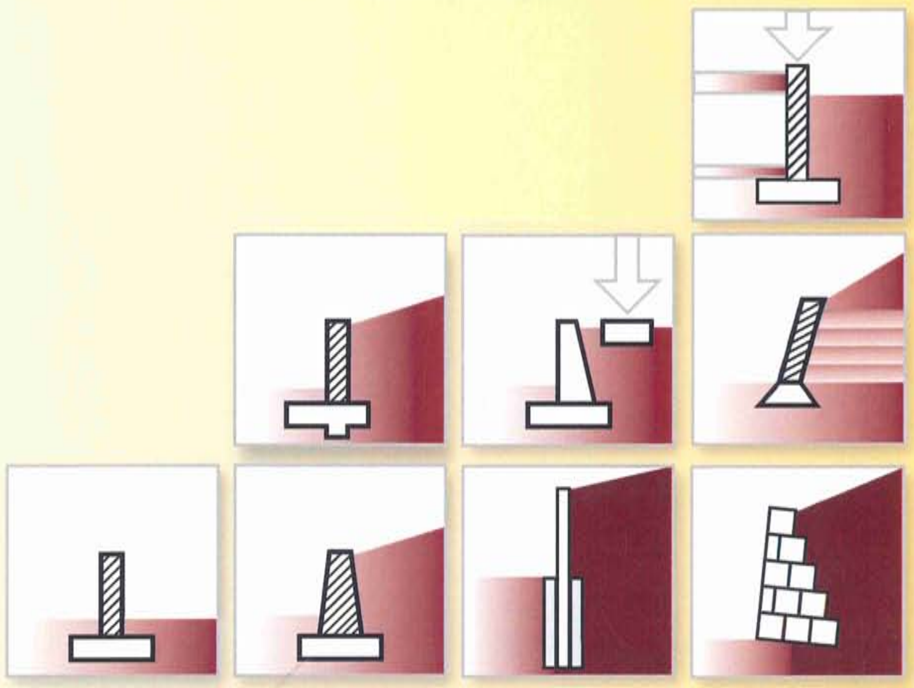
Basics of Retaining Wall Design

8th Edition



A guide for the practicing engineer

Hugh Brooks
Civil & Structural Engineer

Basics of
RETAINING WALL
Design

Eighth Edition
Revised

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A Guide for the Practicing Engineer

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Disclaimer

Although it is intended that the material herein is accurate and represents good design practices, it is possible–and even likely–that errors may occur. These are my views, my code interpretations, and my design practices. Other engineers may have differing views. Therefore, each of you, as engineers-of-record, must assess this information and assume responsibility for your designs. Neither Hugh Brooks nor HBA Publications, Inc. can assume any liability for damages resulting from your use of the information in this book.

"Of all things, but proverbially so in mechanics, the supreme excellence is simplicity"

James Watt (1736 – 1819)
Inventor of the steam engine

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PREFACE TO THE EIGHTH EDITION

Successive editions of this manual become necessary because building codes change – seismic design requirements for example – and I get suggestions for topics to be added or expanded I also learn from conversations with users of Retain Pro who offer suggestions, point out errors and often enlighten me.

This Eighth Edition is updated and expanded throughout. New topics were added, some corrections made, all design examples updated and new ones added. New topics include soldier pile shoring and multi-wythe large block (and gabion) gravity walls, and there is a design example for each.

This Eighth Edition is a continuation of a series that began with a relatively modest manual, first published in 1996. These manuals are intended as a companion to Retain Pro software to refresh and update the practicing engineer on the basic principles and procedures used to design a variety of retaining walls.

This book is not an in-depth treatment of the design of retaining structures. Retaining structures are far too complex a subject to treat in a single small volume. There are dozens of references and foundation engineering texts and countless technical papers available for review, however, finding what you need is time consuming; hence this compendium. My challenge was to decide what to put in and what to leave out of a manual. My goal was to put in the most helpful things a designer needs to know to design most types of retaining walls. Surely there will be omissions and probably some errors, but my hope is that you will find this book helpful in your practice.

A reference bibliography is included in Appendix H for those wishing more detailed information. And, of course, there is always the Internet.

I express my 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. We've had some interesting discussions, from which hopefully we all benefit.

I hope this new edition will be helpful in your practice, and as always your comments and suggestions will be most welcome.

Hugh Brooks, P.E., S.E.

1. WHY THIS BOOK?

The User

This book is primarily for the practicing engineer who has become a bit rusty on the complex subject of retaining wall design. It is a review of basic principles and building code requirements. It is also for the student, assuming he or she has already acquired a basic knowledge of statics, soil mechanics, and the design of simple masonry and concrete structures. It will also be helpful to plan checkers (barring arguable code interpretations).

Why It Was Written

During my many years of providing technical support for Retain Pro, it became increasingly apparent that many engineers infrequently design retaining walls and need some brushing-up, particularly code requirements; the design of retaining walls is not an every-day design task. Over half of the technical support questions I receive are about basic concepts and code requirements, rather than about use of the program. I also discovered that there apparently does not exist a single reference book specifically addressing retaining wall design. Although there is very considerable amount of information available, it is widely scattered in numerous textbooks and technical papers on soil mechanics, foundation engineering, concrete design, masonry design, and all sorts of related topics. However a single volume on retaining wall design for the professional practitioner could not be found. Hence, I attempted to condense, simplify, and compile information from many sources, including my own experience, into this book. Hopefully, it will ease your comfort level to design retaining walls and give you a good overview of the process. Those that desire to dig deeper for particular topics there is a comprehensive bibliography in Appendix H of this manual.

Scope of This Book

This book treats most types of retaining walls: conventional cantilevered, restrained (basement), gravity, and segmental retaining walls, both gravity and with geogrids. Other topics include sheet pile walls, tilt-up retaining walls, soldier piles, gabion walls, counterfort walls, pilaster walls and pile/pier foundations.

Feedback

Your comments, corrections, and suggestions will be welcome. You can email me at hbrooks@retainpro.com.

2. 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 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 thousands of years, but they used what they had, and learned how to do it better with each succeeding structure. The Great Wall of China, for example, transverse bamboo poles were used to tie the walls together – a forerunner of today's "mechanically stabilized earth". These 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 their ingenuity and accomplishments.

Major advances in understanding how retaining walls work and how soil generates forces appeared in the 18^{th} and 19^{th} centuries with the work of French engineer Charles Coulomb 1776, and who is better remembered for his work on electricity, and later by William Rankine in 1857. Today, their equations are familiar to every civil engineer. 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, and a better understanding of soil behavior, and hopefully safer and more economical designs.

A Definition:

A retaining wall is any constructed wall that holds back soil, a liquid or other materials, where there is an abrupt change in elevation.

The Precision Illusion

Let's not fool ourselves. Even though the science of soil mechanics is well developed and reasonably well understood, it is still not an exact science and remains both an art as well as a science. Soil is a mixture of earth materials and although its characteristics can be closely defined its actual in situ behavior will not precisely fit theory. For example: the straight line we assume for the angle of rupture is actually somewhat concave; and, the "equivalent fluid pressure" of soil is not truly triangular. We make simplifying assumptions to make our designs manageable. As the adage goes: *engineering is an exact science based upon assumptions*. Our calculations are the best we can do with assumptions we make and the results are never fully accurate. That's why we use factors of safety. So keep precision in mind when calculating beyond the first decimal point.

Types of Retaining Structures

There are many types of retaining structures for soil and other materials, but listed below are the types of retaining walls used today. Most of these will be discussed in later chapters.

Cantilevered retaining walls

These are the most common type of retaining walls. Cantilevered walls are classified as "yielding" because they are free to rotate without any lateral restraint. Cantilevered retaining walls are generally 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

Masonry stems are usually either 8" or 12" concrete block masonry units, partially or solid grouted, and reinforced. Higher walls require 12" blocks and are often stepped back to 8" thick as the retained height diminishes.

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) and with infilled 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 incorporates wing walls projecting from the heel into the stem. The stem between counterforts is thinner and spans horizontally 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 where moments are higher. The high cost of forming the counterforts and infill stem walls make such walls usually not practical for walls less than about 16 feet high. See **Figure 2-1.**

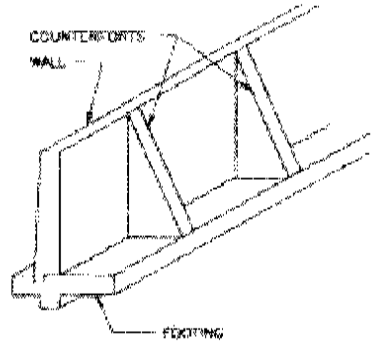


Figure 2-1 *Counterfort Retaining Wall*

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 interior face provide limited space for the heel of a traditional cantilvered retaining wall.

Gravity retaining walls

This type of wall depends upon 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 relegated to landscaping features and retaining less than about four feet high. Engineering for these 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 global stability, overturning, sliding, and to verify that little or no flexural tension exists within the wall because these 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 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 have also been used.

Wood retaining walls

These are commonly used for low height retaining walls. Wood retaining walls usually consist of laterally spaced wood posts embedded into the soil, or set in 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 walls have 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. The footing is then placed under the wall to complete the construction. Tilt-up walls are economical for higher walls, but need panel casting space.

Segmental retaining walls (SRWs)

There are many manufacturers offering various systems of stacked segmental concrete units, steel bins, or other devices that retain soil by stacking components. Most are patented systems that are typically battered (sloped backward) to reduce lateral soil pressure, thus requiring a minimal foundation. Footings, reinforcing, or mortar are not used. Stability of SRW gravity walls depends solely upon the resisting moment exceeding the soil pressure overturning moment. To attain greater heights – up to 40 feet and more – SRW's utilize mechanically stabilized earth (MSE), also called *reinforced earth*, whereby geosynthetic fabric layers are placed in successive layers of the backfill to achieve an integral soil mass that decreases overturning and horizontal sliding. A variety of facing block configurations and surface 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. If the girder provides lateral support, this must be accounted for in the design. Design requirements for bridge structures are usually governed by AASHTO and state Departments of Transportation (DOTs).

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 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 are usually restrained at the top by either a slab 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 the top, thereby with less or no dependence upon fixity at the foundation. Technically, they 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. This is the reverse of a cantilevered wall.

Footings for these walls are usually designed for vertical loads only --not for overturning-- however, it is often desirable to design a basement wall as a retaining wall too, because backfill can then be safely placed without having to brace the wall, or waiting until the lateral restraint at the top, such as a floor, is in place. Note that conventional wood floors framed into the top of a basement wall do not provide a sufficient stiffness to allow for the restrained case. 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.

Anchored (tieback) walls

This method is used for higher walls. Restraint is achieved by drilling 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 drilled holes. The latter are also known as *soil nails*.

Cantilevered Retaining Wall Terminology

Cantilevered retaining walls have unique descriptive terminology as illustrated below:

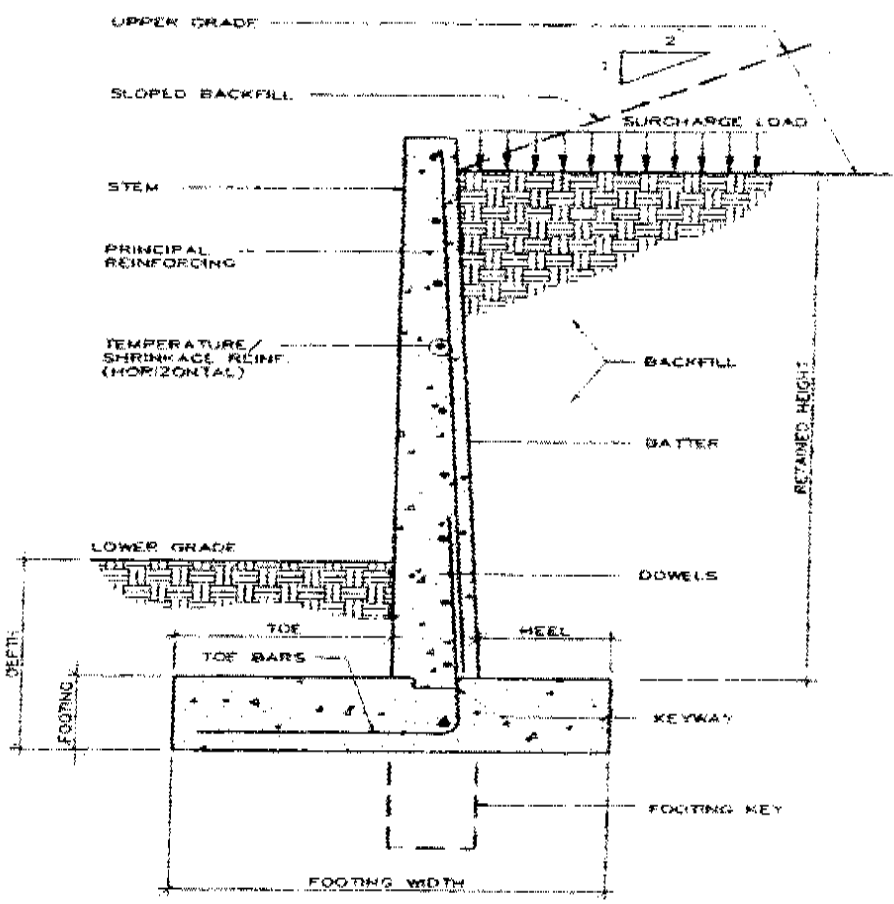


Figure 2-2. ***Retaining Wall Terminology***

What The Terms Mean:

Backfill: The soil placed behind the 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.

Grade: The surface of the soil or paving; can refer to either side of the wall.

Toe: That portion of footing which extends in front of the front face of the stem (away from the retained earth).

Heel: That portion of the footing extending behind the wall (under the soil).

Footing key: A deepened section of the footing for greater sliding resistance.

Keyway: A horizontal slot located at the base of the stem for greater shear resistance.

Principal reinforcing: Reinforcing used to resist bending in the stem.

Horizontal temperature/shrinkage reinforcing: Longitudinal horizontal reinforcing usually placed in both faces of the stems and used primarily to control cracking from shrinkage or temperature changes.

Retained height: The height of the earth to be retained, generally measured upward from the top of the footing.

Stem: The vertical wall cantilevering above the foundation.

Surcharge: Any load placed in or on top of the retained soil, either in front or behind the wall.

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".

3. DESIGN PROCEDURE OVERVIEW

The four primary concerns relating to the design of nearly any retaining wall are:

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

And, it is equally important that it is constructed according to the design.

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 that follows).
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).
4. Compute overturning moments, calculated about the front (toe) edge of the footing.
5. Compute resisting moments based upon an assumed footing width, and again calculated about the front edge of the footing.
6. Based upon (4) and (5) calculate the eccentricity of the total vertical load. Is it within or outside the middle-third of the footing width?
7. Calculate the soil pressure at toe and heel. Since contact between the footing at the heel and the soil below cannot resist tension, the eccentricity of a triangular resultant soil pressure will shift outside the middle-third of the footing width. Preferably keep the resultant within the middle third.
8. Design footing for moments and shears. Select reinforcing.
9. Check sliding. A key or adjusting the footing depth may be required.
10. Check and review. Have all report requirements been met?

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 that follows).

2. Compute all applied loads (at-rest earth pressures, seismic, wind, axial, surcharges, impact, or any others. *Select "height" to lateral restraint.*

3. Design the stem. *If the stem is assumed pinned at the base, the maximum moment will be a positive moment near mid-height—design stem material, thickness, and reinforcing for this location. Usually the same material (concrete or masonry) and thickness will be used for the full height. If the stem is fixed at the footing, determine shear and moment at base and design this location. If the stem is fixed at the base, check dowel embedment depth into the footing assuming a 90° bend (hooked bar).*

4. Using statics. *Determine reaction at top support and at base. If a floor slab is present at the top of the footing, check its adequacy for this lateral force (sliding).*

5. Design the footing. *If the stem is assumed fixed at base check the soil pressure and design for 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 the moment because of eccentricity and if adequate the soil pressure will be uniform.*

6. Check sliding. *If a restraining floor slab is not present, a key or adjusting the footing may be required.*

10. Check and review. *Have all report requirements been met?*

Establish the Design Criteria

The following information will be needed before starting your design. The values shown in parenthesis are only given to illustrate those frequently used.

Retained height(s)
Depth of soil in front of wall
* Depth of footing required below grade
* 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 1.5:1 unless OK with geotech)
Wind, if applicable
Axial loads on stem
Surcharge loads
* Seismic criteria if applicable
* Soil density (110 to 120 pcf)
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 report.*

Design Criteria Checklist

After you have established all your criteria, the following checklist indicates additional items to check 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 my footing be?
- 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.
- 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?
- If the wall extends above higher grade, and is a parking area, is there an impact bumper load?
- 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 to restrain sliding?
- 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 investigation 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?
- 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, zero 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 designed.

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

Basic Design Principals for Cantilevered Walls

A cantilevered retaining wall must, for stability, resist both overturning and sliding, and material stresses, including allowable soil bearing, 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.

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, plus the passive pressure in 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. (Note: For cohesive soil, the coefficient of friction is replaced by a reduced value of the unit cohesive bond between the footing and soil in psf.)

The stem must be designed to resist both the bending caused by earth pressures, including the effect of surcharges placed behind the wall, seismic if applicable, wind if applicable, and any axial (vertical) loads imposed on the wall. The maximum bending and shear stresses in a cantilevered wall will, of course, be at the bottom.

Each of these subjects will be discussed later.

Figure 3-1 is a free-body force diagram illustrating forces on the wall sec.

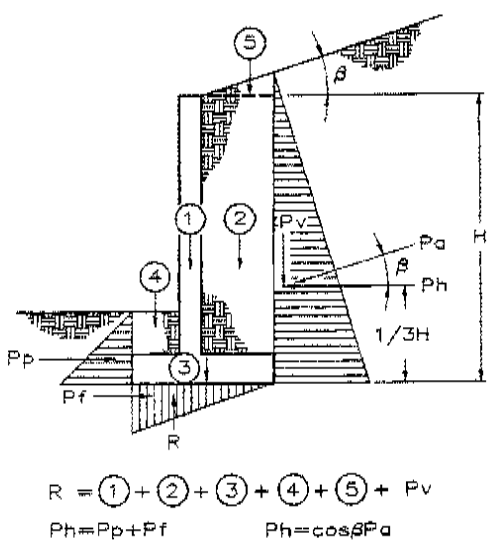


Figure 3-1. ***Free-Body of Cantilevered Retaining Wall***

4. SOIL MECHANICS SIMPLIFIED

A Soil Primer

Most of us remember very little from our Soil Mechanics 101 course taken in college. We rely on the expertise of those of our peers who went on to become engineers. It's a complex subject for which most of us have neither the time nor inclination to master, so we employ consultants.

Here are a few basic concepts about soil:

Most soil is the result of the decomposition of rock, and is classified according to the mix of the grain size of the particles making up the mass.

The most generally used classification by particle size is the *Uniform System for Classification of Soil (USCS)*, and is reproduced in **Appendix A**. The distribution of grain size in a soil sample is determined by a grain size analysis. 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 engineer can classify the soil per the USCS chart. Sieve sizes use a numbering system where the number indicates the number of spaces per inch. For example, a #4 sieve has four spaces 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 (0.074 mm)
Clay	<	0.005 to 0.002 mm

There are other classifications systems, such as the AASHTO system, but the USCS classification system 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 from the cohesive bond between particles, as represented by 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 lateral pressures against a retaining wall.

Expansive soil consists of clay that changes in volume with changes in water content. Such swelling can cause considerable pressure on retaining structures, for this reason clay backfill should be avoided, and if the site contains expansive soil, the 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 – if it is kept dry it won't swell.

The depth of penetration 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 8 feet or more. To prevent the added pressure of swelling because of freezing and thawing, foundations should be placed below the frost line. The 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.

The strength of the soil is usually thought of as its bearing capacity, that is, compression capacity. However, the bearing capacity of the soil is actually a function of shearing stresses between the particles. 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 clayey soil, its cohesion. Sandy soil requires confinements to develop shear strength, as for example a lack of confinement is illustrated when you step on sand at the beach you notice that the sand displaces sideways under your feet.

When soil samples (cores retrieved from drilling) are taken to the laboratory for testing, the engineer will determine the bearing capacity of the particular soil by determining its frictional resistance. He will also test to determine density (weight) of the soil, coefficient of friction, soil modulus, and other properties applicable to the design of the structure.
The coefficient of friction within a soil mass cannot be measured as easily as, say between two solid surfaces. A soil sample is confined in two opposing boxes, separated by a virtual slip plane. While a principal force P is applied perpendicular to the plane, a shear force, F, is applied laterally. The point of slip is noted, and successive tests are recorded for increasing normal stresses. This data is used to determine the coefficient of friction, which is F/P. The corresponding angle, called the *angle of internal friction*, Φ, is therefore the $\tan^{-1}$ (F/P).

This is an oversimplified explanation. Any soil mechanics text will cover this topic thoroughly.

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

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

s = total shear resistance (stress); p = normal stress; c = resistance due to cohesion usually expressed in psf; and Φ = angle of internal friction.

The Soil Wedge Theory for Retaining Walls

How much pressure does the retained earth impose on a retaining wall?

One of the early investigations of this problem was reported in a 1729 publication by French engineer Bernard Belidor. He started with a simple premise: If a wall retaining soil was suddenly removed, the soil behind it would slide down, slipping along a plane he assumed was 45°. He reasoned, and solved by simple statics, that if the plane was without friction, the horizontal force against the wall would be equal to the weight of the "wedge" of soil. He then assumed a 0.5 friction factor along the slope plane, which then halved the lateral force; the lateral pressure was about one-half the weight of the soil wedge.

French engineer Charles Coulomb further developing this theory in the 1770's, (also famous for his work with electricity Coulomb's Law – and other scientific achievements). He solved the problem of differing lateral pressures for varying assumed slip planes, by use of differential calculus to identify the range of rupture planes to determine the minimum horizontal thrust. His solution is the well-known Coulomb formula used today (see page 26). This formula also accounted for varying backfill slopes, batter of the wall, and friction between the soil wedge and the face of the retaining wall.

To keep this soil wedge in place, the three forces shown in the free-body diagram in **Figure 4-1** must be in equilibrium. The three forces are the weight of the wedge, which is its area times the soil density, and which acts vertically downward; the reaction against the wall surface, which is assumed to have a direction inclined at the wall friction angle; and the reaction against the soil behind the wedge. The latter force, or reaction, has two components, one normal to its plane, and one parallel to the plane, which is the coefficient of internal friction times the normal force.

