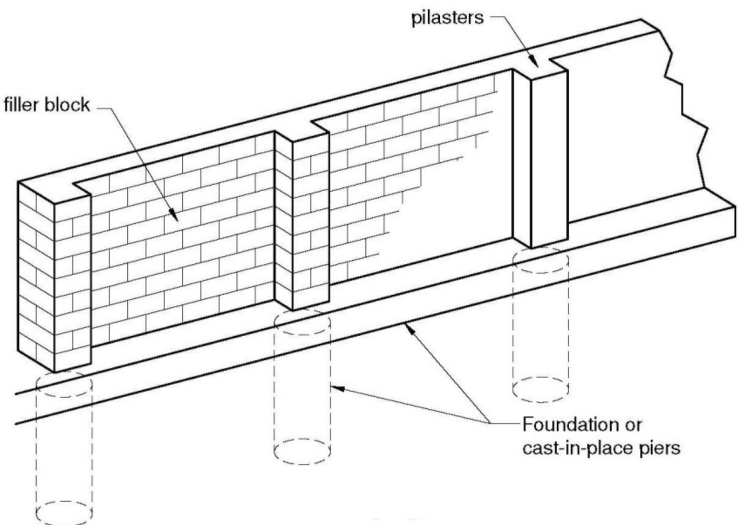

Figure 18.1 Pilaster masonry wall

Filler Wall Design

The filler wall spans horizontally between pilasters and those walls usually control the spacing of the pilasters. Freestanding walls are designed for wind and, if applicable, a seismic force. That horizontal reinforcing is placed in the center of the wall because lateral wind and seismic loads can be from either direction. To take advantage of continuity, it may be more economical to place the horizontal reinforcing at the center and design for the controlling positive (mid-span) or negative (at pilasters) moments, generally use $w\,(L)^2 / 12$.

If the filler wall retains earth, some or all of the courses will be subjected to lateral earth pressures and this controls the thickness of the filler wall. In that case, vertical reinforcing should be placed on the earth face between pilaster supports. Reduce reinforcing higher up the wall as moment decreases. The first step would be to determine the lateral pressure at the base of the wall, as select a wall thickness and vertical reinforcing, and then the reinforcing to span between pilasters.

A minimum amount of horizontal and vertical reinforcing should be used. The combined total area should be .0002bd, with not less than .0007 in either direction. Vertical reinforcing is often #4 bars at 32" o.c. or 48" o.c.

Pilaster Design

Pilasters are usually 16" by 16" masonry units, or smaller for lower walls and usually spaced 6’ to 8’ apart. Use conventional procedures for the design. Lateral load reaction to the pilasters from

19. RESTRAINED (NON-YIELDING) WALLS

Description

Retaining walls are broadly defined as either yielding or non-yielding. The former refers to cantilevered walls, which are free to rotate, thereby allowing a lateral displacement at the top which activates the soil wedge concept upon which both Rankine and Coulomb theories are based.

Non-yielding walls are restrained at the top to prevent movement and therefore generate a reaction at the top and reduce moments at the base of the wall. A typical restrained, non-yielding, wall is the so called “basement wall”. The designer must assess whether the wall really is “restrained” at the top against lateral movement. Wood diaphragms may be too flexible.

Lateral restraint at the top can also be accomplished using tie-backs, also called anchored walls, which are another example of restrained non-yielding walls. These walls use drilled and grouted anchors placed into the backfill as the wall is constructed to provide lateral restraint. If multiple levels of lateral restraint are required, such as for a multi-level structure, the design becomes complex due to varying wall moments, shears and reactions. Tie-back forces can also be affected by earth movement.

Dual Function Walls

Often it is desirable to prepare two designs for the same wall. For example a basement wall may be backfilled before the lateral restraint at the top (such a floor or roof diaphragm) is in place. It can first be designed as a conventional cantilever wall as for an assumed depth of backfill, and perhaps lessening the factors of safety because of a temporary condition. This would require a larger footing for overturning and result in a larger moment at the stem base. Then a second design for the final condition when the top restraint is in place and backfill completed. Then you’ve covered both conditions, but only if the contractor placed the backfill to meet your design/soil placement assumption.

If the bottom of a basement wall is fixed at the footing, and assuming a triangular earth pressure against the wall, the base moment will be about one-half the pin-pin positive moment, and the positive moment if fixed at the bottom will reduce to about one-quarter the pin-pin positive moment condition.