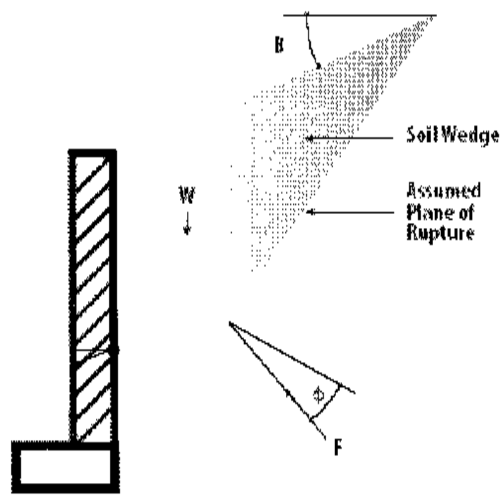


Figure 4-1. ***Free-Body of Soil Wedge***

Later, Scottish engineer William Rankine simplified the Coulomb formula. In the 1850s he presented the equally well known Rankine Formula that neglects wall friction and is more conservative than the Coulomb method (see page 26). His formula takes into account the effect of a backfill slope, but assumes the back face of the stem is vertical, and that there is zero friction at the soil-stem interface, and the resultant acts against the wall parallel to the backfill slope.

Explanation of Design Terms:

Some commonly used terms, particularly as they apply to retaining walls, are defined in the following:

The rupture (or failure) plane

This is the line along which the soil wedge is assumed to slip. It is actually concave, but a straight line is assumed for mathematical simplicity.

The angle this plane makes with the horizontal is theoretically: $\alpha = 45° + \phi/2$. For cohesion less soil this is roughly equivalent to a slope of one horizontal to two vertical (1:2).

Angle of internal friction: This is the most important value for determining lateral pressure and bearing capacity of granular (non-cohesive) soil. It is a measure of the shearing resistance of the soil because of intergranular friction, obtained from one of several laboratory tests, such as the Direct Shear Test. Angles of internal friction range from 32-35° for well graded sand, 27-32° for silty sand, less for sandy silt, and further diminishes for clay because of the lack of coarse particles. The angle of internal friction (usually designated Φ) is used in both the Rankine and Coulomb formulas to determine lateral earth pressure.

Active soil pressure

This is the unit pressure, expressed in pounds per square foot per foot of depth (pcf), imposed upon the wall by the wedge of soil behind the wall. It is mobilized at the moment the wall begins to tilt (or slide) and the wedge begins to slide down along its angle of rupture. It is assumed to obey Pascal's law, that is, to increase linearly with depth, forming a triangular pressure gradient behind the wall. Its value increases with increasing backfill slope, because the volume of the wedge of soil increases (see **Figure 4-2**). This pressure is the coefficient of active pressure (K_a) multiplied by the soil density. K_a for a level backfill is generally close to 0.30. The engineer generally gives this value. Multiplying a soil density of 110 pcf by a K_a value of 0.27 would, for example, result in the oft-used lateral pressure of 30 pcf. Also see ASCE 7-05, Table 5-1, for Design Lateral Soil Loads, which specifies a minimum of 35 pcf for sandy soil and up to 80 pcf for clayey soil.

The active pressure is usually given to you by the engineer as an **equivalent fluid pressure** (EFP), or can be computed from the Rankine or Coulomb formulas if the soil angle of internal friction and, if applicable, the wall friction angle, are given. It is assumed to be a triangular distribution with zero at the ground surface and a maximum pressure at the bottom of the stem (for stem design) or bottom of footing (for overturning design). The pressure diagram will be trapezoidal if a surcharge is applied.

To use the Rankine Formula, you need to know the angle of internal friction, and the slope of the backfill. This will give you the coefficient of active pressure, K_a, which when multiplied by the soil density gives the active pressure in pounds per square foot per foot of depth (pcf).

As discussed later, a surcharge load over the backfill is considered an additional depth of soil, thereby resulting in a trapezoidal lateral pressure.

The line of action of the resultant for the Rankine formula is assumed to act at an inclination of β, the angle of the backfill slope. (Note that for the Coulomb method, which is not generally recommended to be used for cantilevered walls, the resultant acts at an angle, from the horizontal, of the friction angle at the soil-wall interface, δ, usually assumed to be $1/2\Phi$ to $2/3\Phi$, plus an angle equal to the batter angle of the back face of the stem, measured from the vertical.

Figure 4-2 shows backfill volume and surcharge volume and not active pressure.

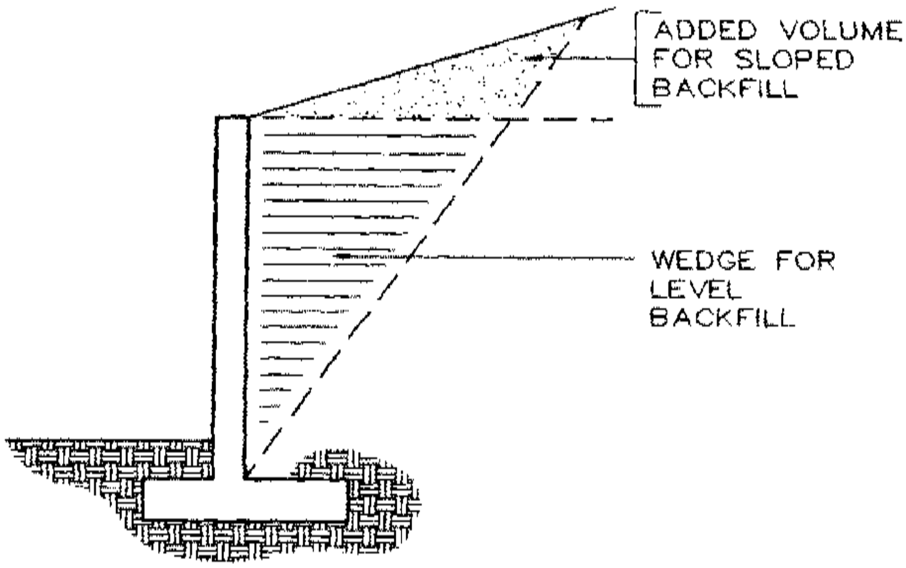


Figure 4-2. ***Increased Wedge Volume with Sloped Backfill***

With a sloped backfill the active pressure on the heel side will increase because of the added height of the soil wedge. The backfill slope angle, β, is in the Rankine formula to reflect this increase. This effect is illustrated in **Figure 4-2.**

Commonly used design values (Rankine) for sloping backfills, assuming a soil wet density of 110 pcf, an angle of internal friction of 34° and "rounding", are:

Level:	31 pcf
5:1 Slope:	33 pcf
4:1 Slope:	34 pcf
3:1 Slope:	36 pcf
2:1 Slope:	45 pcf
1½: 1 Slope:	77 pcf

The slope angle cannot exceed the angle of internal friction.

These values are guides only and determination should be made by a engineer, particularly for slopes steeper than 2:1.

Passive soil pressure: This is the resistance of soil to being pushed against by a rigid surface. It is obtained by multiplying the soil density by the coefficient of passive pressure, K_p. Passive pressures are usually in the range from 250 - 350 pcf. The engineer generally gives this value. The Rankine formula for Kp is the reciprocal of Ka ($K_p = 1 / K_a$). Passive pressure provides resistance to sliding by opposing the active earth pressure, or other applied external forces directed into the retained earth by surcharge loads on the backfill.

Passive pressure applied to retaining walls does not cause overturning; it is the resistance to the net driving lateral forces, expressed by:
(Heel active force) – (toe active force) – (soil/earth friction resistance)
= (passive resistance required)

Passive pressure is discussed further in *Soil Bearing and Stability.*

"At rest" soil pressure: This lateral pressure, designated K_o, applies to non-yielding walls which are laterally supported and restrained from movement at the top and bottom, such as so-called "basement walls". This will be discussed further in the section *Restrained (non-yielding) Walls*. The at-rest condition also occurs when the backfill is highly compacted.

Angle of repose: This is the angle, measured from the horizontal, that a pile of dry, granular, soil will form when loosely poured on a flat surface; for sand, it is about 34°.

Soil density: Weight of soil is usually assumed to be 110 to 120 pcf, depending upon gradation, water content and degree of compaction. Saturated soil has a higher density, because of the added weight of water filling the voids between particles. Soil below the water table is described as being submerged and the weight is the estimated weight minus the unit weight of water.

Backfill slope: The slope of the backfill behind the wall cannot exceed the angle of internal friction for cohesionless soil. A general rule is to limit this slope to 1.5:1 (which corresponds to an angle of internal friction of 34°).

Equivalent fluid pressure (EFP): The equivalent "hydrostatic" soil lateral pressure (i.e., obeying Pascal's Law). EFP values are the product of $K_a * \gamma$ or $K_p * \gamma$.

Coefficient of friction: This is the frictional resistance at the contact surface between the bottom of footing and the soil. It is a function of the roughness of the bottom of the footing, but it cannot excel Φ. Its value is usually between 0.25 and 0.40, with the latter commonly used. It is used to compute resistance to sliding by multiplying the total vertical force by the coefficient of friction. This, together with passive pressure resistance at the toe (toe & key), prevents the wall from sliding. Note that for cohesive soil, such as clay, the resisting force is the adhesion between the footing and soil, rather than the frictional resistance. This **cohesive** force is given in pounds per square foot of contact area, and is generally around 100 psf. Do not use the full value of cohesion for adhesion on other than very soft to soft clay, or else adhesion approximately equals cohesion times a reduction factor.

Soil modulus. Also known as the *coefficient of sub grade reaction*, designated "k", it is an indicator of the compressibility of a soil. It is often used to estimate the tilt of a cantilevered retaining wall. Its units are lbs. per cubic inch (lbs/in^3) and its value varies depending upon the size of the footing. Load tests to determine its value are done on a one-foot square loaded plate, and the value thus obtained must be adjusted for the width of the footing in accordance with the following often used formula:

$$k = k_1 \left(\frac{B+1}{2B}\right)^2$$ k_1 = value obtained by plate test; B = footing width.

The value k can vary from a low of less than 100 lbs/in^3 for loose sand to over 1000 lbs/in^3 for clay. Its value is should be provided by the engineer.

The Pickle Jar Test

I've done this and it's an interesting and informative way to learn about soil. Also, use it to give an approximate classification of your site soil. You need a tall slender pickle jar with clear glass and a capacity of at least two cups (16 oz.), but a similar jar will do. Scoop up a sample of soil to fill the jar about half full, preferably mostly sand with some silt and a little clay, but any soil will do. Pour water into the jar until the water reaches the soil surface. Now you have saturated soil, no change in volume, and you can visualize the voids. You were probably able to add a volume of water equal to about ¼ of the volume of sample soil. You will likely notice a slight slump of the soil because of consolidation.

Now pour in more water, screw the lid on tight, and shake vigorously for 30 seconds to mix the soil and water. Let the jar stand for 30 minutes. Watch how the soil settles and stratifies. You'll notice fairly clear lines of stratification: gravel to sand on the bottom, overlain with silt and probably a thin layer of clay on top, and maybe some floating organic debris. You can now classify your soil sample fairly well by comparing it with the Uniform System for Classification of Soil (see Appendix A).

Now remove the lid and push a table knife to the bottom. Wiggle it side-to-side and watch the pressure bulge. Then slowly withdraw it and notice the friction resistance. Fold a paper towel over the top and turn the jar upside down to drain the water. Watch the soil cling to the sides (adhesion). Let it dry for a few days (don't use the microwave!) then shake it up and pour it out. The slope of the soil is the angle of repose.

Play some more, it's a learning experience!

The Investigation

Most agencies require a soil report prepared by a geotechnical engineer to establish permissible soil design parameters and identify other geotechnical concerns for your project.

Here is a list of information that may be included in a report:

- Soil classification
- Allowable soil bearing value
- Adjustments in soil bearing for width and depth of footings.
- Passive soil pressure
- Active soil pressure for various backfill slopes.
- Coefficient of friction (concrete to soil).
- At-rest active pressure for restrained (non-yielding) walls.
- Presence of ground water
- Liquefaction potential.
- Slope stability analysis
- Seismicity (peak ground acceleration, proximity to faults, etc.)
- Presence of fill and site preparation requirements.
- Any other precautions the designer should be aware of.

Engineering is a specialty authority beyond the civil license and subject to state licensing laws similar to other professional disciplines.

Soil Bearing Values

Generally, allowable soil bearing pressures range from 1,000 psf to 4,000 psf. Additional increases are permitted for increased width and/or depth of a footing beyond the minimum values specified by the engineer. When applicable, these values can be increased by one-third when wind or seismic forces are present.

Although we leave the computation of allowable bearing value to the engineer, for those interested in the process, you can refer to Bowles, *Foundation Analysis and Design, 5th Edition*, Chapter 4, and other texts.

Alternatively, subject to acceptance by the local building official, you could use the presumptive values presented in the IBC '09 Table 1806.2. This table lists allowable bearing values for soil classified by the Uniform System for Classification of Soil in Appendix A.

When is a Soil and Foundation Investigation Required?

The local building official may have the authority to waive an investigation report if the soil is reasonably well known or a report was prepared for a nearby site. However, the requirements for when a report is required are specified in IBC '09, Section 1803.

5. BUILDING CODES AND RETAINING WALLS

What Building Code(s) Apply To My Project?

Always check with the building official having jurisdiction to learn what code(s) they are using and if any local amendments apply to your project.

The following codes are most often adopted or cited.

International Building Code (IBC)

This is now the dominant code adopted by most jurisdictions, some with local modifications (California Building Code, for example). The IBC was a culmination of efforts to merge the "model codes" (Uniform Building Code, Southern Building Code, and Standard Building Code) into one national 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 2009.

IBC 2009 references or modifies other standard codes, principally ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures*. Seismic design requirements for retaining walls per IBC are discussed in Chapter 7 of this book.

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.*

California Building Code (CBC) '07

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

There are not any specific seismic design requirements for retaining walls, with the exception of state-owned or leased public schools and essential service facilities, for which retaining walls over 12 feet require seismic design (see 1611A.6 of CBC'07). Also refer to Earthquake *(Seismic) Design*, Chapter 7.

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. Their current edition is *NFPA 5000: Building Construction and Safety Code, 2009 Edition.* The NFPA web address is www.nfpa.org.

This code references ACI 318, ASCE 7, and ACI 530 for structural design issues.

Referenced Codes

IBC 2009, CBC '07, and other regional codes often refer to the following standards for structural issues:

Minimum Design Loads for Buildings and Other Structures, ASCE 7-05
Published by ASCE. Reston, VA. This often referenced code covers loads and seismic design.

Building Code Requirements for Reinforced Concrete (ACI 318-08), American Concrete Institute (ACI), Detroit, MI. The standard for concrete design.

Building Code Requirements for Masonry Structures (ACI 530.1-08)
Also known as MSJC, this masonry code is published jointly by ACI, SEI, and The Masonry Society.

National Earthquake Hazard Reduction Program (NEHRP), 2003, developed by the Building Seismic Safety Council for FEMA (Federal Emergency Management Agency). This is not a code, per se, but referenced by IBC and NFPA as guidelines for seismic design. The 2003 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. This is discussed in Chapter 7 of this book.

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

Depending upon the jurisdiction, the following may also apply:

AASHTO LRFD Bridge and Highway Design Specifications, 4^{th}. Edition, 2007, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C. www.aashto.org.

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

U.S. Army Corps of Engineers Design Manuals. Comprehensive design procedures, standards, and sample calculations: The web address is: www.usace.army.mil.

6. 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 later):

- Lateral earth pressure
- Surcharge loads
- Axial loads
- Adjacent footing loads
- Wind on projecting stem
- Impact forces
- *Seismic earth pressure
- *Seismic wall self-weight forces

*Discussed in Chapter 7

Lateral Earth Pressures

The purpose of a retaining wall is to retain soil; hence the lateral pressure of the soil against the wall is a primary design concern. Most lateral pressure theories are based upon the sliding soil wedge theory. This, in simple terms, assumes that if the wall was suddenly removed, a triangular wedge of soil would slide down along a rupture plane, and it is this wedge of soil that the wall must retain.

The soil wedge theory

The development of the soil wedge theory for cantilevered retaining walls was discussed in Section 4. There are the two basic equations for computing lateral earth pressures: The Coulomb formula and the Rankine formula.

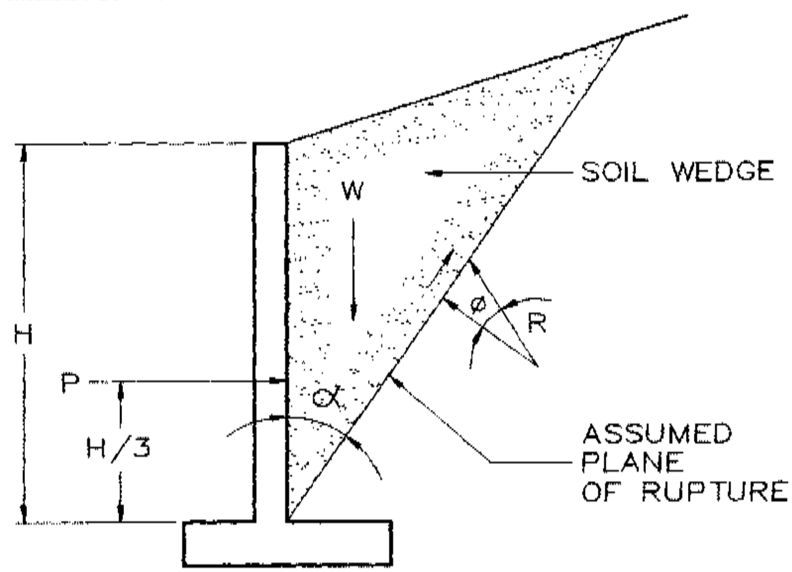


Figure 6-1. ***Free-Body of Lateral Forces***

The Coulomb Formula

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

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

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

β = Angle of backfill slope
ϕ = Angle of internal friction of the soil
α = Wall slope angle from horizontal (90° for vertical face)
δ = Angle of friction between soil and wall
(usually assumed to be $2/3\phi$ to $1/2/\phi$)

Figure 6-2. ***The Coulomb Formula***

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

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

The Rankine Formula

In the 1850s, Scottish engineer William Rankine further developed the Coulomb approach (along with many other scientific accomplishments) and introduced what is probably the most commonly used formula for lateral soil pressure. The Rankine equation is a simplified version of the Coulomb formula and 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 formulas. The lateral pressure factor Ka will be the same for the case of a level backfill and zero wall friction.

The Rankine Formula for active pressure:

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

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

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

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

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

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

0.41 K_a horiz. = .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 horizontal backfill

Figure 6-3. *The Rankine Formula*

Note that in the Rankine analysis the active pressure force is assumed to be applied at one-third the retained soil height and inclined at the angle of, and parallel to, the backfill slope.

IBC '09 and ASCE 7-05 have identical tables of minimum lateral pressures, condensed below:

Backfill Material	USCS Classification	Lateral Pressures (pound per square foot per foot of depth)	
		Active pressure	At-rest pressure
Well-graded, clean gravels; gravel-sand mixes	GW	30	60
Poorly graded clean gravels; gravel-sand mixes	GP	30	60
Silty gravels, poorly graded gravel-sand mixes	GM	40	60
Clayey gravels, poorly graded gravel-and-clay mixes	GC	45	60
Well-graded, clean sands; gravelly sand mixes	SW	30	60
Poorly graded clean sands; sand-gravel mixes	SP	30	60
Silty sands, poorly graded sand-silt mixes	SM	45	60
Sand-silt clay mix with plastic fines	SM-SC	45	100
Clayey sands, poorly graded sand-clay mixes	SC	60	100
Inorganic silts and clayey silts	ML	45	100
Mixture of inorganic silt and clay	ML-CL	60	100
Inorganic clays of low to medium plasticity	CL	60	100

Figure 6-4 – **Lateral Soil Pressures (Condensed from IBC '06 and ASCE 7-05)**

Surcharge Loads

A surcharge is any additional vertical load applied to the soil above the top of the wall. It can be live load from a parking lot or highway, paving or an adjacent footing. See **Figure 6-5** (Active Pressure from a Uniform Surcharge Load against Wall) to illustrate this effect.

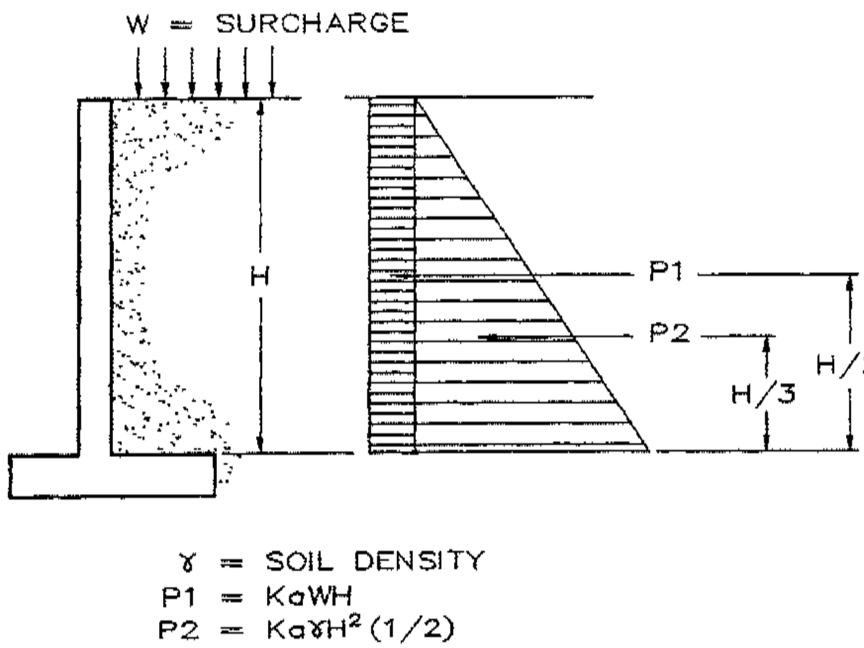


Figure 6-5. ***Active Pressure from a Uniform Surcharge Load against Wall***

Highway surcharges

The usual added surcharge when highway traffic is close to a retaining wall is to add two feet of earth. This is equivalent to a uniform surcharge of 240 psf (assuming a soil density of 120 pcf). A 250 psf surcharge is commonly used for highway loading. The lateral pressure will have minimal effect if the load is located more than the retained height away from the wall.

If H-20 truck loading could occur close to the wall (closest wheel within about one-half the height distant from wall) then a Boussinesq analysis can be done.

Backfill compaction surcharge

Backfill is often placed by a front end loader dumping sand or gravel behind the wall. The backfill should be placed in layers of about one foot, compacted by repeated by back-and-forth runs of the compactor or loader, coming within inches of the wall. Compaction testing may be required. A typical loader will weigh about 30,000 lbs, and have a footprint under each track of about 30 square feet if the loader is track-mounted. This results in short-time construction loads of about 1000 psf, far in excess of most surcharge design loads. Grading contractors are aware of this and often report tilting of the wall during these operations, and sometimes assign a worker to monitor plumb of the wall during these operations. Backfill compaction can produce a K_o condition, especially if the wall is restrained. It is the contractor's responsibility to place backfill so as to not damage or overstress a wall.

Adjacent footing surcharges

If there is an adjacent footing that overlays the area of the soil wedge this surcharge will exert lateral pressure against the wall, and must be considered.

A rule-of-thumb is that an adjacent footing will have little effect on lateral pressure against the stem if it is further than the height (base of stem to base of applied adjacent load) away from the wall face at a slope ratio of 1:1.

Adjacent footing loads are classified as either "line" or "strip" loads" which are uniform loads parallel to the wall, or "point loads", such as square or rectangular footings.

The Boussinesq equation, though computationally very laborious is often used to calculate the influence of adjacent loads on a wall is shown in **Figure 6-6.** Based upon the theory of elasticity, it results in a curved pressure diagram as illustrated in **Figure 6-7**.

The Boussinesq equation follows:

$$\sigma_r = \frac{P}{2\pi}\left(\frac{3r^2z}{R^5} - \frac{1-2\mu}{R(R+z)}\right)$$

Where terms are defined below and in **Figure 6-5** for a point load:

σ_r = Lateral Pressure, psf
P = Point load, lbs.
r = Horizontal distance from point of application on wall to plumb under P
z = Depth, ft.
R = Diagonal distance from P to point of application
$= \sqrt{r^2 + z^2}$
μ = Poisson's ratio

NOTE: If a "line load", multiply the computed lateral pressure from the above equation by $2\pi = 6.28$ (derived from the Boussinesq equation).

The resultant force acts about 0.60h above the bottom. Also, note that the Boussinesq formula is sensitive to the assumed Poisson's ratio (μ) for the soil. This value for sand and sandy-clay ranges from about 0.2 to 0.5. The Bowles text is an excellent reference on the use of the Boussinesq equations.

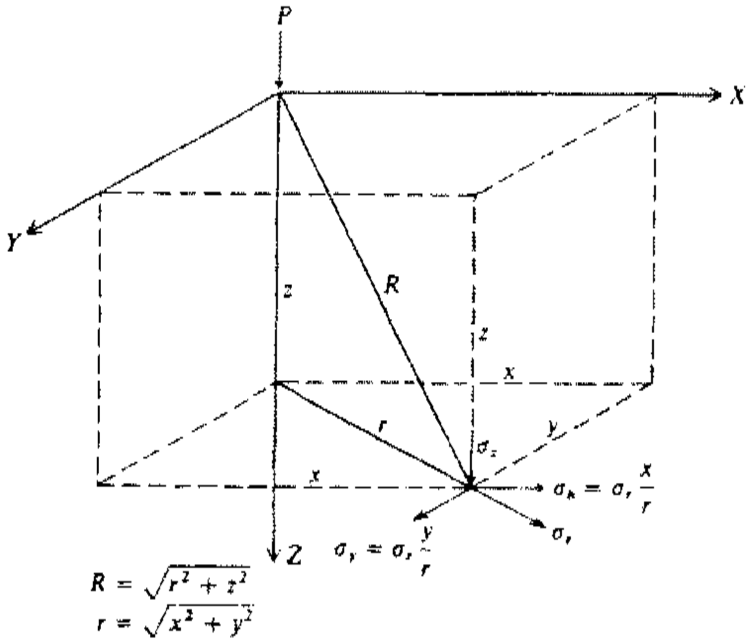


Figure 6-6. ***Boussinesq Equation***

Figure 6-6: (Boussinesq Equation Pressure Diagram) shows a plot of the resulting pressure curve.

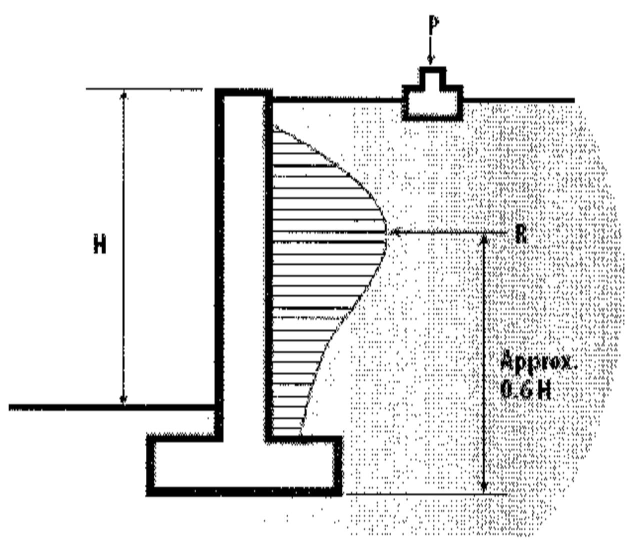


Figure 6-7. ***Boussinesq Equation Pressure Diagram***

This is a very time consuming computation because it requires a calculation for each increment of wall height. The computations are further compounded if it is other than a point load, because it then requires a separate computation for each square foot under the footing, or in the case of a line load or strip load (the latter is a line load but several increments wider, such as a wide footing parallel to the wall), numerous computations must be made for the cumulative pressure affects against a vertical unit length of wall.

Terzaghi and Peck propose a simple method for computing the lateral load and point of application from a line load (e.g. continuous footing) behind the wall.

The line load P, acting at a distance x from the wall, exerts a resulting force R acting against the wall at a distance y below load P, where $y = x (\tan 40°)$.

$R = C_1 P$, where C_1 is a lateral pressure coefficient depending upon the type of soil (paraphrased from Terzaghi, page 364): 0.26 for clean sand and gravel; 0.30 for coarse grained soil with some silt; 0.39 for fine silty sand and some clay; 1.00 clay and silty clay. For simplicity, the above **Figure 6-7** uses $C_1 = 0.30$.

However, neither of these computations provides the distribution of lateral wall pressure against the wall. A simplified solution may be to assume a uniform load against the wall equal to the adjacent footing load divided by the height from footing to bottom of adjacent footing. This would yield overly conservative moments and shears near the top of the wall, and un-conservative shear and moment at the base.

Some engineers merely assume the bearing pressure under the adjacent footing projects downward at a 1.5:1 slope (Terzaghi proposed an equilateral triangle), and computes the adjusted surcharge at the level where the projection intersects the wall. See **Figure 6-8.** This may be an underestimation because it does not give the pressure bulge near the top as shown by the Boussinesq equation.

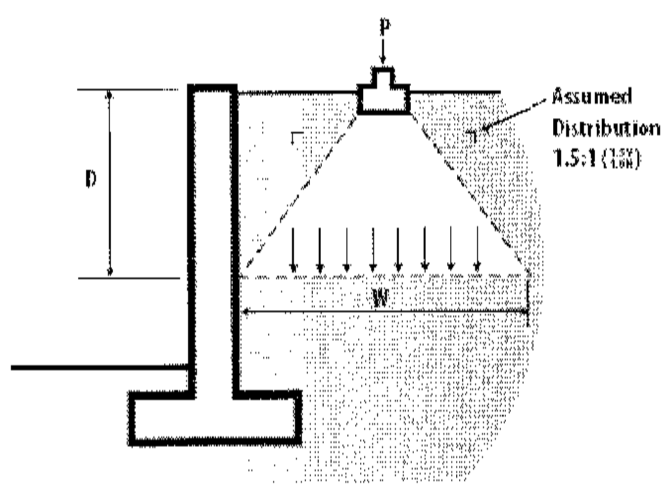


Figure 6-8. ***Simplified Lateral Pressure from Line Load***

This treatment of adjacent footing loads is brief; a more in-depth treatment is not within the scope of this book. For further reading consult the texts in the bibliography, **Appendix F**. NAVFAC and AASHTO also have frequently referenced charts for determining lateral pressures from line and point loads.

Wind Load on Projecting Stems

When an exposed wall extends above grade, it is subject to wind pressure which creates an additional overturning force. The customary formula for wind pressure is $F = .0026\ v^2$, where F is in psf and v is wind velocity in mph.

Using IBC 2006, reference is made to ASCE 7-05 for wind design (with some exceptions generally not applicable to "fences" – see IBC Section 1609). Equation 6-25 of ASCE 7-05 gives the following simplified formula:

> $F = q_z\ G\ C_f$, where F is design wind pressure in psf, G is gust factor which can be taken as 0.85, C_f can generally be taken as 1.2, and q_z is the velocity pressure at mid-height and can be computed by Equation 6-15:
>
> $q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I$, which terms can be determined from Section 6.5.10. V is wind velocity in mph.
>
> For example, considering Exposure "C", 80 mph wind, and ignoring the Importance Factor "I", this results in "F" = about 12 psf.

If wind overturning and stem moments are significant stress components, the IBC code permits a one-third stress increase for short-term loading if ASD combinations of 1605.3.2 are used. Except for freestanding walls the one-third increase is generally not applicable. For Strength Design the IBC load factor for wind is 1.6 when using strength design method.

Other codes and conditions may apply when wind is a consideration.

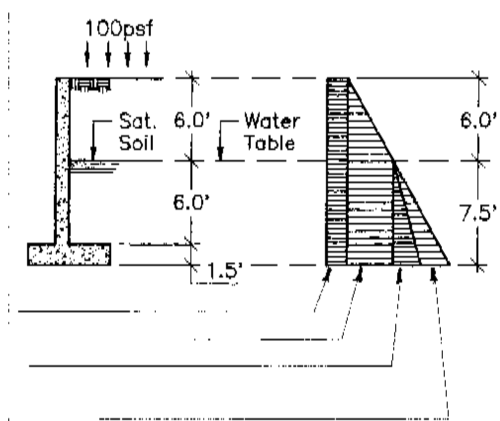
Water Table Conditions

If a portion of the retained height is below a water table, the active pressure of the saturated soil will increase below that level. This additional pressure for the saturated soil is equal to the pressure of water, plus the submerged weight of the soil (its saturated weight - 62.4), plus the surcharge of the soil above the water table. The submerged weight of a soil can be approximated as 5/8 x its dry unit weight. This pressure diagram is shown in **Figure 6-9.** *Water Table Force Calculations*.

Assume: $K_a = 0.27$
γ = 110 pcf
Surcharge - 100 psf
Water = 62.4 pcf

Forces:

0.27 x 100 psf = 27 psf.
0.27 x 110 x 6' = 178.2
0.27 (110-62.4) x 7.5= 96.4
62.4 x 7.5 = 468



Summary for overturning:

P			$\bar{x}$	M
27 x 13.5	=	365#	6.75'	2,464'#
178.2 x 6/2	=	535#	9.5'	5,079'#
178.2 x 7.5	=	1337#	3.75'	5,012'#
96.4 x 7.5/2	=	362#	2.5'	904'#
468 x 7.5/2	=	1755#	2.5'	4,388'#
		4354#		17,847'#

Ht. to point of application = 17,847 / 4,354 = 4.10'

Figure 6-9. ***Water Table Force Calculations***

Detention ponds / flood walls

When retaining liquids the procedure is similar to an earth retaining wall except that the equivalent fluid pressure is 62.4 pcf (or that of the liquid). If the liquid can seep under the footing, then the pressure above and below the heel equalize and only the buoyant-adjusted dead weight of the heel can be used to resist overturning.

Hydrostatic pressures

If the water table is above the foundation the soil density below must be adjusted for buoyancy (saturated weight minus water weight). Weight of saturated soil is about 10% - 13% grater than dry weight. The footing weight should be reduced (concrete weight less water weight) to account for its buoyancy.

Cascading walls

Occasionally walls will be stacked one behind another, piggyback style, or cascading, as sketched in **Figure 6-10.**

This requires very careful design for the lower walls, because not only is there a surcharge from the wall above, but a horizontal thrust as well. Two possible solutions are suggested:

Alternate #1 shown on **Figure 6-10**, would be to sketch a fail-safe slope that would model the event if both walls were considered one mass exerting pressure on the lower wall.

Another possible solution, Alternate #2, suggested by a engineer, is to apply the vertical load P_v of the lower wall as a line-load surcharge force P_v located "x" from the wall, use the Boussinesq equation to obtain a cut force on the wall, then apply its horizontal thrust P_h as an assumed uniform load against the stem of the lower wall.

Cascading wall conditions come up frequently and a good reference for design is not known to the author (it would make a good PhD thesis!).

For this condition, be careful, consider the horizontal thrust of the upper walls, and be conservative! advice is recommended and the nature of the underlying soil may require a global stability analysis (most consultants have software to perform this).

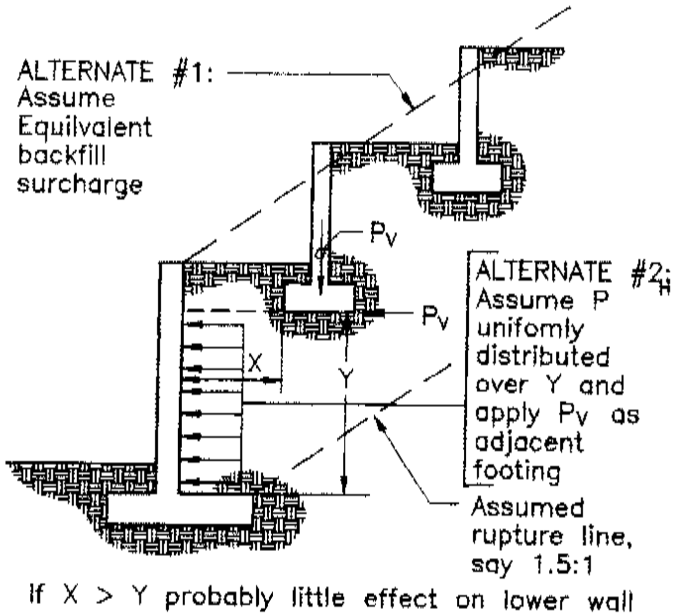


Figure 6-10. ***Cascading Walls***

Vertical Loads

Vertical loads provide stability by resisting overturning.

Vertical loads include:

Axial loads on the stem

These loads are applied directly to the stem, such as from a beam reaction, ledger, or bridge member. Any vertical load imposed upon the stem of a cantilever design retaining wall must not provide lateral support, otherwise the wall does not perform as a cantilevered retaining wall. If one side of a building, for example, rests on top of a wall – it could be reactions from a floor or roof – the abutting diaphragm should not restrain rotation of the wall. If restraint does occur, the wall should be designed as a "basement wall," whereby the restraint at the top results in a positive bending moment in the stem. Sometimes a wall is designed for both conditions, such as when it is designed as a retaining wall so that backfill can be safely placed before the restraint is provided, then designed as a basement wall for the condition after the restraint is in place.

Axial live loads on the stem will increase soil bearing pressure and resisting moments, therefore need to assessed separately from axial dead load for most critical condition.

Vertical point loads on walls, such as from girder reactions, are assumed to spread downward at a slope of two vertical one horizontal. This spreading of the load results in relatively low compressive stresses at the base of the stem. For example, a 24 kip load atop a 12" concrete wall on a two-foot wide bearing, and assuming 14 ft high, would result in an axial stress (in addition to wall weight) of just 125 psi. Bearing stresses directly under a beam or girder reaction must be checked.

Also, consider the eccentricity with respect to the stem centerline because it will affect both stem design and stability. But remember that live load acting at a negative eccentricity (toward the backfill) could produce unconservative results.

Axial loads are usually from a connecting floor or roof, and rarely would exceed about 2,000 lbs/ft. and typically much less. This results in relatively low axial stress in the stem. For example, for a 12" masonry wall this added compressive stress for 2,000 lbs/ft would be [2000 / (11.5 x 12)] = 14.5 psi. Caution should be used if for some reason a very high axial load was applied because it could change the bending characteristic of the footing. For example a very high heel soil pressure because of a high axial load could reverse the bending from negative to positive, simulating a spread footing design.

Weight of soil

This includes the soil over both the heel and toe. This is assumed to be a straight-up-and-down column of earth (although in actuality this is probably an unlikely assumption).

Weight of structure

This includes the weight of the stem and footing.

Stem and footing weight

These loads add to soil bearing pressure and contribute to overturning and sliding stability.

Vertical component of active pressure

The vertical component of active pressure force is another vertical load. If the backfill is sloped as illustrated in **Figure 6-1**, then the line of action of the resultant earth pressure, P_v, is inclined from the horizontal. When using the Rankine formula, this inclination is assumed to be the same angle (parallel to) the backfill slope. In the Coulomb method, P is inclined at the wall friction angle at the soil-stem interface. These inclined forces resolve into a horizontal and vertical component. The latter is assumed to act at the plane of the back of the wall footing heel for a cantilever wall. This vertical component could be used for added resistance to sliding, reduced soil pressure, and increase the overturning resistance. Most textbooks advise that it be used only for overturning resistance; it is conservative to ignore the stabilitizing influence of this force.

Impact Loading

If the wall extends above grade and a parking area is adjacent, you may want to design for impact from a car bumper. ASCE 7-05 specifies 6,000 lbs. applied at a height of 18" above grade. Guard rails require 50 plf applied to the top, or a single concentrated lateral load of 200 lbs. A short-term stress increase of 1.33, or more for impact, would seem appropriate for these conditions.

When considering the effect of impact, the stem should be checked at incremental descending points as the impact force spreads over a greater stem length. Assume the impact load spreads out at one horizontal to two vertical. This is equivalent to spreading its effect over a length of wall equal to the distance from point of application down to the plane being checked.

7. EARTHQUAKE (SEISMIC) DESIGN

Seismic Design Overview

Texts that address seismic design of retaining walls (e.g. Bowles, Kramer) acknowledge that seismic design of retaining walls is a highly complex issue, compounded by the assumptions that must be made to allow an indeterminable problem to become solvable using concepts of statistics and differential calculus. The determination of both static and seismic (dynamic) pressure on retaining walls is still an emerging science. The selection of an "effective" site acceration for use is somewhat arbitrary, though becoming codified. reports usually give only the peak ground acceleration applicable to the location and leave the application of this information to the designer.

Some argue the necessity of seismic design of retaining walls, considering compensating safety factors (e.g. 1.5 or higher for overturning) and lack of seismic damage incidents to retaining walls (waterfront structures subject to liquefaction excepted). It is also argued that because retaining walls are often at a distance from structures that would be affected by such failures and thus are not a life-safety issue. However, these arguments appear moot considering the mandatory language of IBC and ASCE 7.

The seismic requirements of IBC 2006, and IBC 2009, Section 1613.1, refers to ASCE 7–05 which in Section 9.14.7.2.1 reads as follows:

> '. . . This section applies to all earth retaining walls. The applied seismic forces shall be determined in accordance with Section 9.7.5.1 *[This section states that "... the owner shall submit to the authority having jurisdiction a written report that includes an evaluation of the items in Section 9.7.4.1 and the lateral pressures on basement and retaining walls due to earthquake motions". Section 9.7.4. identifies items to be included in the report to be submitted ", when required by the authority having jurisdiction"]*

This clearly requires a seismic analysis of "earth retaining structures", based upon the recommendations of the report, but implies some discretionary latitude by the "authority having jurisdiction".

However, IBC '06, 1802.2.7 or IBC'09, 1803.5.12, requires "A determination of lateral pressures on basement and retaining walls due to earthquake motions", but exempts this requirement if a peak ground acceleration of S_{DS} / 2.5 is used for design. This is the k_h acceration used in the M-O equation (See following) and presented in NEHRP, Part 2, Commentary, 7.5.1.

The now defunct Uniform Building Code (UBC '97) and its successor California Building Code (CBC '07) do not appear to specifically require seismic design of "earth retaining structures" with the exception of state-owned or leased public schools and essential faculties, which require a seismic design if the retaining wall is more than 12 feet high.

Designer should check applicable local and State codes that may have specific seismic design requirements for retaining walls. They often vary with jurisdiction.

The Mononobe-Okabe equations

Of the many investigations of dynamic forces on retaining walls, one of the most important and influential is 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 this paper they cite the pioneering studies by Mononobe (1929) and Okabe (1926), widely referenced today. Another contribution was a subsequent ASCE paper by Robert Whitman titled, *Seismic Design and Behavior of Gravity Retaining Walls,* 1990. They considered this lateral force to be an inverted triangular wedge of soil behind the wall. Seed-Whitman proposed a simplified formula, based upon the Mononobe-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 was an adaptation of the Coulomb formula to calculate the <u>total</u> (seismic and static) pressure and introduced the variable θ, which is defined as the angle whose tangent is the ground acceleration ($\theta = \tan^{-1} k_h$).

This equation is presented in **Figure 7-1.**

K_{AE} = active earth pressure coefficient, static + seismic

$$\frac{\sin^2(\alpha+\theta-\phi')}{\cos\theta' \sin^2\alpha \sin(\alpha+\theta'+\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\theta'-\beta)}{\sin(\alpha+\delta+\theta')\sin(\alpha-\beta)}}\right]^2}$$

Where $\theta = \tan^{-1} k_h$, α = wall slope to horiz. (90° for a vertical face), ϕ = angle of internal friction, β = backfill slope, and δ – wall friction angle.
The horizontal component is $K_{AE} \cos \delta$.

For a vertical wall face and δ assumed to be $\phi/2$, this becomes:

$$K_{AE} = \frac{\sin^2(90+\theta-\phi)}{\cos\theta \sin^2(90+\theta+\phi/2 \left[1+\sqrt{\dfrac{\sin 1.5\phi \sin(\phi-\theta-\beta)}{\sin(90+\phi/2+\theta)\sin(90+\beta)}}\right]^2}$$

The total force (active and earthquake), $P_{AE} = \frac{1}{2} (\gamma) K_{AE} H^2$ where γ = soil density and H = retained height.

Figure 7-1. ***Mononobe-Okabe Equation***

When the acceleration is zero, $k_h = 0$, K_{AE} becomes the familiar Coulomb K_A formula.

The passive earth pressure coefficient, K_{PE} is:

$$K_{PE} = \frac{\sin^2(\alpha - \theta + \phi')}{\cos\theta' \sin^2 \alpha \sin(\alpha + \theta' + \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta' + \beta)}{\sin(\alpha + \delta + \theta')\sin(\alpha - \beta)}}\right]^2}$$

Note: The passive pressure coefficient decreases under seismic conditions.

K_{AE} has two components (seismic and static). The seismic component (K_{AE} - K_A) is assumed to be an inverted, near-triangular, trapezoidal pressure diagram with the resultant force (maximum at the ground surface) acting at a height of 0.6 H. For stem design, H is the height from top of footing to retained height; for overturning and sliding, H is the height to the back face of the footing, along a virtual vertical plane extending from the bottom of the footing to its intersection with the backfill grade.

The K_A component is the familiar triangular distribution acting at H / 3.

The height to the combined resultant can be obtained by the formula:

$$\bar{y} = \frac{P_A(H/3) + (P_{AE} - P_A)\ 0.6H}{P_{AE}}$$

The direction of force application, per the Coulomb formula, is assumed to be inclined at an angle (from horizontal) equal to the friction angle at the back face of the wall, δ, which is often assumed to be $\phi/2$. Therefore, the horizontal components can be assumed to be

$$P_{AE\ horiz.} = \cos\left(\phi/2\right) P_{AE}.$$

A simple approach to the design for seismic is suggested by the overlapping force triangles, which tend to combine into a uniform load over the height of the wall, if the height of the resultant is at 0.5H.