“At Rest” Active Soil Pressure

If a wall is restrained from movement at the top and therefore the sliding-wedge active pressure cannot be mobilized, the lateral soil pressure is somewhat higher. This is termed the “at rest” pressure, (designated K_o) and is applicable to a wall rigidly restrained at the top, such as a basement wall (but light framing with a flexible diaphragm may be inadequate “restraint” and the active soil wedge may be activated). The at-rest soil pressure is: $K_o = 1 - \sin\phi$, where ϕ is the angle of internal friction. For example, if $\phi = 34°$, $K_a = 0.44$, as opposed to $K_a = 0.28$ (assuming level backfill). For sloping backfill, a suggested equation is

$$K_o = (1 - \sin\phi)/(1 + \sin\beta).$$

For a well-drained granular soil, a typical value for $K_o = 0.50$. For a saturated sandy soil the density could be 125 pcf giving a lateral pressure of 0.5 (125 – 62.5) + 62.4 = 93.7 pcf. Clayey soil can be higher. Some agencies require $K_o = 1.0$, giving 110 pcf for a soil density of 110 pcf. ASCE 7-10 specifies a minimum of 60 pcf for “relatively rigid” walls, and states that basement

21. SOLDIER BEAM WALLS

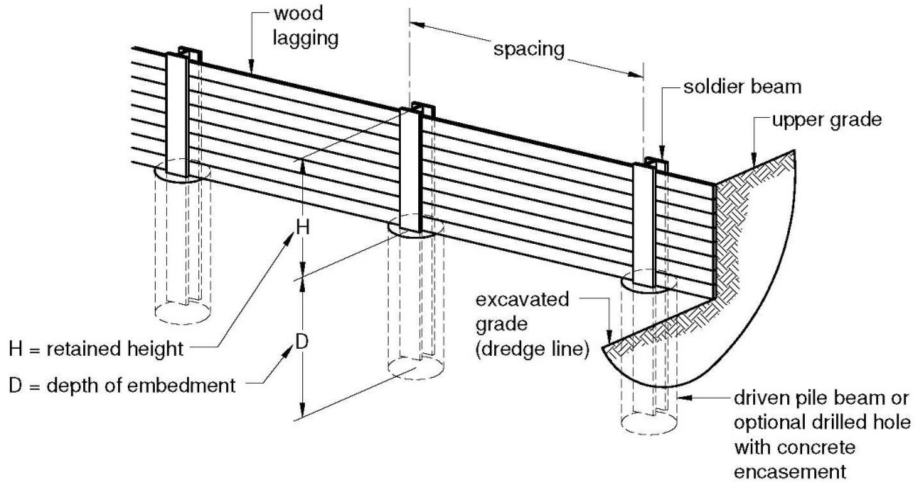

Figure 21.1 Typical soldier beam construction

Description

Soldier beam retaining walls are used to temporarily retain soil, such as at a construction site. They can also serve as permanent retaining walls as shown in Figure 21.3. This concept is illustrated in Figure 21.1. Steel HP (wide flange) beams are driven into the soil a sufficient embedment depth to resist by passive pressure the moment imposed by the retained soil. The soldier beams (also called soldier piles) are usually spaced from six to eight feet apart and can also be dropped into pre-drilled holes and encased in lean concrete. Soldier beams are usually cantilevered, but if space is available, and for retained heights over about 15 feet, tiebacks can be used to reduce the beam size and depth of embedment.

As excavation proceeds on the down-grade side, wood lagging is placed horizontally to support the retained soil. Lagging is supported at their ends by the beam outer flanges.

Design Procedures

Consult with the geotechnical engineer for design criteria. This information will include nature of the soil, phi angle, soil density, active and passive allowable pressures, arching factors to use, and any other site-specific recommendations. It is advisable to also consult with the contractor to verify the most economical beam selection and any other concerns he or she may have.

There are numerous design methodologies used and most foundation engineering textbooks propose various design approaches. This text selected a relatively simple procedure which is often used.

This procedure assumes non-cohesive (sandy) soil. If the soil is clay a different passive resistance diagram will apply and the geotechnical engineer should be consulted. It should be noted that although clay is usually assumed to have a zero phi angle, it actually can vary in a range from 6° to 12° or more.

22. WHY RETAINING WALLS FAIL AND COST EFFECTIVE FIXES

The above photo is a rare occurrence. No building permit, not engineered, minimal reinforcing in ungrouted cells, and other oversights.

“Failure” of a retaining wall does not necessarily mean total collapse as shown above, but rather local signs of impending instability and likelihood of a total collapse. Total collapses are relatively rare. In a total collapse the wall overturns, slides, topples, or otherwise causes a massive letting loose of the retained earth with resulting damage above and below the wall. Such walls cannot be saved – the remedy is rebuilding. The engineer who provided this photo was retained to investigate the deficiencies causing the collapse and to design a new wall.

Fortunately, retaining walls are quite forgiving, nearly always display telltale signs of trouble alerting an observer to call for professional evaluation before a collapse. After an evaluation, and determination of the causes, most walls can be saved.

The most common sign of distress is excessive deflection of the wall – tilting out of plumb – caused by a structural overstress and/or a foundation problem. Some structural deflection is to be expected and a rule-of-thumb is $1/16^{th}$ inch for each foot of height, which is equivalent to one-half inch out-of-plumb for an eight foot high wall. More than that is suspect. It’s easy to check with a plumb bob.