Therefore, $w = \dfrac{K_{AE}\,\gamma\,H^2}{2H} = 0.5\ K_{AE}\,\gamma\,H$, where w is the equivalent uniform lateral static plus seismic force. This simplification, is particularly helpful for checking stem moments and shears at various heights when $\bar{y}$ = H / 2.

Seed and Whitman (1982) suggest an approximation of $K_{AE} = K_A + 0.75\ k_h$. If, for example, k_h is 0.30 and K_A = 0.27, then K_{AE} approximate would be 0.495. This would suggest an 83% increase over static K_A. However, the stem moments and overturning are greatly increased because 0.75 k_h act at an assumed height of 0.60H. Some engineers use this method to give an added uniform seismic force over the full retained height with resultant acting at 0.6H. Such a requirement, therefore, might read "for seismic design add a uniform lateral force = 20 H^2 with the resultant applied at 0.6H". *Note that the inverted "triangle" is actually trapezoidal because of 0.6H, however, calculation error resulting from assuming a triangular distribution is not significant.*

Seed and Whitman's paper suggests that few building codes (at that time) required seismic provisions for retaining walls, and concluded that the factors of safety for static design, which are generally around 1.5, are adequate to protect the wall for short term seismic forces, because such forces would merely reduce the safety factor to an acceptable value greater than 1.0.

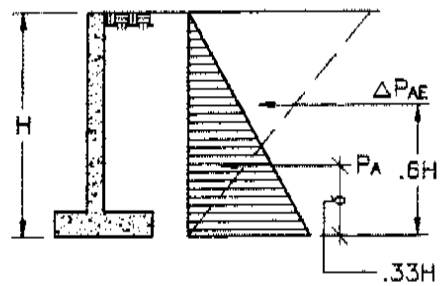


Figure 7-2. ***Application of Seed Whitman Method***

An arguable issue is whether to include the inertial force of the wall combined with seismic earth pressure both NAVFAC and U.S. Army Corp of Engineers appears to require this concurrent force. It seems excessively conservative. If designing for these agencies check their requirements.

Seismic forces are factored forces, generally 1.0, and for ASD these forces can be reduced by 0.7 to convert for design for overturning, sliding and soil bearing. Additionally, the factor of safety may be 1.1 when seismic is included. See IBC 2009, 1807.2.3.

Determining k_h

k_h is the horizontal ground acceleration used in the Mononobe-Okabe (M-O) equation to compute lateral seismic earth pressures against retaining walls. This is a design value and not necessarily the most severe acceleration that could occur at the site. Unless an arbitrarily reduced value of k_h is used, one-third to one-half the peak ground acceleration is often used (see Kramer and others).

The starting point is to determine the peak acceleration applicable to your design. Assuming your code is IBC 2009, or ASCE 7-05, which have identical charts, select from the contours the Maximum Considered Earthquake (MCE) ground motion for 0.2 second, spectral response acceleration at 5% of critical damping, with a 2% probability of exceedence in 50 years. Note that retaining walls are "short period", hence the 0.2 second selection.

There is an easier way. Go to http://earthquake.usgs.gov/research/hazmaps/design/index.php (Java required). This is a U.S. Geological Survey address. Just enter your zip code. (A latitude-longitude web.) For example, Newport Beach, California (high seismic area!), zip code 92660, gives 185.6 percent "g", or 1.856, and a peak ground acceleration (PGA) of 84.75%g.

Here is an example procedure for obtaining a design k_h using the USGS Hazard Maps:

From charts, $S_s = 1.856$
(All terms defined in referenced codes)

$S_{MS} = F_a\ S_s$

$F_a = 1.0$ (This is a function of soil characteristics and value of S_s. See Table 1613.5.3 in IBC '06).

$\therefore \quad S_{MS} = 1.0 \times 1.856 = 1.856$

$S_{DS} = 2/3\ S_{MS} = 0.667 \times 1.856 = 1.24$

Per NEHRP, Part 2, Commentary, 7.5.1: See also IBC '09, 11.8.3

$$k_h = \frac{S_{SD}}{2.5} = 0.40 \times 1.24 = 0.50$$

Simplified Seismic Force Application

The NEHRP 2003 Part 2 Commentary, 7.5.1, states Seed and Whitman's proposed simpler approximation:

$\Delta K_{AE} \sim (3/4)\ k_h \qquad \therefore\ \Delta P_{AE} \sim (1/2)\gamma H^2(3/4)k_h \sim (3/8)k_h\gamma H^2$

k_h is the peak ground acceleration modified per Provisions Sec. 7.5.1:

where $k_h = S_{DS} / 2.5$

Base moments, using this simplification, are therefore:

$$M_{AEbase} = P_A\ (H/3) + (\Delta P_{AE})\ (0.6\ H)$$
$$= \gamma H^3\ (0.17\ K_a + 0.225\ k_h)$$

An observation from this is that the base moments from static and dynamic (seismic) are equal when $k_h \sim 0.75\ K_A$

Vertical Distribution of Seismic Force on Stem

Here is a simplified method for assuming a uniformly applied force to the stem:

By Definition: $\quad P_{AE} \quad = \quad P_A + \Delta P_{AE}$

$$\Delta P_{AE} \simeq \frac{\gamma H^2}{2}\ (.75\ k_h) = .375\ k_h\ \gamma H^2$$

$$P_A = \frac{K_a\gamma H^2}{2}$$

Total force on stem: $\quad P_A + \Delta P_{AE} = .5K_a\ \gamma H^2 + .375\ k_h\ \gamma H^2$

If resultant acts at 0.5 H, the approximate uniform lateral pressure on a stem is:

$$\frac{.5K_a\ \gamma\ H^2 + .375\ K_h\ \gamma\ H^2}{H} = (.5\ K_a + .375\ k_h)\ (\gamma H)$$

For design example, assuming $K_a = 0.35$, $k_h = .34$, $\gamma = 120$

$F_P = 36H$ lbs / ft lateral pressure

Note that this simplified formula is not valid if there is a sloped backfill which would significantly increase seismic forces.

Seismic for Stem Self-weight

This is an arguable issue: whether to include the seismic force due to self-weight of the wall acting simultaneously with the lateral seismic to earth pressure force. It does not appear to be defined in the codes. AASHTO, however, in 5.5.4 states: "*...seismic design forces should account for wall inertia forces in addition to the equivalent static force, where a wall supports a bridge structure...*". But section 5.6.4, referring to flexible cantilever walls, states that "*Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure*".

Judgment indicates that seismic self-weight should be applied simultaneously with seismic due to earth pressure.

Using ASCE 7-05, Section 15.6.2 (Rigid Nonbuilding Structures):

F_P in equation 15.4-5, for cantilevered wall and assuming $Ip = 1.0$,

Reduces to: $Fp = 0.30\, S_{DS}\, W_p$

Per above design example where $S_{DS} = 0.85$,:

$$F_P = 0.30 \times 0.85 \times 1.0\, W_P = 0.26\, W_p$$

Alternatively per ASCE 7-05, 13.3.1:

This method for F_p applies if there is a lateral support at top.

$$F_P = \frac{0.4 a_p S_{DS} I_p}{R_p}\left(1 + 2\frac{h_x}{h_r}\right) W_P$$

$$a_p = 1.0,\quad R_P = 2.5,\quad \frac{h_x}{h_r} = 0 \text{ at bottom and } 1.0 \text{ at top.}$$

F_p minimum is $0.30\, S_{ds}\, I_p\, W_p$

F_p for design is average between top and bottom

For example design, $S_{ds} = 0.85$, $I_P = 1.0$

$$\therefore F_P = [[(0.4 \times 0.85 \times 1.0 \times 1.0 \times 3) / 2.5] + (0.30 \times 0.8 \times 1.0)]]\, 0.50W_p = 0.33W_p$$

Seismic Force on Non-Yielding (Restrained) Walls

Several texts (e.g. Kramer) propose the following formula (slightly revised):

$\Delta P_{eq} = \gamma\ k_h\ H^2$, acting at a resultant height of about 0.6H

Where ΔP_{eq} is the total added lateral force due to seismic, γ is the unit weight of soil, and H is the retained height.

The resultant acting at 0.6H gives a slightly trapezoidal force diagram, however, for ease of calculation a uniform load can be assumed with less than 2% unconservative error.

It should be noted that there are so few incidents of earthquake damage to such walls that many experts agree that seismic design of restrained (e.g. "basement") walls may not be necessary, particularly given an adequate factor of safety for the service level design.

8. DESIGNING THE CANTILEVER WALL STEM

Basics of Stem Design

First, here are two very rough rules of thumb for assuming stem thickness: If a concrete stem, try one inch in thickness for each foot of retained height, but not less than eight inches. If masonry stem, 8" is usually adequate for walls about six feet high, and 12" for walls to 12 feet. Less height for walls with sloped backfills.

The controlling design condition for reinforcement occurs at the bottom of the stem (top of footing), where the maximum stem moment occurs. 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 this point alternate bars can be dropped, or sizes reduced, for economy. If the wall is very high, you may want three or four cut-off levels and perhaps a change in stem thickness or material. The diagram in **Figure 8-1** illustrates this concept.

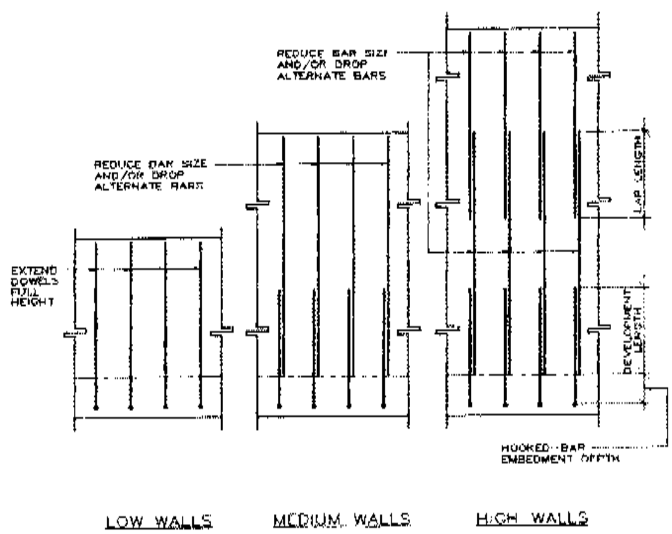


Figure 8-1. ***Reinforcing Placement in Stem***

A handy rule to remember is that for a triangular equivalent fluid pressure behind the stem, the moment diminishes to one-half of that at the base 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. Therefore, for nearly all cases, the moment is one-half or less at the tops of the dowels.

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, with very little earth retention, then remember that the wind can blow from either direction, which will require the wall and footing to be checked for both conditions.

If there is a concentrated vertical or lateral load, such as impact, assume that a point load spreads downward, along the length of the wall, at about two vertical to one horizontal. In other words, the overturning force at the base of the stem is spread over a distance equal to the height from the base to point of impact.

Dowels from Footing into the Stem

The reinforcing at the bottom of the stem will consist 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 (as stated earlier is halved at about $1/5^{th}$ the height). Vertical bars must only extend up to where they are no longer required, at which point either alternate bars can be dropped, or spliced (lapped) with lesser size bars.

Bars 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 they extend at least 12 bar diameters beyond where they are no longer needed for moment requirements. 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 below the Table.

Lap Splice Lengths(1) and Hooked Bar Embedments (inches)

Bar Size		Masonry(2) f'_m =1500 psi		Concrete (3)		
		Grade 60	Grade 40	2000 psi	3000 psi	4000 psi
#4	L	24	20	34.9	28.5	24.7
	H(4)			9.4	7.7	6.7
#5	L	30	25	43.6	35.6	30.8
	H(4)			11.8	9.6	8.3
#6	L	36	30	52.3	42.7	37.1
	H(4)			14.1	11.5	10.0
#7	L	42	35	76.3	62.3	54.0
	H(4)			16.5	13.4	11.6
#8	L	48	40	87.2	74.2	61.6
	H(4)			18.8	15.4	13.3

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

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

(3) Min. 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 that IBC '06, 2107.5, modifies ACI 530-05, Section 2.1.10.7.1.1 which has the effect of deleting the following onerous development length equation (2-9) in ACI-530-05:

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

Development length in masonry is given in MSJC 2008 as:

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

γ	= 1.0 for #3,4,5 bars, 1.4 for #6, 7, and 1.5 for #8
K	– Masonry cover but not less than 5 d_b

This requirement results in much longer lap lengths and has met with considerable objection. IBC 2009 modified this requirement (only for ASD) to: $l_d = 0.002\, d_b f_s$ but not less than 12".

For Grade 60 reinforcing this equation requires 48 bar diameters.

Horizontal Temperature / Shrinkage Reinforcing

Horizontal reinforcing is necessary to control cracks from temperature changes and shrinkage. The table below shows minimum requirements for both concrete and masonry (CMU). There may be conditions (climate, aesthetics, better crack control) where you may want additional reinforcing.

Horizontal Temperature/Shrinkage Reinforcement for Concrete and Masonry Walls

Typical Horizontal Rebar Spacing for .0007 A_g Masonry and .002 A_g for concrete						
Mat'l	**Thick**	**#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

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

Key at Stem-Footing Interface

Another use of the term "key" is a longitudinal slot formed into the top of the footing and into which the bottom of the stem fits. This slot can be the full width of the stem, or just the middle half. The purpose is intended to offer more shear resistance at the interface plane. We often compute the shear stresses at the base of the stem as if it was monolithic, rather than a "cold joint." By providing a keyway, all or part of the shear can be resisted by compression against the side of the keyway, if its depth is sufficient to resist the shear force.

However, another way of resisting shear at this interface is to consider "shear friction" across the joint. Shear friction theory considers the reinforcing steel that crosses the joint as clamping the joint together such that sliding of the joint cannot occur unless the coefficient of friction is overcome, or the reinforcing yields to allow slippage. This requires a certain amount of tension in the reinforcing must be used for this clamping force, which is in addition to tension requirements for bending design. Let's investigate this for an assumed condition:

$$v_u = \frac{3800}{12 \text{ x } 9.63} = 32.9 \text{ psi}$$

$$v_{allow} = \phi 2\sqrt{f'_c}$$

$\phi = 0.75$

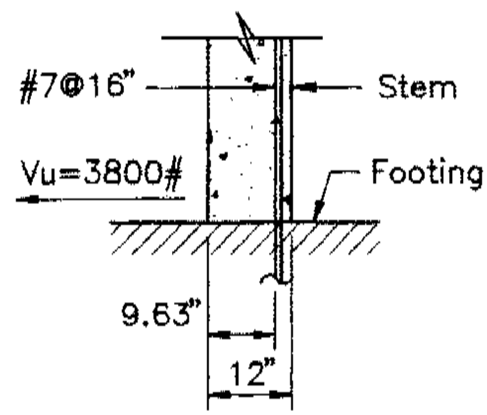
$= .75 \text{ x } 2\ \sqrt{2000}\ = 67.1 > 32.9$ OK

But also check shear friction available:

$v_u = A_s\ f_y\ \mu \leftarrow$ assume = 0.60 coef. of friction

$= \frac{0.60}{1.33} \text{ x } 60{,}000 \text{ x } 0.60 = 16{,}240^{\#} > 3{,}800^{\#}$

$\therefore$ OK if only consider shear friction



In this case, concrete shear is adequate, but it can be seen that shear friction offers considerable resistance if necessary.

Alternatively, you could use shear values for embedded bolts – in this case 7/8" "bolts" at 16" o.c. =3350# / 1.33 = 2519 plf – assuming 2000 psi concrete or grout.

Design of Masonry Stems

Masonry is designed using two methods: Allowable Stress Design (ASD) and Strength Design (SD). Both are code-permitted options

Using ASD, loads are factored by 1.0, except earthquake forces are already factored, therefore to convert seismic forces to ASD divide by 1.4. Allowable flexural stress is: $f_m = 0.33\ F_m$.

Strength Design, also known as LFRD (Load Resistance Factor Design), is the design procedure similar to Strength Design for concrete. Use load factors per ASCE 7-05. Strength-reduction factor, Φ, = 0.90 for flexure and 0.80 for shear and splices. f'_m is typically 1500 psi and f_y for Grade 60 reinforcing is 60,000 psi. Refer to MSJC Chapter 3 for SD requirements.

In **Appendix B** you will find a summary of masonry design formulas and allowable stresses.

An excellent reference for masonry design is James Amrhein's *Reinforced Masonry Engineering Handbook, 4th Edition*, published by the Masonry Institute of America, Los Angeles. Somewhat outdated code wise, but good information. Another good reference is *Masonry Designer's Guide*, 4th Edition by the Masonry Society (www.masonrysociety.org).

Masonry stem thickness is nominal block thickness[1]: 6", 8", 10", 12, and rarely, 14" or 16". With six-inch walls the reinforcing must be placed in its center, but on thicker walls the bars can either be centered or next to the inside face (the face adjacent to the earth!). Shear and moment calculations are based upon the "effective depth d" of the moment.

See Table 1 below for "d" dimensions for various masonry stem thicknesses. These are industry standards, and assume about 2" from the face of wall to the centerline of the bar.

When the stem thickness is reduced higher up the wall, the step should be made on the inside (earth side) so that the outside of the wall is a flush vertical surface. When stepping the wall, consideration must be given to providing sufficient lap development length for the reinforcing extending into the section below.

Table 1 — Typical "d" Distances for Masonry Stems

Bar	**"d" Distances for Masonry Stems**				
Position	**6" wall**	**8" wall**	**10" wall**	**12" wall**	**16" wall**
Bars in center	2.8	3.8	4.8	5.8	—
Bars at edge		5.25	7.25	9.0	13.0

Concrete masonry units (CMU) are designated either lightweight, medium weight (most common) or heavy weight, and are either solid grouted, or grouted only for cells containing reinforcing are grouted. Remember that for masonry stems the vertical bars must be spaced on eight-inch modules to accommodate the block cells. Although only cells containing reinforcing need be grouted, it is usual to solid grout the wall. The wall weights for these combinations are shown in the Table below:

Table 2 — Weights of Masonry Walls

		Concrete Masonry Units											
		Lightweight 103 pcf				**Medium Weight 115 pcf**				**Normal Weight 135 pcf**			
Wall Thickness		**6"**	**8"**	**10"**	**12"**	**6"**	**8"**	**10"**	**12"**	**6"**	**8"**	**10"**	**12"**
Solid Grouted		52	75	93	118	58	78	98	124	63	84	104	133
Vertical Cored Grouted at	16" o.c.	41	60	69	88	47	63	80	94	52	66	86	103
	24" o.c.	37	55	61	79	43	58	72	85	46	61	78	94
	32" o.c.	36	52	57	74	42	55	68	80	47	58	74	89
	40" o.c.	35	50	55	71	41	53	66	77	46	56	72	86
	48" o.c.	34	49	53	69	40	45	64	75	45	55	70	83

To relieve water pressure, "weep joints" should be provided at the lowest course at the outside grade. This can be done by omitting the head joint (side joint between blocks) at every other block, or 32" on center. Specify gravel behind so the joints won't clog.

Minimum reinforcing in masonry stems

The ACI Code, MSJC 2008 requires that the sum of the vertical and horizontal reinforcing ratios be at least 0.002 and that the least in either direction be 0.0007. Spacing should not exceed 48 inches. As the principle reinforcing is always vertical, it should be at least 0.0013 times the gross cross-sectional area, and at least 0.0007 horizontally. In the latter case, #5 bars at 48" on center or #4 bars at 32" would suffice for an 8" wall. Accordingly, vertical reinforcing for an 8" wall would be a minimum of #5 at 32" or #4 at 16". See Table on page 44.

Maximum reinforcing in masonry stems

When using Allowable Stress Design also there is not a maximum, however, it is generally not practical to exceed #8 bars at 8" on center.

Dowel bars into masonry stems

Footing bars bent up into a masonary wall must extend at least the development length of the bar. Per MSJC this is 0.0015 $d_b F_{fs}$, which, for F_s = 24,000 psi equals 36 bar diameters. Although arguable, this length cannot be reduced by the ratio of actual stress in the bar to its allowable stress.

Stress increases for Allowable Stress Design (ASD)

Using the Alternate Basic Load Combination per IBC '09, Section 1605.3.2 a one-third stress is permitted when wind or seismic is combined. This combination also allows a reduction for seismic by 0.7 to convert to ASD.

Concrete Stem Design

Concrete stems should be at least eight inches thick to allow space to place the reinforcing within the forms. The maximum spacing of reinforcing in a concrete wall, both vertical and horizontal is 18" per ACI, but not more than three times the wall thickness.

Strength Design is commonly used for concrete stem and footing design, where all applied loads are factored per ACI requirements: 1.2 for dead load, and 1.6 for earth pressure, wind, and live load. Use 1.4 for fluid pressure (or any well-defined density). Earthquake forces are already factored, therefore the seismic load factor is 1.0. Always check with most recent and applicable code!

For a summary of concrete design formulas see **Appendix B**.

Concrete ultimate compressive strength is usually specified as f_c' = 2,000 or 2,500 psi. For f_c' = 3,000 psi or greater. Nearly all reinforcing is now specified as ASTM A 615-90, Grade 60 (f_y 60,000 psi).

Horizontal temperature reinforcing is required to be at least .002 x the gross cross sectional area of the wall (i.e. the total horizontal reinforcing should be at least equal to .002 times the wall width times its total height). If the wall is over ten inches thick temperature reinforcing is required in each face unless in contact with soil (basement walls). But remember that more horizontal reinforcing decreases visibility of cracks.

Minimum reinforcing in concrete stems

The minimum amount of reinforcing required to ensure a ductile failure is:

$$\text{min} = \frac{200}{f_y} \text{ (= .0033 for } f_y = 60{,}000 \text{ psi)}$$

For example for an 8" wall with d = 5.5", #5 bars at 17" o.c. would be required.

But, if the provided reinforcing is more than one-third greater than required by design, the above minimum can be waived.

Maximum reinforcing in concrete stems

The maximum amount of reinforcing to ensure a ductile failure is 0.75 *x* ρ (rho) balanced:

$$\text{Max} = 0.75\rho \text{ (= 0.019 for } f_c' = 3{,}000 \text{ psi and } f_y = 60{,}000 \text{ psi)}$$

Where p is the ratio As/bd

For an 8" wall, d = 5.5, this would be #5 bars at 3" o.c.

Determining areas of reinforcing required

A handy formula for determining area of reinforcing, using strength design method, for a given M_u is given below (taken from the CRSI Handbook):

$$A_s = \frac{1.7f_c bd}{2f_y} - \frac{1}{2}\sqrt{\frac{2..89(f_c bd)^2}{(f_y)^2} - \frac{6.8f_c bM_u}{\Phi(f_y)^2}}$$

(b and d in inches, f_c and f_y is ksi, and M_u in inch-kips)

For f_c = 3,000 psi, and f_y - 60,000 psi, this formula becomes:

$$A_s = 0.51d - \sqrt{.26d^2 - .0189\ M_u}$$

Reinforcing cover

The cover distance from reinforcing to face of concrete, must be at least 2" when exposed to earth or weather for #6 bars and larger, and 1½" for #5 and smaller. When concrete is placed against earth, such as at the bottom of the footing, or if the wall is placed directly against earth without forming, the minimum cover is 3", however, this is rarely the case.

Development length of reinforcing

The development length formula is given in **Appendix B** (Summary of Design Formulas). Note that development length can be reduced by the stress level in the reinforcing. Development length per se will rarely apply (except for footing heel and toe bar development extensions) but is used to determine lapped bar splice lengths. See Table on page 44.

Laps and splices

Where bars are spliced (lapped) the splices are classified as either Class A, or B. The required lap lengths for each are, respectively, ℓ_d and 1.3 ℓ_d. To qualify as a Class A splice, less than one-half the bars are spliced and A_s provided must be twice A_s required. If more than half the bars are spliced, and A_s required is more than one-half A_s provided, it is a Class B splice, requiring 1.3 ℓ_d. The usual case for retaining wall stems is Class B splices. Note that reduction in lap length for stress level is not permitted per ACI '08, 12.15.1.

See **Appendix D** for development and lap lengths.

Extension of dowels above footing

For low walls, the dowels from the footing need only extend upward for the development strength of the bars, plus 12 bar diameters. However, in nearly all cases the dowels are stopped a certain distance above the footing, where they are lapped with continuing bars of lesser size and/or increased spacing (because of a reduced moment higher up the wall). In these cases it is the lap length, not the development length, that must be met. Lap lengths (discussed above) cannot be reduced by level of stress.

Special inspection requirements for concrete and masonry

Inspection requirements for concrete are in IBC 2009, Table 1704.4.

Inspection requirements for masonry are in IBC 2009, Table 1704.4.5.1 and 1704.5.3.

Note that UBC '97 required a somewhat more stringent Special Inspection if full stresses were used, and this requirement was waived if half-stresses were used. With the demise of UBC this issue is now moot.

9. SOIL BEARING AND STABILITY – CANTILEVERED WALLS

Tabulate 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. With this, you can view an overturning/ resisting summary and check your computations. An example of such a table is shown on Design Example #1 in Chapter 24.

Proportioning Pointers

Here are a few points and guidelines to help you 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 existence.
- If there is a property line on the heel side, try to get at least some heel width for the additional soil weight. Otherwise, you will have a sliding problem requiring a key.
- If you need a key for sliding resistance, try to keep its depth less than about one-fourth the retained height, and not over about two feet.
- If there is a property line on the toe side, the footing may need to be wider because soil pressures are usually greater at the toe.

Overturning Moments

Overturning moments are horizontally applied forces multiplied by the moment arm from the bottom of the footing to their point of application. The primary force causing overturning is the lateral earth pressure against the wall. Because it is a triangular load, its point of applications will be 1/3 the retained height above the bottom of the footing. If the backfill is sloped, the height used to compute over-turning is at the plane of the back of the footing (i.e., where this plane intersects the ground surface). Lateral pressure due to surcharges is a uniform load applied to the back of the wall, therefore its point of application is ½ the height and the moment arm is from that point down to the bottom of the footing.

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

Seismic may also contribute to overturning. This was discussed in Section 7.

Overturning moments are illustrated in **Figure 9-1.**

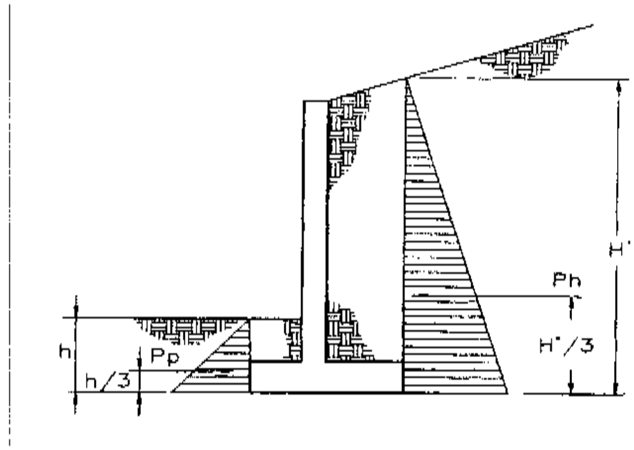


Figure 9-1. ***Overturning Moments for a Cantilevered Retaining Wall***

Resisting Moments

By convention, resisting moments are the sum of all vertical loads about the front edge (toe) of the footing. These forces include the stem weight, footing weight, the weight of the soil behind the wall and over the footing, a surcharge if applicable and any axial 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 edge of the footing. Overturning moments can be visualized as shown on Design Example #1, and on **Figure 9-2**.

The generally accepted factor of safety against overturning is 1.5 to 2.0, although some agencies require more. When seismic is included, a factor of 1.1 is permitted by IBC 2009.

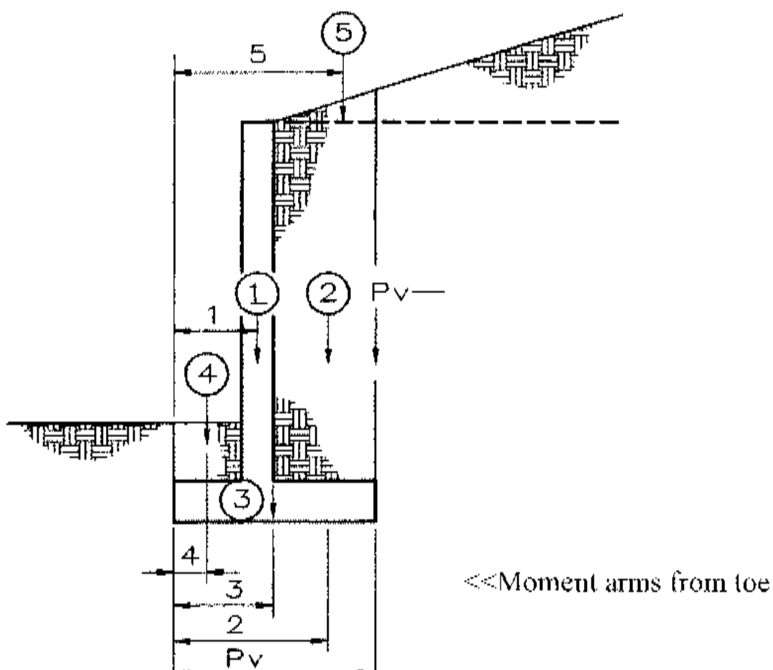


Figure 9-2. ***Resisting Moments***

Vertical Component of Active Pressure From a Sloped Backfill

In the case of a sloped backfill, there is also a vertical component of the lateral pressure resultant, which is assumed to act on a vertical plane at the back of the footing. This vertical component acts to resist overturning. When the wall starts to rotate there will be a frictional resistance along that plane tending to anchor the heel of the wall. This vertical component is also assumed to resist sliding, by adding additional weight to the footing. See **Figure 9-3** (Vertical Component of Active Pressure). See also **Figure 6-1**.

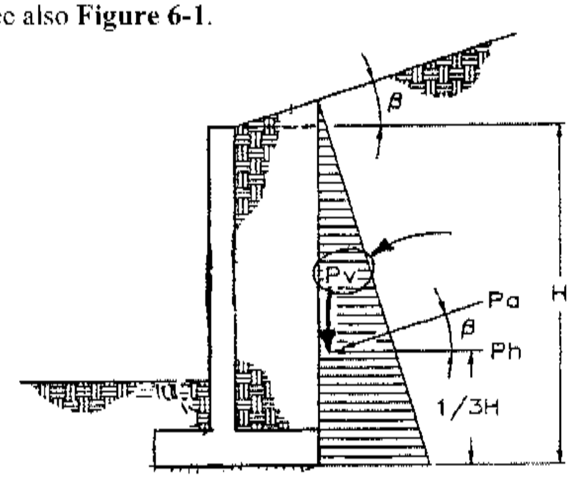


Figure 9-3. ***Vertical Component of Active Pressure***

There is, however, controversy over whether to use this vertical component for soil pressure calculations because its use significantly reduces soil bearing pressures. Most texts recommend using the vertical component only to resist overturning not sliding or to reduce soil bearing. However, this decision is left to the engineer.

If the backfill is level, the Coulomb formula, which assumes the line of action is the friction angle against the stem face, results in a vertical component equal to
P(sin δ). Typically, this results in a vertical component of about 30% of the horizontal pressure.

Determining Soil Bearing Pressure

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

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

$$\bar{x} = \frac{M_{resting} - M_{overturning}}{W}$$

W = Total vertical load

Then the eccentricity is the difference between this distance and half the footing width.

$$e = \frac{\text{ftg width}}{2} - \bar{x}$$

The eccentricity must be less than one-sixth of the footing width (that is, within the middle third) for the footing to be in theoretical contact with the soil for its full width. If this is the case, the soil pressures at toe and heel can be computed as shown in the following formula:

$$\text{Soil pressure} = \frac{W}{d} \pm \frac{6We}{d^2}$$

W = Total vertical load

d = Width of footing

e = Eccentricity

If the resultant is outside the middle third, because soil cannot sustain "tension" between the soil and footing, the triangular pressure diagram shifts to the left and becomes triangular and the resultant moves outside the middle-third. If this condition is allowed, then:

$$\text{Soil Pressure.} = \frac{W}{0.75d - 1.5e}$$

The allowable soil bearing value is usually dictated by the engineer, and usually varies from 1000 psf for poorer soil (or without a substantiating soil investigation) to 4000 psf for dense soil.

Meyerhof Method

An alternate method for determining soil bearing is to assume a rectangular, rather than trapezoidal, pressure distribution under the footing. In this method, often referred to as the Meyerhof Method, assumes a uniform stress block on the toe side. It is similar to Strength Design for concrete. The uniform soil pressure is the total vertical load divided by an assumed width of d – 2e, where d is the footing width, and e is the eccentricity of the total vertical load with respect to the footing centerline. This results in somewhat less toe bending (and easier to compute toe moments and shears!). This method is used, and discussed, in the Segmental Retaining Wall chapter.

Overturning Stability

The generally accepted safety factor against overturning of the wall is 1.5. Some engineers and agencies require 2.0. With seismic, 1.1 is used.

This factor is the ratio of the total resisting moment to the total overturning moment, or:

$$F.S. = \frac{M_{\text{resisting}}}{M_{\text{overturning}}}$$

Sliding Resistance

The sum of all the horizontal forces pushing against the wall must be resisted to prevent a sliding failure. The net driving force causing a wall to slide is the active pressure on the heel side, less active pressure on the toe side. The latter pressure derives from the depth of soil in front of the wall. However, the depth of soil above the toe is usually neglected in the determination of sliding resistance. (Justification: If the depth of soil on the toe side was the same depth as on the heel side, the net driving force would be zero).

The customary minimum safety factor against sliding is 1.5, with some agencies requiring more.

Sliding is resisted by two components:

Friction resistance: This is the resistance of the total vertical weight multiplied by the coefficient of friction between the base of the footing and the supporting soil. The coefficient of friction is usually determined by the engineer, and varies from about 0.25 to up to 0.45. Tests have shown that actual friction coefficients are closer to 0.70.

Passive pressure: Passive pressure is the resistance of the soil at the toe to lateral movement from the active force at the heel section. It is the reverse of active pressure. The wedge of soil in front of the wall must be pushed upward and out of the way for failure to occur. The Rankine or Coulomb formula can be used to compute the passive pressure if the angle of internal friction is know. More commonly, the engineer provides this value. It generally ranges from about 200 pcf to about 350 pcf. It is considered a triangular distribution, zero at the ground surface in front of the wall and maximum at the bottom of the footing or bottom of a key if applicable However, because the soil above the footing, and in front of the toe, is usually loosely placed, its passive pressure is usually neglected, resulting in a trapezoidal passive pressure distribution.

Another theory, suggested by Amrein, increases the passive resistance when a key is added by assuming an increased depth for computing passive resistance. It assumes an additional depth below the footing equal to the average soil bearing pressure divided by the soil density, resulting in considerably greater resistance.

Both frictional resistance and passive pressure can be combined to provide resistance, however, reports often limit the percentage of each which can be used in combination (e.g., 100% friction; 50% passive).

Cohesion resistance: With cohesive (silt and clay) soil, friction resistance is not applicable, and the cohesion (adhesion) between the bottom of the footing and soil provides lateral resistance. If this is applicable, the report will give its value, usually around 100 psf of contact surface.

Footing Keys: If the frictional resistance to sliding plus the passive pressure resistance is not sufficient to give a 1.5 safety factor against sliding, a key can be used or the thickness of the footing increased. A key is a deepening of a near-central part of the footing, usually accomplished by trenching, so that an additional depth of footing is available to further resist sliding by increasing the passive resistance. With a key, the triangular passive pressure distribution extends to the bottom of the key, thereby significantly increasing the passive resistance. Keys usually vary from 12 to 18 inches wide and from 12 to 36 inches or more in depth. See also the above discussion of the Amrein method.

Here's a concern regarding using a very deep key, say a depth greater than half the footing width. Is there then a "paddle-wheel" effect whereby the passive pressure against the key adds to the overturning moment? It's been discussed, and may be valid.

Bending stresses in the key because of passive pressure must be investigated. If the ratio of depth of a key to its width is less than about two, reinforcing is usually not required; the flexural strength of the cross section is sufficient. To compute the flexural stresses in the key, see the example below, Figure R.

Assume $K_p = 2.0$

Allow. passive $= 2.0 \times 120$

$= 240$ pcf

$M_{key} = (1200 - 480) \times 2' \times 1'$

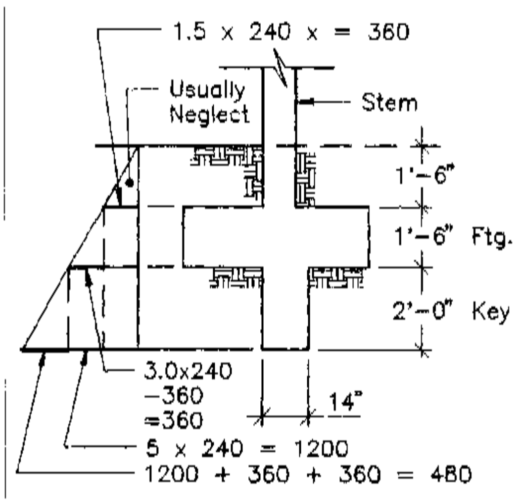
$+ \dfrac{480 \times 2}{2} \times \left(\dfrac{2}{3} \times 2\right) = 2080'^{\#}$

$S = \dfrac{12 \times (14-2)^2}{6} = 288$

$M_u = 1.6 \times 2080 \times 12 = 39{,}936''^{\#}$

$f_r = \dfrac{39{,}936}{288} = 139 \approx 5\phi\sqrt{f'_c}$

$= 5 \times .55 \times \sqrt{2000} = 122$ psi



$$\left[\begin{array}{l}\text{Note that passive force req'd} \\ = \text{total active heel side - passive force toe side} \\ \text{- coef. of friction } x \text{ total vert. load.}\end{array}\right]$$

In above example, total <u>available</u> passive $= \dfrac{240 \times 5^2}{2} - \dfrac{243 \times 1.5^2}{2} = 2730^{\#}$

Figure 9-4. ***Checking Stress in Key***

Deflection (Tilt) of Walls

A cantilevered wall must rotate slightly at the top to mobilize the soil wedge assumed in the design (some texts say .005 times wall height). The horizontal movement at the top is the sum of the deflection of the stem and the rotation of the base of the footing because of soil pressure compression at the toe. By knowing the toe soil pressure and the k value (modulus of subgrade reaction equivalent to modulus of elasticity), the settlement of the toe can be computed, and by geometry, the horizontal movement at the top of the wall. The soil modulus can vary from 200,000 pci to 2,000,000 pci, depending upon soil conditions. This value must be provided by engineer. Tilt (lateral deflection) at top can be given by the formula $\Delta_{top} = \Delta_{soil}$ H/W where Δ_{soil} is the compression of the soil based upon stress and soil modulus, H is the overall height, and W is the width of the footing. The deflection of the stem can also be computed by conventional means (using effective moment of inertia of the stem section). The front face of the wall can be battered for appearance. A rule of thumb might be a batter of 1/200th the height of the wall (e.g. 5/8" for a 10 foot wall).

Global Stability

Global Stability is a term similar "slope stability", whereby an entire soil mass under and encapsulating one or two tiered retaining walls slips in a rotational pattern because of poor shear resistance of a lower layer of soil. With this type of failure the walls remain in tact but the soil mass slips and rotates as a bowl shaped mass.

Slope stability analysis is similar to global stability but the latter includes super imposed "loads" on the slope plane. Slope stability analysis is a vast subject and numerous methods of analysis are in text books.

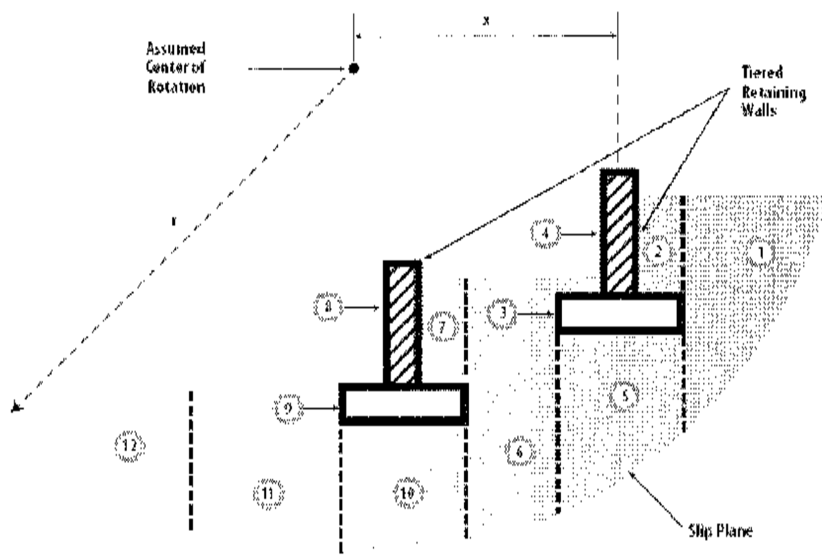


Figure 9-5

One method of analyzing global stability is illustrated in **Figure 9-5.** This is a trial and error method to determine the most critical slip surface (circular) plane which is a function the shear value of the soil. An arbitrary center of rotation is assumed ("0" above) and the rotational force caused by the weight of the components within the soil mass are evaluated by taking their weights x distance from 0, designated + or - in the illustration by summing the moments causing the mass to rotate and comparing this force with the unit resisting shear along the curved lower boundary a factor safety can be determined.

The engineer will determine whether this is an issue based upon analysis of the underlining soils where the proposed retaining walls will be constructed. If there is a potential for global stability failure they will recommend remedial measures such as deepening the footings or otherwise repositioning or reconfiguring the walls.

10. FOOTING DESIGN

Basics of Footing Design

Use the Strength Design (SD) Method to design retaining wall footings. Strength design requires the soil pressure to be factored to compute shears and moments. See the Design Examples for procedure. Both the toe and the heel of the footing are subjected to bending and shear forces. The critical section for bending for both toe and heel is at the face of the concrete stem, or in the case of masonry stems the toe moment critical section is at one-quarter of the stem thickness in from the face. These moments are the sum of the upward acting moments from the soil pressure and the downward moment of the weight of soil and footing. Note that by statics the toe or heel moment cannot exceed the stem moment so the latter may control. The critical section for maximum shear, at the toe is at the "d" distance out from the face of the wall, and for the heel it is at the face of the wall.

Footing design based upon strength design requires factoring the upward soil pressure attributable to earth pressure by 1.6, and pressure attributable to the weight of earth or other dead loads be factored by 1.2. Some engineers believe all soil pressure should be factored by 1.6.

Embedment of Stem Reinforcing Into Footing

It is important to extend the stem reinforcing into the footing. That is, the dowels are considered hooked bars and the embedment required is determined by the following formulas (see ACI 318-08, 12.5):

$$l_{hb} = \frac{0.02\, d_b f_y}{\sqrt{f'_c}}(0.7)\left(\frac{A_s\text{required}}{A_s\text{provided}}\right)$$

or $8d_b$ or 6"

where d_b = bar diameter

l_{hb} = required hooked bar embedment

Whether or not the embedment depth can be reduced by the stress level in the reinforcing depends upon the interpretation of ACI 318-08, Section 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 10-1** (Hooked Bar Bend Requirements).

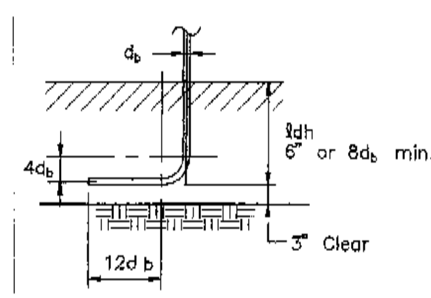


Figure 10-1. ***Hooked Bar Bend Requirements***

If there is a key directly under the stem wall, the vertical stem reinforcing may extend down into the key at least the development length. This will serve the dual purpose of providing key reinforcing, if it is required.

Toe Reinforcing

Reinforcing for the toe generally consists of the dowel bars bending outward toward the toe. If the toe distance is large, over four or five feet, alternate bars may be dropped as the moment decreases toward the toe, but be sure to provide sufficient bending reinforcement at the face of the stem plus development length. Where there is not a toe, such as a property line condition, the stem reinforcing dowels bend back toward the heel.

As stated above, for concrete stems, the critical section for moment is at the face of the stem, and for masonry stems it is one-fourth of the stem thickness
in from the face. In either case, concrete or masonry stems, shear is computed from a distance "d" from the face of the stem.

The allowable shear stress is $0.55\,(f'_c)^{1/2}$.

Check the development length beyond the face of the stem for the toe bars, because the "hooked bar" development length may not be adequate. This required development length will be significantly less than that required for stem bars into the footing, because the "d" distance in the footing is greater than in the stem. See **Figure 11-2** below.