Here are Twelve Things That Can Go Wrong and Signal Distress:

1. Reinforcing not in the right position. If the stem shows sign of trouble (excessive deflection and/or cracking) the size, depth, and spacing of the reinforcing should be verified. Testing laboratories have the devices (usually a magnetic field measuring Pachometer) which can locate reinforcing and depth with reasonable accuracy, up to about 4 inches depth. For exact verification you can first locate the reinforcing then chip out to determine its exact depth and bar size. More elaborate devices are also available if needed – check with your testing laboratory, they’ll come to you jobsite. Unbelievably, cases have occurred where the reinforcing was placed on the wrong side of the wall, either through a detailing error, or contractor error. When the actual reinforcing size, location, and spacing are determined, and perhaps a core taken to verify strength of stem material, a design can be worked backwards to determine actual design capacity and thereby guide remedial measures.

23. CONSTRUCTION TOPICS AND CAVEATS

Vertical Control Joints

Vertical joints along the length of the wall are intended to control cracking and are largely a matter of judgment. Shrinkage in a wall cannot be eliminated. As the adage goes, concrete shrinks and ice cream melts, or “if it ain’t cracked, it ain’t concrete.” We can attempt to control where the cracks form by forming crack control joints and by increasing the horizontal reinforcing. With a little more than minimum reinforcing there are few reports of problems when control joints are 100 feet or more for masonry, and somewhat less for concrete. The more horizontal reinforcing, the less likely vertical cracks will be obvious, and the further apart joints may be spaced. In the case of a concrete wall, a ratio of $0.002A_{gross}$ is suggested; for masonry $0.0013A_{gross}$ is suggested (#5 bars at 32" o.c. for an 8" CMU wall).

Vertical joints for both concrete and masonry should be “cold joints”, allowing for movement, but it is suggested that some horizontal dowels extend into the adjacent wall to assure out-of-plane alignment. Usually one end of horizontal dowels are wrapped, sleeved, or greased to prevent bonding.

Drainage

Improper drainage causing water seepage into the backfill is the leading cause of retaining wall problems. Lateral earth pressure design is usually based upon drained soil. Saturated soil can substantially increase pressures. Therefore it is important to have weep holes at the base of the wall for any percolating water to escape. In concrete walls drain holes are 3" to 4" in diameter to facilitate cleaning and spaced five or six feet on center. Gravel should be placed along the inside base for any water to freely flow, otherwise the only thing coming out of a weep hole will be grass.

“Weep holes” in masonry walls can be provided by leaving the head joints open at alternate blocks (no mortar in end joints at 32" on center).

In lieu of weep holes, or for basement type walls, horizontally placed perforated Sch 40 pipe should be laid along the base of the heel adjacent to the stem, slopped to an outlet, and encased in a generous amount of coarse gravel. It is also recommended to lay a filter fabric over the gravel to keep out soil fines.

The most important drainage control is to keep water off the top slope as much as possible. This can be done by slope control, paved swales, paving, or other means. <u>Preventing water from entering the backfill is critical important because it changes the soil characteristics and increases lateral pressures.</u>

Backfill

Backfill material should be sandy non-cohesive material. Clayey soil are to be avoided because clay swells when wet, causing additional lateral pressure. An excellent practice is to fill the soil wedge with gravel.

Compaction

Compact the gravel behind the wall with care. You don't want settlement to occur later. Place the gravel in layers about one foot thick and start compacting at the face of the wall and work away from the wall. Gravel is best compacted with multiple passes of a vibrating plate compactor.

Inspections

If a consultant was employed, he or she will verify that the footings are excavated into the anticipated soil and indicate any corrections deemed necessary. They can also approve the backfill material.

Placement of reinforcing dowels projecting from the footing into the wall are critical to the design, and the Engineer-of-Record (EOR), or a deputy inspector, should verify that the dowels were properly placed. Several retaining wall failures were attributable to the dowels being on the wrong face of the wall!

Other inspections may be required by the building official, or by the EOR.

The Investigation

The geotechnical report for a project will nearly always have recommendations for site preparation (e.g. if fill is present or there is a liquefaction problem) in addition to design criteria information. This investigation report is usually a part of the contract documents and should be carefully reviewed and observed.

Forensic Investigations

If a problem is evident, or suspected, an independent engineer may be retained to investigate the problem. This will involve a review of the design, particularly to determine if the site conditions match the design criteria (e.g. a wall designed to retain eight feet, and actually retaining ten feet). The plans will be reviewed for clarity and conformance with the design intent and applicable building codes. The wall will be measured, deflection checked, and testing done to determine positioning of reinforcing and material strengths. Cores are often taken to determine both concrete strength and grout penetration into cells. The geotechnical report will be reviewed and perhaps more soil samples recommended.

When the cause of the problem is discovered, the most economical solution acceptable to the owner should be determined. This can be contentious, particularly if opposing parties offer different solutions. Hopefully the issues can be resolved equitably and with civility without resort to litigation. If an impasse, mediation can be a very effective and less costly resolution of a dispute.

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