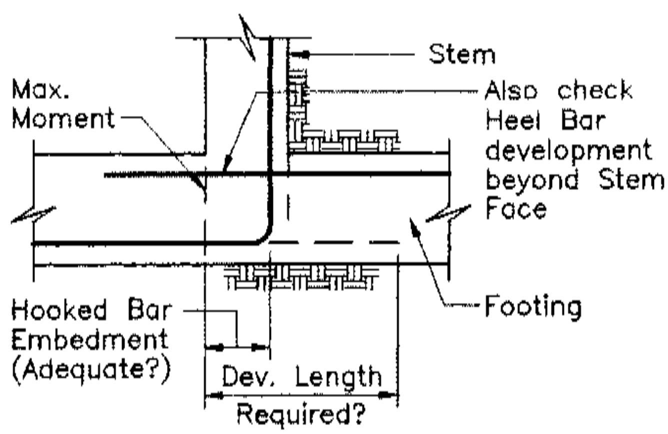


Figure 10-2. ***Development of Toe Reinforcing***

Heel Reinforcing

These bars at the top of the footing resist bending in the heel. The maximum moment occurs, as stated above, at the face of the stem for concrete, and one-fourth the stem thickness in for masonry stems. Shear is computed from the face for both concrete and masonry. These bars must extend a sufficient development length past the face of the stem. These bars are typically positioned 2 inches clear below the top of the footing.

Minimum Footing Thickness

The minimum footing thickness is the sum of the required hooked bar embedment length, plus cover of the reinforcing at the bottom.

Minimum Cover for Footing Reinforcing

The required concrete cover over reinforcing bars at the bottom of the footing is three inches. At the top of the footing it is two inches.

Adequacy of flexural strength

If the toe or heel distance is small, less than the footing thickness, reinforcing may not be required. In this case, the flexural strength of the concrete may be adequate to resist the moments. When computing flexural stresses in unreinforced concrete the footing thickness used to calculate the section modulus must be reduced by two inches (A;I 22.4.8) to allow for possible cracks.

The allowable flexural stress for plain concrete (Strength Design) is:

$$f_r = 5\phi\sqrt{f'_c}$$

where $\phi - 0.60$

See ACI '08, 9.3.5 and 22.5.1.

Although reinforcing may not be <u>theoretically</u> required, its omission is at the discretion of the engineer considering the conditions. Usually, it is wise to provide a "minimum" amount of reinforcing.

Horizontal Temperature and Shrinkage Reinforcing

Stem horizontal temperature and shrinkage reinforcing was discussed in the section on Designing the Stem. There is not a similar code requirement for footings, however, a minimum area ratio of 0.0012 is suggested. Given a 15" thick footing and 3" cover, this would require a #5 bar for each 18" of footing length. A minimum of two horizontal bars (longitudinal) should be provided.

$$p - 0.31 / [(15 \quad 3)\ 18] = 0.0014 > 0.0012$$

11 PIER AND PILE FOUNDATIONS

Piles, Piers, and Caissons

Each of these foundations performs 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 either 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 12. Their supporting capacity is achieved by a combination of lateral surface friction and end bearing. If a masonry retaining wall has spaced pilasters, the pilasters can be cantilevered up from an embedded pier (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 engineer. Conditions which would suggest using piles include poor or compressible underlying soil, the need for greater lateral (sliding) 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 by the wall above. With higher walls a double row of staggered piers is common practice. The staggering provides for greater overturning resistance using small diameter piers. Small implies conventional diameters < 24", as opposed to large diameter piers that might be needed for overturning or high retaining walls.

Design Criteria

Design criteria for piers and piles is usually provided by the engineer because IBC '09 Section 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 pile spacing; driving and installation requirements; testing requirements; and related recommendations; and, site-specific precautions.

To aid the 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 (sliding 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 engineer to observe the aspects of the installation for conformance with the recommendations of the report.

Pile Design

The design requirements for piles are covered in IBC '09 Sections 1808 through 1812.

Sliding stability is an essential consideration for any retaining wall. To resist a lateral force piles may be either battered such that they resist the axial component of the lateral force, or the lateral force can be resisted by bending in vertically aligned piles. In the latter case, the report should provide criteria for designing or checking the piles for bending, such as the depth to contra flexure (maximum moment), passive pressure that can be included, and what lateral deflection under seismic conditions will be tolerable.

Consider possible site clearance problems and consult the installing subcontractor for suitability of your design when using battered piles. Generally, a batter exceeding 4:1 should be avoided. Combining lateral pile bending with battered pile resistance is not recommended – use one or the other.

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

Pile Design Example:

For this example assume the same vertical load and horizontal force as Design Example #1:

Use two rows of piles, space 4 ft. apart, 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} = 4,253 # $P_{vert.}$ = 8,034 #   e (eccentricity from C.L. ftg.) = 1.54 ft.

Convert to 8 ft tributary length: V_{base} = 34,024 #   P = 72,232 #

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 = 34,024 x 1.54 = 52,397 '#

$\therefore P = \frac{72,272}{2} \pm \frac{52,397x2}{4} = 62,335\#$ max.

V to each pile = 34,024 / 2 = 17,012 #

Determine moment, pile size, and reinforcing for bending per criteria in geotechnical report.

Determine required length (penetration) of pile for 62,335 # per curves in geotechnical report. If impractical to resist by bending, use one battered pile on outer row,
V = 33,584 #

Assume batter – 1:3

$\therefore$ Axial load into pile from shear = 34,024 x $(3^2 + 1^2)^{1/2}$ = 35,864 #

Total axial load in pile – 35,864 + 62,335 = 98,199 #

Determine total penetration required for this total axial load. If loads or moments are excessive, reduce pile spacing or use an additional longitudinal row.

Pier Foundations

These are most commonly drilled bores, aligned in a single row under the footing (which serves as a pile cap), and cast-in-place concrete after the reinforcing is placed. The consultant may recommend piers where upper soil is weak, or where space is not available for conventional foundations. Piers are usually spaced from six to twelve feet on center and diameters vary from a minimum required 24" to 36" or more. Spacing and diameter depend upon design requirements for sustaining both vertical and lateral loads.

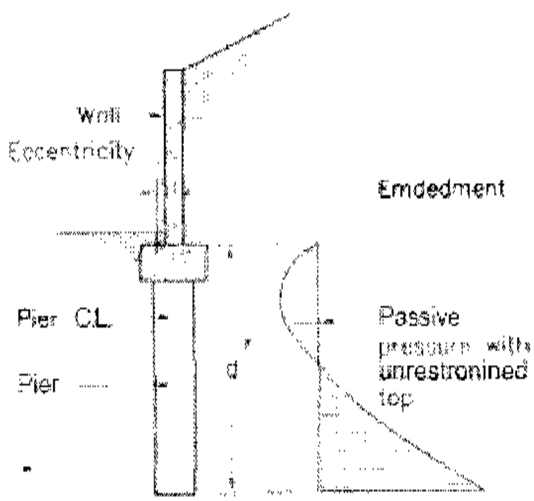


Figure 12-1

The report will give recommendations and design values for end bearing values, skin friction if allowed, and permissible lateral (passive) bearing values. The engineer may allow an increased lateral area for passive resistance, such as 1.5 times the pier diameter. Creep is another factor the investigation may require, which is input as an added lateral force over a given depth of the pier. See discussion under *Soldier Pile Walls*, Chapter 21.

Specific design requirements are covered in IBC '09, Section 1810.

For a design example see Design Example 12, Chapter 24.

An important design consideration is the depth required such that passive pressures are not exceeded. The embedment depth will vary depending upon whether the pier is laterally restrained at the top or unrestrained. The depth required is a function of the pier diameter, allowable passive pressures, and the applied moment and lateral shear.

For the condition where there is not any lateral restraint at the ground surface (such as a slab), the formula per (BC'09, 1807.3.2.1) is:

$d = 0.5A\,[1 + [1 + (4.36h/A)]^{1/2}]$

$A = 2.34P/S_1b$

b = Diameter of round footing or diagonal dimension of square footing.

d = Depth of embedment required, but not over 12 feet for use in the computation of S.

h = Distance in feet from ground surface to applied load

P = Applied lateral force

S_1 = Allowable lateral passive pressure per IBC '09, 1807.3.2, based upon a depth of one-third the depth of embedment, in psf. This value is usually given in the report.

Where a moment, M_g, and shear V_g are applied, such as from a retaining wall with a triangular lateral load, it could be assumed that P could be substituted for V_g and h_1 could be equivalent to M_g/V_g in the above formula.

The solution of this equation requires iteration to determine "d", that is, assume a value for "d", compute S, and solve the equation for "d" iterate until $d_{assumed} = d_{calculated}$, usually three cycles. It is important to note that this equation is an equilibrium statement, that is F /S = 1. Usual practice is to increase "d" by 15 to 20%, or to apply a factor of safety to the lateral pressure or to the passive pressure provided by the engineer. Also, note that "b" can be increased by a factor up to 2 to get the "effective diameter" consult the engineer.

If there is lateral restraint at the ground surface, the formula per IBC 1807.3.2.2:

$d = [4.25(M_g / S_3\,b)]^{1/2}$

M_g = Applied moment at ground surface

S_3 = Same as S_1 above, except the allowable lateral passive pressure per IBC '09 Table 1806.2. This solution also requires iteration for "d".

When the diameter and depth have been determined, the next design task is to design the pier for lateral bending. An alternative to a rigorous analysis for point of contra flexure (maximum moment and zero shear), fixity is often assumed to be one-third the depth of the pier below the ground surface. However, finite-element/spring analysis, observation tests, and practice have commonly reduced this to one-sixth the depth. With this determination the maximum design moment is obtained.

Calculating the moment capacity of a round column with bars in a spiral reinforcing configuration is highly complex, because not only are the bars at varying "d" distances, but also the depth of the traditional Whitney stress block changes with the depth of the circular segment. Help for this

difficulty came from an ASCE *Transactions* paper published in 1942 by Charles Whitney. In it he devised an equivalent rectangular section, thus vastly simplifying the calculations and reportedly with close agreement with a rigorous analysis.

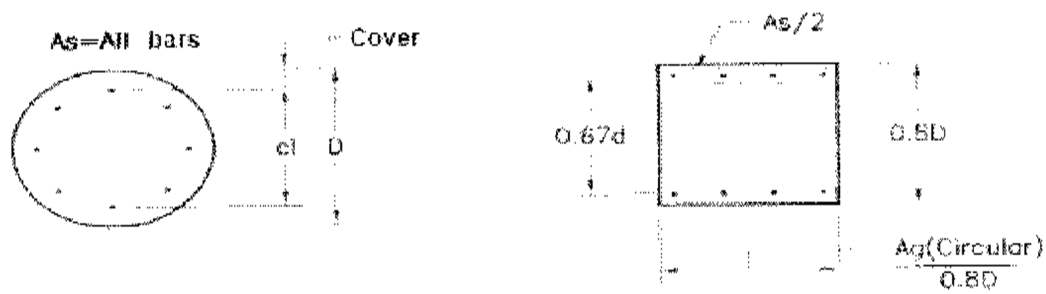
In the Whitney approximation method, assume an equivalent rectangular section with total depth equal to 0.80 times the diameter of the circular column. The width is assumed to be equal to the gross circular column area divided by 0.8 times the diameter. The reinforcing is assumed to be one-half on each face, with the separating distance equal to 2/3 the diameter of the circular configuration. If compression-side reinforcing is neglected, (conservative and easier computation), then the "d" distance for design is assumed to be 0.67 times the circular diameter. This is illustrated in **Figure 12-2**.

Design Example using Whitney approximation method to determine M_n

Assume 30" diameter; 8 - #8 bars; $f_y = 60,000$; $f_c' = 3000$ psi;, clearance = 3"; $\Phi = 0.90$

Gross area of circular column = $\pi\ 30^2 / 4 = 707$ sq. in.

Whitney equivalent rectangular width = 707 / (0.80 x 30) = 29.5"



Circular Pier

Whitney Equivalent

Figure 12-2 – Whitney Approximation Method

Whitney equivalent "d" = 2/3 (30") = 20"

Use ACI equations:

$$a = \frac{A_s f_y}{.85 f_c b} = (8 \text{ x } ½ \text{ x } 0.79 \text{ x } 60,000) / (0.85 \text{ x } 3000 \text{ x } 29.5") = 2.52"$$

$$\Phi M_n = 0.90\ A_s f_y \left(d - \frac{a}{2}\right) = 0.90 \text{ x } 8 \text{ x } ½ \text{ x } 0.79 \text{ x } 60,000\ [\ 20 - (2.52 / 2)] = 3198 \text{ in-kips}$$

Compare this allowable moment with applied moment, assuming applied moment is increased by point of fixity being 1/6 depth of pier below footing cap (assuming no lateral restraint at surface, if applicable).

Allow shear, $\Phi V_n = 0.55 \text{ x } 2 \text{ x } (f_c')^{1/2} \text{ x } A_g = 0.55 \text{ x } 2 \text{ x } (3000)^{1/2} \text{ x } 707$ sq in = 42.6 kips

Axial stress can generally be ignored because it usually is less than 10% of the allowable axial stress. For example, an 8,000 psf end bearing pressure results in only 56 psi, versus an allowable of 0.25 x 3000 = 750 psi. But check if considered significant.

Between piers the footing will be subjected to torsion. Shear is generally not a problem considering the typical footing width (for lateral force) and wall above for vertical shear. Refer to, ACI 318-08, Section 11.5.1 that allows a "threshold torsion" value below which no torsion reinforcing is not required. This equation is:

$\Phi\,(f'_c)^{1/2}\,(A^2_{cp}/P_{cp})$ where A_{cp} is the footing area and P_{cp} is its perimeter.
If this is exceeded, additional torsion reinforcing is required per ACI 11.6.

12 COUNTERFORT RETAINING WALLS

Description

A "counterfort" wall should not be confused with a "buttressed" wall. The two are different. A counterfort wall has a stiffening element on the inside of the wall, within the retained earth. A buttress wall has its stiffening element on the outside exposed side of the wall. The decision to use either a buttress or counterfort depends up site restraints, such as property line locations, and aesthetics.

Proportioning

The spacing between counterforts for economical design should not exceed the height of the wall, and is often 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 three-quarters 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 below as a two-way slab, fixed at the base.
3. Design the toe as a cantilever from the wall.
4. Design the heel as a longitudinal beam between counterforts.
5. Design the counterfort. It will be a tapered tension member.
6. Check the final design for stability, overturning, sliding, and soil pressures.

Designing the wall

The wall is a two-way slab, fixed at the bottom to the footing, and supported (fixed) at each end where it crosses the counterforts. An assumption for vertical moment must be made based on the magnitude of the negative cantilever moment from the footing. One text (*Foundation Engineering Handbook*, Winterkorn/Fang, 1975) suggests (modified): $-M = 0.03\ K_a\ \gamma\ H^3$ which is roughly equivalent to the fixed-end moment with a triangular loading with fixed bottom and laterally supported at top. Therefore, an approximation could be made to design the cantilevered base as 1/6 the moment of a pure full-height cantilever. It is suggested that this negative moment reinforcing (placed on the earth side, of course) be extended up to about one-quarter of the height of the wall, then drop or delete alternate bars.

See **Appendix J** for tables showing moments and reactions for two-way slabs with varying end conditions. These were prepared for the Water Resources Division of the Bureau of Reclamation. Included are tables (they may be difficult to read) for various end conditions, span ratios, and other variables. Using these would give more accurate values; however, the simplified procedures described herein should be adequate for most cases.

There will be some continuity across counterforts, therefore it is suggested the horizontal reinforcing be placed in the center of the wall. Designing such horizontal reinforcing for a lateral pressure at one-half the wall height would seem prudent. Theoretically, the pressure reduces nearer the top, but it is probably practical to use the same horizontal reinforcing full height. Use your judgment to detail the reinforcing because the need for negative vertical reinforcing diminishes near the counterforts, as does horizontal reinforcing near the wall bottom.

Designing the counterfort (or buttress)

The counterforts are generally tapered, flaring from the top – or slightly below the top of the wall for aesthetics – to near or at the edge of the footing heel. The heel dimension will be determined by stability calculations of the counterfort (overturning, soil pressure, and sliding). Counterforts are usually 12 inches thick. The counterfort can be considered to be a vertical tapered beam with tension on the earth side. Its applied lateral load from the retained soil will be a triangular distribution based upon the tributary area between the counterforts. The base moment and shear can be determined, and because the counterfort tapers, the moment and shear lessen higher up the counterfort, hence less reinforcing will be needed. Perhaps check the design at the top of the dowels then use that reinforcing thereon. Dowels from the footing should extend into the counterfort about three feet, therefore at that height the moment should be re-calculated and a lesser amount of reinforcing provided that would continue to the top.

When the moment (M_u) and "d" (effective depth) distance have been determined, the following CRSI equation can be used to determine reinforcing required:

$$A_S = \frac{1.7 f_c bd}{2f_y} - \frac{1}{2}\sqrt{\frac{2..89(f_c bd)^2}{(f_y)^2} - \frac{6.8 f_c bM_u}{\varphi (f_y)^2}}$$

(b and d in inches, f_c and f_y is ksi, and M_u in inch-kips)

For $f_c = 3{,}000$ psi, and f_y - 60,000 psi, this formula becomes:

$$A_s = 0.51d \cdot \sqrt{.26d^2 \quad .0189\ M_u}$$

Designing the heel

The heel can be designed as a longitudinal beam spanning between counterforts, with the appropriate uniform load being the net difference between the downward weight of the soil and concrete in the heel, and the upward soil pressure. This beam can be designed as a continuous beam ($w\ L^2 / 12$) with top reinforcing between counterforts and bottom reinforcing under counterforts. If the moment is not large it may be prudent to place all reinforcing at mid-depth of the heel.

Designing the toe

The toe is designed as a cantilever from the wall, similar to a conventional non-counterfort wall, and the dowels in the stem bend outward toward the edge of the toe.

Stability

Overturning and sliding calculations assume the wall and counterforts act as an integral unit, as if it is a conventional continuous cantilever wall. Include the weight of the counterforts. The overturning and resisting moments are then computed to determine safety factors and soil bearing pressures.

See **Figure 12-1** for forces acting.

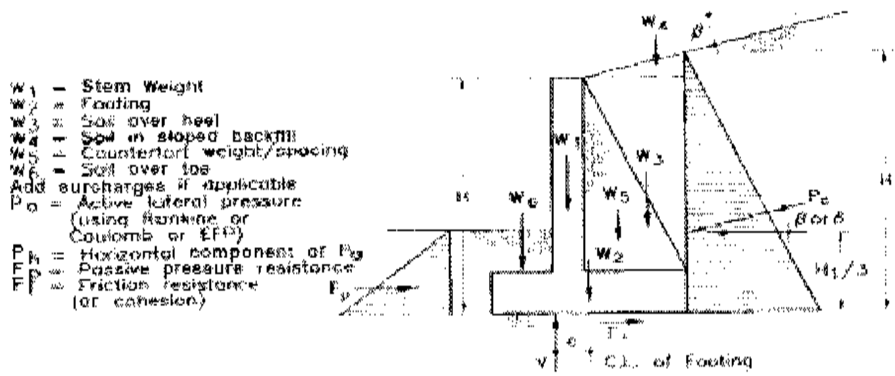


Figure 12-1 - Force Components

Alternatively, a counterfort wall may be constructed of masonry, as shown below:

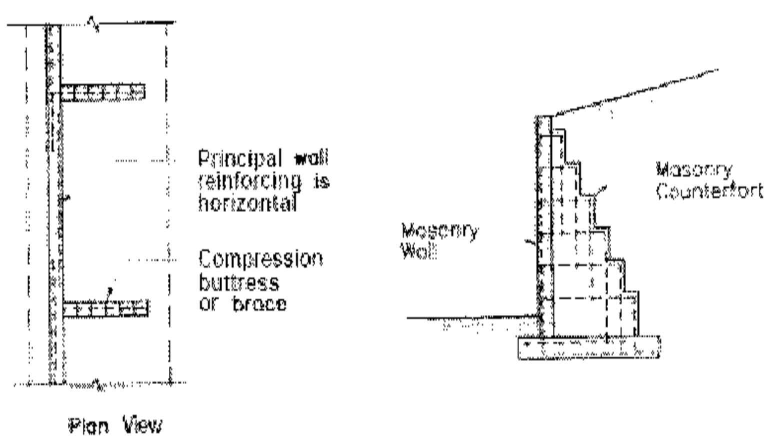


Figure 12-2 – CMU Counterfort Wall

13. 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.

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 for 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" – 4" thick concrete casting slab (later wasted), edge forms are set, a bondbreaker is sprayed on the bed to prevent bonding of the wet concrete to the 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 high as desired.

Unique to using tilt-up panels, 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 when concrete is placed.

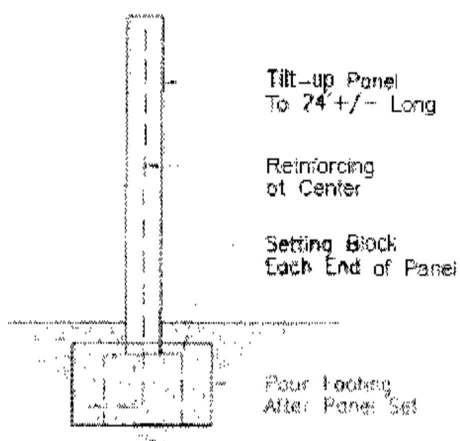


Figure 13-1 – Tilt-Up Freestanding Panel

Design procedure

Design of the wall and foundation are the same as a cast-in-place wall. Just remember when detailing the individual panels to show the dowels projecting the proper distance out of the bottom of the each panel. The temporary setting blocks at each end of a panel, remain in place and integral with the footing as shown in **Figure 13-1.** Check the soil bearing pressure under the setting pads for the panel weight – it's usually reasonable to allow double the allowable bearing pressure for short-term bearing.

A low-slump concrete mix should be specified for the footing to ensure minimum vertical shrinkage which could leave a gap under the wall. Depending on the application, it might be prudent to leave a one inch gap under the panel (cast the concrete short of the wall) for dry-packing a few days after the foundation has been placed.

Free-standing walls

Tilt-up can also be advantageous for free-standing walls provided the length of walls justifies the use of a crane for erection. The vertical reinforcing is best placed in the center of the panel of free-standing walls because these walls are subjected to walls subjected to wind and seismic forces which can occur from either direction.

Foundation design

Design the foundation width, depth, and reinforcing as for a conventional cantilevered wall.

Erecting the panels

This type of wall is relatively low (as opposed to tilt-up panels for a building) so that the panels can be "end picked", meaning inserts are cast into the top edge of the walls, near each end, to which the lifting cables are attached. The crane then lifts (tilts) the panel free from the casting surface and, with the panel hanging plumb, caries it to its final position and lowers it onto the setting pads. Design for lifting stresses and inserts is usually done by a lifting hardware provider. They will check for tensile stress in the panel when it first lifts free (when it is a simple-span beam with bottom resting on casting slab and top supported by lifting hardware). If these concerns are understood, the design for lifting can also be done by the design engineer.

Resources

A Chapter standard reference for tilt-up design and construction is *The Tilt-Up Design & Construction Manual*, published by the Tilt-Up Concrete Association (TCA). Their web site is www.tilt-up.org.

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 2. 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.

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 '09, 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 six 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 Formula (see Chapter 4) should be used because it includes backfill slope, batter of the wall, and the soil friction angle at the wall interface. If the backfill is sloped, you can use a 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 formula with the force diagram in **Figure 14-1(a)**.

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

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

6. 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.

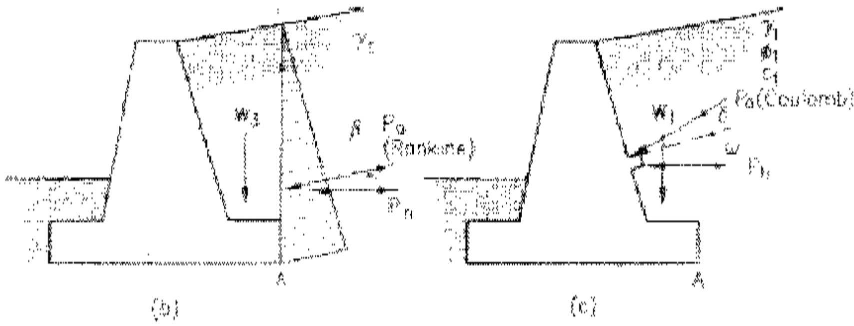


Figure 14-1. ***Force Diagram for Concrete Gravity Wall***

For concrete gravity walls some reinforcing is advisable for crack control. ACI requires $0.002A_{gross}$ minimum horizontal reinforcing for walls.

For example calculations for a gravity retaining wall see Design Example 9, Chapter 24.

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 wire basket walls 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, they 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 the cages. They 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. If the infill is well graded it increases density (weight). 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.

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. They can be laid with the front exposed side flush or with successive blocks stepped back.

For aesthetics, if the front face is flush, it is usually tilted into the soil 6°.

Design Methodology

Since the units are wired together and due to their mass they are considered one cohesive mass for design purposes. They 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. For forces acting on a Gabion wall see **Figure 15-1.**

Density of the Gabion units is usually taken as 120 pcf because soil and vegetation can penetrate the rock intercises and weight can be affected by gradation of the infill.

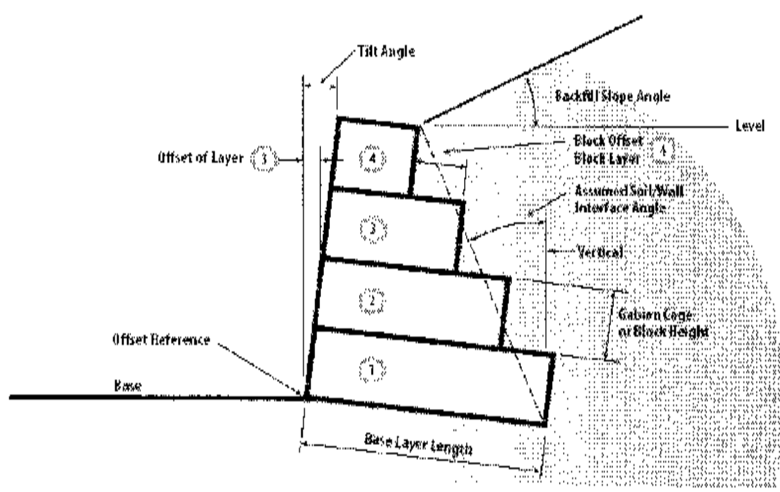


Figure 15-1. ***Forces Acting on Gabion Wall***

Lateral pressures are computed by the Coulomb method shown in **Figure 15-2** below.
A safety factor of 1.5 for overturning is considered adequate.

Sliding resistance is the ratio of total weight of the wall divided by the total lateral thrust. This value should be at least 1.5.

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

β = Angle of backfill slope
ϕ – Angle of internal friction
$\alpha = 90° +$ (tilt angle) – (soil-wall interface angle as shown in Figure 16-1).
δ = Angle of friction between soil and wall (usually assumed to be $2/3\phi$ to $1/2/(\phi)$)

Figure 15-2. ***Coulomb equation***

When the Coulumb equation is used to compute lateral pressure, the α angle to insert is $90° +$ (positive tilt angle) – (assumed soil/wall interface angle per figure 15-1).

The total lateral force, $Pa = K_a * \gamma * H^2$, where H is the vertical retained height adjusted for inclination if applicable. Hence the horizontal component of P_a is cos [($\delta \pm$ (soil-wall interface angle, depending upon back face inclination)] .

Overturning moment resistance is simply the weight of each course multiplied by the distance from the front reference point edge to its center of gravity on a per foot of wall basis. Successive stack courses are added and accumulated to obtain a total overturning resisting moment.

Foundation Pressures

Gabion walls generally do not have a concrete footing but rather are set on a firm level base, often gravel.

To compute soil bearing value from knowing the resisting moment and overturning moment the following equation can used for determining eccentricity from the center of the mass. This eccentricity should be within the middle third of the base width.

From this the soil bearing value is (assuming resultant within the middle third):

$$e = W/2 - (M_r - M_o) / V$$

e – Eccentricity; W = Base width; M_r – Resisting moment; M_o = Overturning moment; V = Vert. load

Resultant is within the middle third of the footing if $e \leq W/6$, then the soil bearing pressure is:

$$\text{Soil Bearing} = V / W + 6*V*e / W^2$$

If resultant is outside the middle third, and since there can be not tension at footing soil interface, the soil bearing becomes:

$$\text{Soil Bearing} = V / (0.75*W - 1.5*e)$$

Sliding

Sliding on the base must also be checked. Sliding safety factor (usually 1.5 or 2.0) = $V * \mu / P_a(\text{horiz})$, where μ is the coefficient of friction at the base-soil interface, usually the range of 0.25 to 0.45.

Seismic Design

Seismic design – if applicable – is similar to the discussion for segmental walls in the next chapter.

Gabion Walls Using Mechanically Stabilized Earth

Although gabion and large block walls can be stacked to accommodate considerable retained heights, conditions may warrant increasing their capability by using horizontal layers of geogrids, or similar mats, embedded between block layers and extending back into the soil to achieve an integral soil mass. Termed *mechanically stabilized earth* (MSE), this concept and the design procedures are discussed in the next chapter on segmental walls.

References

Few textbooks discuss gabion walls. The best source of information is from vendor literature. For example: http://www.gabions.net/technical.html.

Note that although this section deals with "Gabion" walls the same methodology may be used for precast concrete blocks stacked in nearly any configuration.

16. SEGMENTAL RETAINING WALLS (SRWs)

Description

Segmental retaining walls (SRWs), composed of dry-stacked masonry blocks are effective and economical and have gained wide acceptance. These are seen everywhere: leaning against hillsides alongside highways, behind shopping centers, providing tiered grade changes for residential developments, and other applications. Reportedly, 200 million square feet are constructed annually.

Advantages include relatively fast construction; a footing is not needed (just a gravel setting pad) and the units are dry-stacked without mortar, reinforcing, or grouting. The designer has a choice of block sizes, textures, 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 these limitations. For example, if a segmental retaining wall requires geogrids for stability, this requires an available space behind the wall of approximately 70% of the wall height within which to place the geogrid layers. If space is unavailable, a segmental wall is not an option.

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.

For 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. 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 3' 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 blocks are placed. The angle of offset from vertical is termed batter.

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

Most blocks have interior voids which can be infilled with backfill material. Weight per square foot of wall surface is often assumed to be based upon 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".

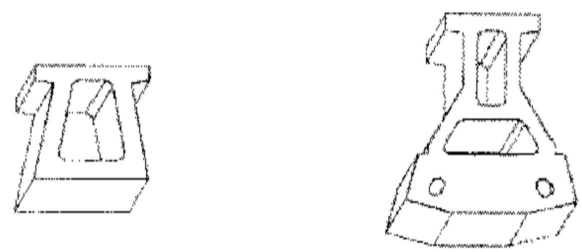


Figure 16-1 - Segmental Block Examples

GRAVITY WALL DESIGN

For stability, segmental gravity walls depend only on their resisting moment exceeding the overturning moment by a factor of safety of at least. 1.5.

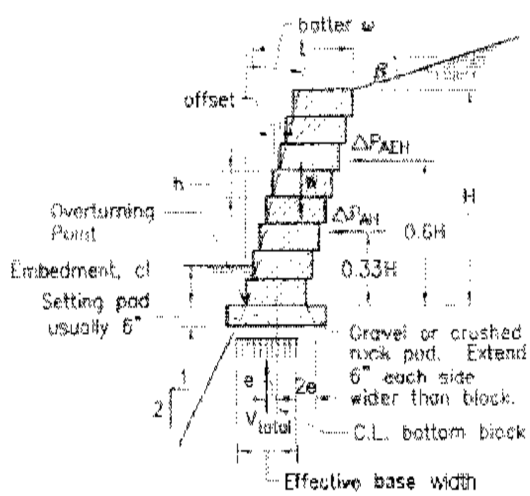


Figure 16-2 – Forces on Gravity SRW Walls

This limits the height of gravity walls to about 4-5-feet, depending upon the batter of the wall and depth of block used.

The design procedure follows these steps:

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

Determine the retained height required and embedment depth below grade. Embedment depth is usually one block course or one foot. Wall height is considered the full retaining height, including the embedment. Determine surcharges if applicable, backfill slope if applicable, and if seismic design is required (see below for seismic design).

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 overturn!

Determine soil properties: density and phi angles for both internal (backfill soil) and external (in-situ, or natural) soil. Backfill should be a well-graded granular soil, for which the phi angle is about 34°. Ideally it would be USCS Group GW (well-graded gravels, gravel-sand mixtures, little or no fines, per Unified Soil Classification System – see Appendix A).

Check Lateral Soil Pressures

Calculate coefficient of active pressure, K_{ah} (horizontal component!). Use the Coulomb equation because it accounts for the friction angle at soil-wall interface and the batter angle. The friction angle is usually assumed to be 2/3 phi (backfill soil). The batter angle is determined by the block-to-block offsets and is equal to $\tan^{-1}$ [(offset per block) / (block height)].

The Coulomb Equation

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

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

β = Angle of backfill slope
ϕ = Angle of internal friction
α = Wall slope angle from horizontal (90° + batter angle from vertical)
δ = Angle of friction between soil and wall
(usually assumed to be $2/3\phi$ to $1/2/\phi$)

Check Inter-Block Shear

The shear at any depth "z" = K_a(horiz) [γz^2 0.50 + (D + L) z]

where γ = backfill soil density and D and/or L = Dead load or Live load.
The maximum interface shear will be at the lowest joint. The shear resistance will be the weight of block above which compresses this joint ("N" value) inserted into the vendor's tested shear resistance equation.

Check Sliding

The total sliding force is the shear at the base of the wall. This is the resistance offered by the coefficient of friction between lowest block and the gravel setting pad, or the friction between the setting pad and in-situ soil below. This is generally given by the formula $R = N \tan \Phi_e$, where R is the resistance available, N is the weight above, and Φ_e is the friction angle of the base (in-situ) soil – often assumed to be 40°. The safety factor against sliding should be at least 1.5.

Check overturning

Overturning moment at any depth "z" = K_a(horiz) [γz^3 0.17 + (D + L) z^2 0.5].

The resisting moment = 0.5 N (t + H * tan ω), where N = weight of block stack,

H = height of wall, t = depth of blocks, ω = wall batter angle.

If more than one wythe, adjust accordingly.

The overturning ratio (resisting moment / overturning moment) should be at least 2.0 per NCMA.

Check soil bearing pressure

For SRW walls the Meyerhof Method is used to determine bearing pressure. This assumes a rectangular pressure distribution under the footing, as opposed to a triangular distribution. The total vertical force is assumed to be distributed uniformly over an effective base width. The effective base width is less than the full width by a distance equal to twice the eccentricity of the imposed load on the full footing width (easily verified with a diagram).

> e = [(base width) / 2] – [[(resisting moment) – (overturning moment)] / (total vertical load)].
>
> B_e = effective bearing width = B – 2e, where B is the total bearing width.

Soil bearing capacity

Ultimate soil baring capacity can be calculated using the classical Terzahi equation:

> $Q_{ultimate} = \gamma\, d\, N_q + 0.5\, \gamma\, B_e\, N_\gamma$ (an additional term to include cohesion is omitted because cohesion is usually assumed zero)
>
> γ = density of underlying (in-situ) soil
>
> d = depth of embedment of bottom block, ft.
>
> B_e = effective bearing width, ft. (see above for methodology)

N_q and N_γ are non-dimensional coefficients per table below for usual range of soil friction angles. For their equations refer to Bowles' *Foundation Design & Analysis, Fifth Edition*, page 220. This reference also gives similar equations by Meyerhof and Hanson.

ϕ_i	N_q	N_γ
31	20.63	26.0
32	23.2	30.2
33	26.1	35.2
34	29.4	41.1
35	33.3	48.0
36	37.8	56.3

Seismic design -- Gravity Walls

Seismic design for segmental gravity walls would rarely be required because of the relatively low height and exemption from most codes. Depending upon the location and local building codes, seismic design may not be required, generally for walls up to 6' height, and in some cases up to 12' height.

However, if seismic is required, two components must be considered: seismic force from earth pressure and seismic force from wall inertia. The former is computed using the modified Coulomb formula below, and the latter uses the k_h factor applied to the wall mass.

K_{AE} = active earth pressure coefficient, static + seismic

$$= \frac{\sin^2(\alpha + \theta - \phi')}{\cos\theta' \sin^2\alpha \sin(\alpha + \theta' + \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta' - \beta)}{\sin(\alpha + \delta + \theta')\sin(\alpha - \beta)}}\right]^2}$$

Where $\theta = \tan^{-1} K_h$, α = wall slope to horiz. (90° for a vertical face), ϕ = angle of internal friction, β = backfill slope, and δ = wall friction angle.

The horizontal component is $K_{AE} \cos\delta$.

Added seismic force = $(K_{AE} - K_a)\,\gamma\, H^2\, 0.5 + (K_{AE} - K_a)\, D\, H + k_h\, w\, H$
The latter term is the inertial weight of the wall.

D = dead load surcharge if applicable; H = height of wall; w = weight of wall psf.

The seismic component is usually designated as ΔK_{AE} (= $K_{AE} - K_A$)

For additional overturning due to seismic the earth component is assumed to act at 0.6 H and the dead load and wall inertial forces at H / 2.

The value of k_h is usually assumed to not exceed 0.15, however, NCMA states k_h should be one-half the peak ground acceleration (PGA). In high seismic areas this results in a seismic force for which most gravity walls could not resist. For example, in my area, Newport Beach, CA, the PGA is about 0.58, which equates to a k_h of 0.29! See Design Example #12 which shows the effect of using just $k_h = 0.05$!

If seismic forces are included, the safety factor for sliding and overturning can be reduced to 1.1.

GEOGRID WALL DESIGN

The soil retaining height of a segmental wall can be increased by placing successive layers of woven synthetic sheets (geogrids) in the backfill as it is being placed, and anchoring each layer into the facing block. This results in a composite mass of "reinforced earth" behind the wall (also called Mechanically Stabilized Earth, MSE) which acts in enmasse to resist overturning and sliding. This enables segmental walls to reach retaining heights of forty feet or more.

Forces acting on a segmental wall with geogrids is shown in **Figure 16-3**. BUT NOTE THAT THE 3RD EDITION OF NCMA's DESIGN MANUAL ALLOWS A RECTANGULAR SEISMIC FORCE DIAGRAM RATHER THAN TRAPEZOIDAL AS SHOW ON THE FIGURE.

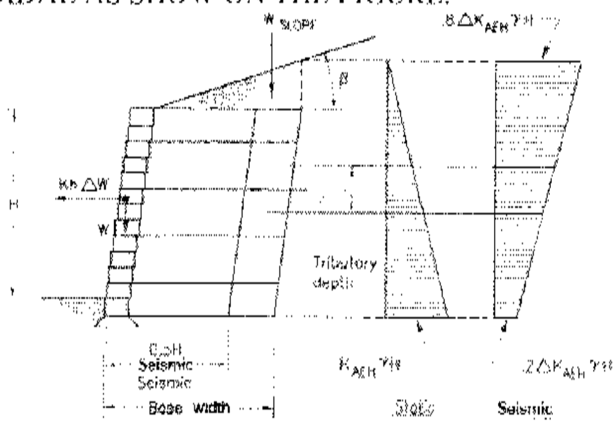


Figure 16-3 – SRW with Geogrids

Construction sequence

Construction begins with an excavation behind the wall extending a distance determined by design, but usually a minimum of 60% of the height of the wall. A gravel or crushed stone leveling pad is used as a base for the masonry units. This base is usually six inches thick and extends a minimum of six inches beyond the inner and outer faces of the blocks. Backfill material should be well graded sand-gravel mix (preferable types GW or SW) compacted to 90% as it is being placed in layers. Care must be taken that the geogrids are not damaged and properly engage the joint between facing blocks, and are of the proper length for embedment beyond the wedge rupture plane.

About Geogrids

Geogrids is the term for the sheet material placed in layers within the backfill. Geogrids are produced by a number of manufactures, each offering a choice of several materials and tensile strengths. The specified geogrid is delivered to the job site in rolls, generally twelve feet wide, and are cut to lengths required by design. Most have bi-axial strength, with the higher strength along the rolled axis (perpendicular to the spool). The geogrid is then cut to the design lengths.

Each manufacturer offers several choices of their geogrids, each with a different strength. Test procedures in accordance with ASTM or Geosynthetic Research Institute (GRI) procedures establish the ultimate tensile strength for each type of geogrid. The long-term design strength (LTDS) is derived from the ultimate tensile strength value and includes safety factors for long-term degradation, allowance for damage during construction, material imperfections, and other strength-affecting factors. A further safety factor is applied for design, generally 1.5. Therefore a Long Term Allowable Design Strength (LTADS) would be LTDS / 1.5.

To be effective in creating the enmasse soil, the geogrid must be anchored at each end: into the in-situ soil beyond the backfill failure plane (described below) and anchorage into the facing block joint to resist the tension in the geogrid. To accomplish this, the geogrids are laid in the joint between blocks. Pullout resistance is both by the coefficient of friction at the joint plus whatever engagement means is used. The latter can be by pins through the geogrid interstices, folding over a lip in the block, or other means proprietary to each block vendor. To establish connection values each block type must be tested for each anticipated geogrid. A typical connection value might be displayed as:

Peak Connection Strength – 425 + 0.27 N, with a maximum of 1900 lbs.
Where 425 is the value (pounds) of the proprietary geogrid engagement to the block; 0.27 is the tangent of the block-geogrid-block friction angle; and N is the weight of the overlying blocks. The generally accepted factor of safety for connections is 1.5.

Another connection value is Serviceability Connection Strength. This is a tested value for failure when the geogrid is pulled to an elongation of ¾". Because this is a failure condition no further safety factor is needed.

The peak connection strength and ¾" serviceability connection strengths are available from the block vendors web site or literature. It is also available from www.icc-es.org which makes available evaluation reports from the various vendors (ICC Evaluation Service, Inc., Legacy Reports).

The factor of safety for peak connection strength is generally 1.5, and factors included should be possible damage during installation, material degradation, creep of the textile, manufacturer, and ratio of ultimate tension capacity and design tension.

Gather design criteria

After determining the site requirements such as retained heights along the length of the wall, plan view alignment (curves?) and contouring that may require a sloped backfill, you will need characteristics of the natural (in-situ) soil both behind and below the reinforced zone. This will be the density of the soil and its angle of internal friction (phi value). This information will be provided by the consultant, and will include other recommendations, such as the need for a global stability check if underlying soil are questionable. Also needed is the density and phi value for the backfill material. Backfill material should be well drained sand/gravel mix, preferably Group GW on the Unified Soil Classification System (see Appendix A).

Determine retained heights, soil properties (densities and friction angles for both in situ ("external") and backfill ("internal") material, loads (surcharges and/or seismic design), and site space available.

NOTE: If slope conditions exist above or below the wall, consult the project geotechnical engineer to determine whether a global stability analysis is required, and if additional geogrids are needed to satisfy global stability requirements.

Select masonry units

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

"Internal" and "External" forces

The term Internal Forces describes the lateral earth pressure within the soil-reinforced zone. This pressure applies force against the wall and creates the tension on the geogrids to maintain the integrity of the soil mass.

External Forces describe the lateral earth pressure acting outside and against the reinforced soil zone.

Determine lateral soil pressures

Both internal and external forces must be considered because the properties of the two soil likely will be different (density and friction angle), therefore the K_a factor needs to be computed for each soil. The Coulomb equation is generally used for both.

The Coulomb Equation is shown below, and note that because the resultant is assumed to act at an angle δ from the horizontal, the horizontal component must be computed. The vertical component is generally ignored.

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

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

β = Angle of backfill slope

ϕ = Angle of internal friction of either backfill or in-situ soil.

α = Wall slope angle from horizontal (90° + batter angle from vertical)

δ = Angle of friction between soil and wall

(usually assumed to be $2/3\phi$ to $1/2/\phi$) but for the external force applied to the reinforced zone δ is assumed equal to Φ.

Select geogrid

The geogrid manufacturer and type is selected based upon the tension resistance required. This is based upon the depth of the geogrid and the vertical tributary area between geogrid layers. The lowest geogrid is usually placed in the first block joint about the base, then every second or third joint, but generally not exceeding two feet apart. Spacing between layers may vary with design requirements, but to simplify instructions to the contractor equal spacing is often used.

When a trial spacing is selected, the internal earth pressure to each geogrid is calculated. This is the force which must be anchored both to the block facing and embedded into the backfill soil a sufficient distance beyond the failure plane.

As described above, the anchorage to the wall is composed of both friction and mechanical devices or other means to further secure the geogrids to the block. This information is available from ICC Evaluation reports or directly from the geogrid vendor.

Check required geogrid length for embedment beyond the failure plane. This is calculated from frictional resistance along the geogrid, based upon the weight of overlying soil, the nature of the geogrid surface, and awareness that it has two surfaces on which to develop pull-out resistance.
Tension in each layer of geogrid, T_a, increases with its depth in the backfill, and can be computed by the equation below:

$$T_a = K_{ahi}\, z\, \lambda\, s$$

where K_{ahi} = horiz component of K_a based upon K_a of internal (backfill) soil; λ = soil density; z = depth of soil above layer, and s = tributary height to the layer.

s = [(height to layer above) (height to layer below)] / 2
(Note: an exception is the tributary height above the uppermost layer and below the lowest layer)

This value, T_a, would be the horizontal tension on the geotextile or geogrid per longitudinal foot of wall, and from this value the geogrid type (LTADS) is selected.

Determine geogrid embedment

To anchor the outer ends of the geogrid it must extend beyond the soil wedge failure plane a distance adequate to resist pullout. The resistance to pullout is provided by the friction between the soil and geogrid (both top and bottom surfaces used), the weight of the overlying soil, and the friction angle of the soil. Additionally, a reduction intereaction coefficient, C_i, is used which is dependent upon the particular geogrid and the surrounding soil. The C_i value usually varies from about 0.70 to 0.90.

The equation for the required embedment beyond the rupture line, neglecting DL, LL, and additional soil in a backfill slope, is:

$$L_e = \frac{F_s\ K_{ahi}\ S}{2 \tan \phi_i\ C_i}$$

F_s = Factor of safety (1.5 minimum)
K_{aih} = Coefficient of active pressure (horizontal component, internal soil).
S = Tributary height between next higher and lower geogrids.
Φ_i = Friction angle of internal soil
C_i = Soil/geogrid interaction coefficient

Note that the above equation is independent of the overlay depth z because this factor cancels out in the equilibrium equation:

$$T_a = K_{aih}\, z\, \lambda\, s = 2\, L_e\, \gamma\, z \tan \Phi_i$$

To include dead load, live load (not recommended) and additional soil over the backfill slope, use the complete equation:

$$L_e = \frac{F_s\ T_a}{2 \tan \phi_i\ C_i \left[\left[D + \gamma\ \ z + \tan \beta \left(\frac{H}{\tan \alpha} - H \tan \omega \right) \right]\right]}$$ D = dead load surcharge

The overall length of one geogrid as required is $L_a + L_e$, where L_a is the length within the soil wedge plus the wall thickness, and L_e is the embedment anchorage length beyond the failure plane. NCMA recommends extending an additional one foot; AASHTO an additional three feet.

Note that this base width is only the minimum for geogrid embedment and additional width may be required for overturning and sliding resistance as discussed below.

Determine depth of reinforced soil (total base width)

The base width is defined as the depth of the reinforced soil (to the end of the geogrids) plus the wall thickness. Although this is initially estimated from 60% to 80% of the wall height, it must be checked.

The criterion is the failure plane angle which extends upward from the base of the wall and defines the limit beyond which the geogrid must extend for proper embedment.
This angle, measured from horizontal, can either be the Rankine failure angle ($45° + \Phi_i/2$) or the more commonly used Coulomb failure plane angle, recommended by NCMA. This angle is:

$$\alpha = \phi + \tan^{-1}\left[\frac{-\tan(\phi-\beta) + \sqrt{\tan(\phi-\beta)\left[\tan(\phi-\beta)+\cot(\phi+\infty)\right]\left[1+\tan(\delta-\infty)\cot(\phi+\infty)\right]}}{1+\tan(\delta-\infty)\left[\tan(\phi-\beta)+\cot(\phi+\infty)\right]}\right]$$

α = Coulomb failure plane angle measured from horizontal.
Φ = angle if internal friction of the internal (backfill) soil)
β = backfill slope, if applicable
δ = friction angle at wall-soil interface (usually $2/3\Phi$)
ω = wall batter measured from horizontal.

The Coulomb line is steeper than the Rankine for most cases; hence it requires a lesser embedment length.

For a given failure angle α, the distance from outside face of wall to failure line intercept, L_a, for any height h_x is:

$$L_a = \frac{h_x}{\tan\alpha} - h_x \tan\omega + t \qquad t = \text{wall thickness}$$

The quickest way to check the minimum required base width for geogrid embedment is to check the uppermost layer of $L_a + L_e$ then add the front batter of the wall to height h_x which is $h_x * \tan\omega$.

To determine the available embedment depth for any height h_x:

$$L_e\,(\text{available}) = B - t - \frac{h_x}{\tan\alpha}$$

When a base width is determined based upon required geogrid embedments, it may not be great enough for the reinforced soil block to resist overturning, which will be checked below.

Check overturning

When considering overturning for an MSE wall the entire reinforced soil zone is considered one mass, therefore the overturning force is the lateral pressure against the end of the reinforced zone – the extremity of the base width. For overturning calculations this is assumed to be a vertical plane (even if the reinforced mass is considered trapezoidal). If there is a sloping backfill, the pressure is against the full height of the vertical plane – from base to intercept with the finished grade.

The Coulomb equation is used, with the density and phi values being those of the in-situ soil, and interface friction angle, δ, is assumed equal to Φ.

If a surcharge is present it is to be included -- both dead load and live load (but not live load if seismic is included (see Seismic Design below).

For overturning, the earth pressure is assumed to act at one-third the total height of the vertical plane, and surcharges at one-half the total height.

Therefore, the total overturning moment is:

$$OTM = K_a(\text{horiz}) * \gamma * \Phi * H^3 * 1/6 + K_a(\text{horiz}) * (DL + LL) * H^2 * \tfrac{1}{2}$$

To calculate resisting moment the weights used are the reinforced soil zone, weight of soil in sloping backfill if applicable, weight of the wall facing blocks, and surcharges if applicable.

Resisting moments are taken about the outer edge of the base of the wall. If the wall is battered, this will increase the moment-arm distances.

Therefore, the total <u>resisting moment</u>, RM, is:

$$RM = w*H*(0.5t + 0.5 \tan \omega * H) + (B - t)\gamma*H[0.5(B - t) + 0.5H \tan \omega] + (B - t)(D + L)\,[0.5(B - t) + 0.5H \tan \omega] + 0.5\gamma(B - t)^2 \tan \beta$$

w = weight of wall, psf; γ = density of backfill soil; Φ = friction angle of backfill soil; B = total base width; t = wall thickness; β = backfill slope;

ω = wall batter angle from vertical; H = height of wall, ft;

D and L = dead load and live load.

The stability ratio (factor of safety) is: RM / OTM, which should be 1.5 or greater per NCMA.

Check sliding at lowest geogrid layer

Compute lateral force same as above with $z = (H - h_x)$ h_x = height to lowest layer

Resistance is provided by both soil friction at lowest layer plus block-joint-geogrid interface value which is obtained from equation provided by block vendor, taking the form (1500 + 0.28N), in pounds, where N = weight of wall above, or:

$$\text{Resistance} = W_{earth} \tan \Phi_e + (XXXX + 0.XX\,N)$$

Check sliding at base

For overturning/resisting, use the same driving force as above for overturning:

$$\text{Sliding force} = K_a(\text{horiz})\,[\gamma * \Phi_e * H_1^2 * 0.5 + (DL) * H_1\,]$$

Note that H_1 is the total height at the back of the reinforced zone from base to intercept with the sloped backfill surface. Therefore, $H_1 = H + (B - t) \tan \beta$. Sliding resistance is provided by friction between weight of reinforced soil mass plus the weight of the wall.

$$W_{total} = W_{wall} + W_{earth} = wH + \gamma\,[(B - t)H + 0.5(B - t)^2 \tan \beta$$

$$\text{Friction Resistance} = W_{total} \tan \Phi_e\, C_i$$

Sliding Safety Factor = Friction Resistance / Sliding Force

Check soil bearing pressure

For SRW walls the Meyerhof Method is used to determine bearing pressure. This assumes a rectangular pressure distribution under the footing, as opposed to a triangular distribution. The total vertical force is distributed uniformly over an effective base width. The effective base width is less than the full width by a distance equal to twice the eccentricity of the imposed load on the full footing width (easily verified with a diagram).

e = [(resisting moment) – (overturning moment)] / (total vertical load)

B_e = effective bearing width = $B - 2e$, where B is the total bearing width.

Bearing pressure = W_{total} / B_e

Soil bearing capacity

Ultimate bearing capacity is calculated using the classical Terzaghi equation:

$Q_{ultimate} = \gamma\, d\, N_q + 0.5\, \gamma\, B_e\, N_\gamma$ (an additional term to include cohesion is omitted because cohesion is usually assumed zero)

γ = density of underlying (in-situ) soil

d = depth of embedment of bottom block, ft.

B_e = effective bearing width, ft. (see above for methodology)

N_q and N_γ are non-dimensional coefficients per table below. For these equations refer to Bowles' *Foundation Design & Analysis, Fifth Edition*, page 220.

ϕ_i	N_q	N_γ
31	20.63	26.0
32	23.2	30.2
33	26.1	35.2
34	29.4	41.1
35	33.3	48.0
36	37.8	56.3

Minimum safety facor for soil bearing per NCMA is 2.0

Seismic design – MSE Walls

If seismic design is required for your locale or applicable code, the first step in the design is to determine the seismic acceleration factor, k_h, which is a function of the Peak Ground Acceleration (PGA) for the side. The PGA can be determined from seismic hazard maps in IBC or from ASCE-7 '05. This can also be obtained by entering the zip code at http://earthquake.usgs.gov/research/hazmaps/design/index.php.

Enter the five digit site zip code and select short term structure (0.2 sec) and 2% probability of exceedance in 50 years.

NCMA recommends $k_h = A / 2$ for internal stability (Wall and reinforced zone) and
$K_h = (A - 1.45)\, A$ for external stability (acting on reinforced zone). A = Peak Ground Acceleration.

In areas of high seismicity, however, the above can yield improbably high design accelerations. For example, for PGA – 0.40, k_h = 0.42 or external stability, results in unreasonably high seismic forces. Consequently, in these areas it is common practice among engineers to use a maximum value of k_h = 0.15 based upon slope stability analogy. Furthermore, NCMA states that "*In practice, the final choice of k_h in any calculation may be based upon local experience, and/or prescribed by local building official or other regulations.*"

For insertion into the Mononobe-Okabe (Modified Coulomb) equation for K_{AE} you must convert k_h to an angle θ: $\theta = \tan^{-1} k_h$

If seismic is required, three components must be considered for overturning and sliding stability:

1. Seismic inertial force from wall, F_1.

   The wall internal force is: k_h w H, where w = unit weight of wall in psf, and H = height of wall. This force acts at one-half the wall h

2. Seismic inertial force from earth pressure within the reinforced zone, F_2.

   For this inertial force a depth of reinforced zone need not exceed one-half the height of the wall.

   Therefore $F2 = k_h \gamma [(0.5H - t) H + 0.5 (0.5H - t)^2 \tan\beta]$
3. Seismic force acting on the back of the reinforced zone, F_3.

   This component is applied to a vertical plane at the back face of the reinforced zone, using a height increased by sloped backfill if applicable. The force may be reduced 50%.

   For this force use the Mononobe-Okabe (modified Coulomb) equation below. The value, K_{AE} is for both static and seismic, therefore you will need to deduct K_A (static) to determine the increased force because of seismic, designated ΔK_{AE}

   K_{AE} = active earth pressure coefficient, static + seismic

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

   Where $\theta = \tan^{-1} K_h$, α = wall slope clockwise from horizontal, (90° for a vertical face), ϕ = angle of internal friction, β – backfill slope, and δ = wall friction angle.

   The horizontal component is $K_{AE} \cos\delta = K_{AEH}$

   For this case α = 90° and $\delta = \Phi_{external}$

   Thus: $F_3 = (K_{AE} - K_a)\gamma H_1^2\, 0.5 + (K_{AE} - K_a) D H + k_h w H_1$

   γ = density of soil, back fill or in-situ depending upon case. D = Dead load surcharge. Note that the value H for this force is the wall height + added height because of sloped backfill, hence:

   $H_1 = H + (B - t) \tan\beta$

These three seismic components must be added to static sliding and to increase overturning moment.

Increase in sliding force = $F_1 + F_2 + F_3$
Increased overturning = F_1 (H/2) + F_2 (H + 0.5H tan β) 0.5 + F_3 0.6 H_1

If seismic forces are included, the safety factor for sliding and overturning can be reduced to 1.1.

Added seismic tension to a layer is calculated by:

$$K_h [(h_+ - h_-) / 2] w + \Delta K_{AEH} [(h_+ - h_-) / 2] [0.8 - 0.6 ([(h_+ - h_-) / H]$$

where h_+ and h_- are heights of next higher and next lower layers.

Before starting with a seismic design for a SRW always check with the building department or agency having jurisdiction to verify applicability and any specific requirements.

Building codes & standards

Neither IBC '09, or CBC '07 directly address segmental retaining walls.

The current standard design references, both published by the national Concrete Masonry Association (NCMA www.ncma.org), are:

Design Manual for Segmental Retaining Walls, 3rd. Edition (NCMA)

Segmental Retaining Walls – Seismic Design Manual, 1st Edition (NCMA)

Acceptance Criteria for Segmental Retaining Walls published by ICC Evaluation Services can be obtained from www.icc-es.org.

Major SRW vendors also offer design handbooks plus other resources, most downloadable in pdf from their web sites.

Also see: Bowles' *Foundation Analysis & Design, 5th. Edition*, Chapter 12.

Getting help

In addition to the references above, all major SRW block vendors have web sites and offer technical support for their products. Some offer free software. Two major vendors are Keystone Retaining Wall Systems and Allan Block. Others can be found through a Google search for "segmental retaining walls".

NCMA offers software for SRW design. Their web site is www.ncma.org.

Retain Pro Software (latest version Retain Pro 9) also includes the design of segmental retaining walls, both gravity and with geogrids. For information: www.retainpro.com.

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 whereby the material is mixed in a hopper before exiting the nozzle, whereby 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, 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 wall 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 pattern is 12" on center each way. Under slab drainage is recommended on sites with expansive soil and special reinforcement and/or thickened slab required for sites with expansive soil 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 vertical segment, 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 accurate results for design. Reinforcing is usually placed in the center of the wall, but for thicker walls it may be to either side of center as required.

The equation for moments at the bottom of any wall segment then becomes:

$$M_y = (K_a \gamma h_y^3)/6 + (K_a w h_x^2)/2 - (W_1 e_1 + W_2 e_2 \ldots..) - (62.4 H_y^3)\ 6$$

Where M_y = moment at depth y; K_a – Rankine or Coulomb active pressure coefficient; h_x = earth height above reference height; γ = soil density, pcf; w – surcharge in psf; W_x – weight of segment (50# for one foot high at 4" thick); e_x – eccentricity of segment x from reference point; and H_y = height of water above reference point. This calculation is performed for each cross section and the critical condition (earth or water pressure controlling) determined. This is best done by a spreadsheet. See **Figure 17-1**.

Before attempting a pool design you may want to check for an engineer specializing in this type of work (for example www.poolengineering.com) – it may be cost effective.

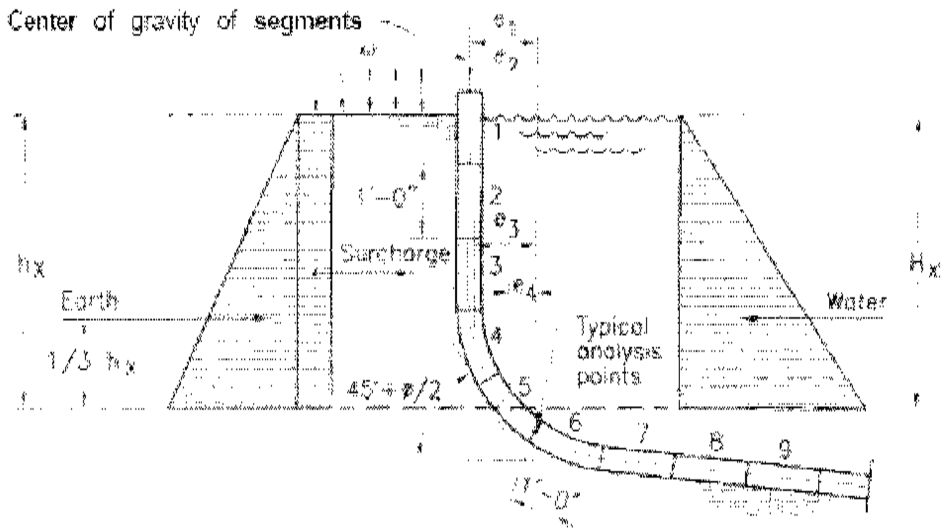


Figure 17-1 ***Analysis of swimming pool wall***

18. PILASTER MASONRY WALLS

Description

As shown in **Figure 18-1**, retaining walls with spaced pilasters and masonry filler 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.

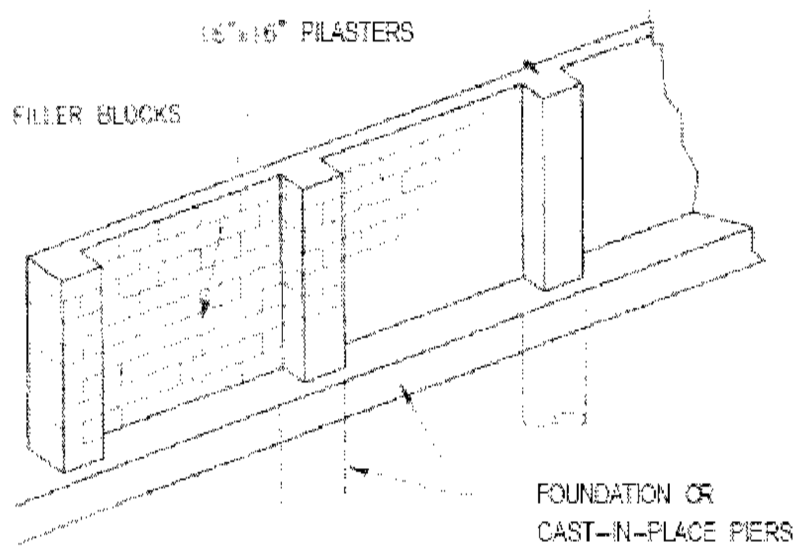


Figure 18-1 ***Pilaster Masonry Wall***

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. Reinforcing is placed in the center of the wall because lateral loads can be from either direction.

If the filler wall retains earth, the lower courses will of course be subjected to higher earth pressures and this controls the thickness of the filler wall. In that case, the reinforcing should be on the outside face between pilaster supports. However, to take advantage of continuity, it may be more economical to place the reinforcing at the center and design for the controlling positive (mid-span) or negative (at pilasters) moments, generally use $w\,(L)^2 / 12$. 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, then select a wall thickness and 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 will be triangular or trapezoidal if retaining earth, and uniform for wind-only or seismic loads. Reinforcing usually consists of four bars and lateral ties.

Alternatively, use only the interior block core and specify high-strength concrete (3,000 psi or greater). This results in nearly the same moment capacity as the full CMU pilaster block using the same reinforcing.

Footing Design

Only a nominal footing is needed under a filler wall. Pilaster footings can be either conventional rectangular spread footings, or cast-in-place piers in drilled holes.

If pilasters are cantilevered from an embedded pier, if not constrained at the surface, the point of contra flexure for moment is below the ground surface. This is often assumed to be one-third the embedment depth. Some engineers and tests suggest a more realistic point of contra flexure is 1/5 to 1/6 below ground surface.

For the design of C.I.P. drilled piers see Chapter 11 – *Pier and Pile Foundations.*

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 in 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 not be sufficient.

Tie-Back, also called Anchored Walls, are another example of restrained non-yielding walls. These walls use drilled and grouted anchors placed into the backfill slope to provide lateral restraint, the design for what can become complex if there are multiple levels of anchors.

Dual Wall Function

Often it is desirable to prepare two designs for the same wall. For example a basement wall may be backfilled before an effective lateral restraint is in place at the top. It can first be designed as a conventional cantilever wall 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 larger moment at the stem base. Then a second design for the final condition when the top restraint is in place and backfill completed. Hence you've covered both conditions.

Note that 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

For a wall restrained at the top against lateral movement the soil wedge will then not mobilize and 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 $\phi = 34°$, $K_o = 0.44$, as opposed to $K_a = 0.28$ (assuming level backfill). For sloping backfill a suggested formula is
$K_o = (1 - \sin \phi)/(1 + \sin \beta)$.

Given 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 7-05 specifies a minimum of 60 pcf for "relatively rigid" walls, and states that basement walls not more than 8 feet below grade and with light roof framing (flexible) are not considered "rigid". You are advised to get design values from the engineer and check applicable code requirements.

An alternate to the triangular lateral pressure distribution, some geotechnical engineers specify a uniform pressure (also applicable for open cut excavations) as shown in **Figure 19-1**. Not that the clipped top and bottom corners can be ignored – a full-height uniform load will give only slightly more conservative wall moments. This uniform pressure, for sandy soil, is often defined as: $0.65\ \gamma\ H \tan^2 (45 \quad \Phi/2)$. Given a level backfill this corresponds to $0.65\ \gamma\ H\ K_a$. This method results in about 25% higher wall moment than an equivalent triangular pressure using the same K_a.

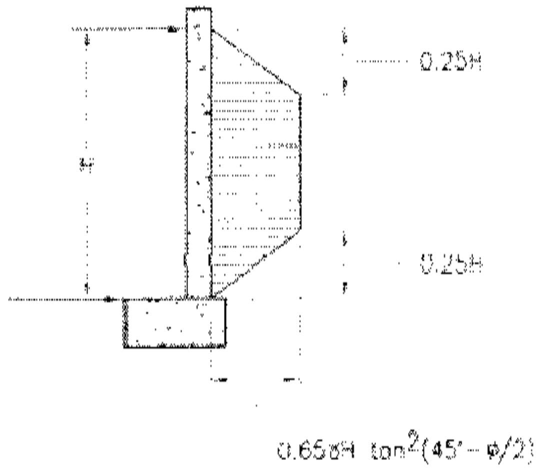


Figure 19-1 – Trapezoidal Soil Pressure

Seismic Force on Non-Yielding (Restrained) Walls

Several texts (e.g. Kramer) propose the following formula (slightly revised):

$\Delta P_{eq} - \gamma\ k_h\ H^2$, acting at a resultant height of about 0.6H

Where ΔP_{eq} is the added lateral seismic force, γ is the unit weight of soil, and H is the retained height.

The resultant acting at 0.6H gives a slightly trapezoidal force diagram, however, for ease of calculation a uniform load can be assumed with less than 2% unconservative error.

It should be noted that there are so few incidents of earthquake damage to such walls that many experts agree that seismic design of restrained (e.g. "basement") walls may not be necessary, particularly given an adequate factor of safety for the service level design.

20. SHEET PILE WALLS

Description

Sheet piles are driven into the ground to retain earth while excavation is done on the opposite side. This can also be permanent retaining structures alongside waterways (bulkheads and quay walls). Most sheet piles are steel, configured in an interlocking Z-shape to increase bending capacity and stresses during driving. Pre-stressed concrete panels are also used for sheet piling.

Sheet piles derive lateral support from embedment into the soil below the base grade, and can either cantilever up from that level, or be laterally restrained near the top by tie-backs in which case a horizontal member must be provided spanning between tiebacks.

Design Procedure

The design considerations for sheet piling involve the following:

1. The embedment into the base soil must be adequate to resist the total lateral thrust. If a footing is planned rather than pile penetration below the base-level, the footing must be designed to provide passive resistance.
2. The bending capacity of the sheet pile material must be checked at the point of maximum moment. The point of contra flexure (zero shear and maximum moment) is usually about one-third down the embedded depth (although some texts state the actual point of contra flexure is closer to 1/6 the embedment depth), thus increasing the design moment. Manufactured sheet piles are designed to resist the driving impact during pile driving.
3. Tie-backs, if used, must extend beyond the line of rupture and a sufficient distance beyond that to mobilize adequate pressure resistance of the anchoring device. Using tiebacks will reduce pile size and depth of embedment.
4. The design of sheet piling is based upon the soil design parameters recommended by the engineer. Input from the sheet pile vendor (most have handbooks and some have software to assist) and involving an experienced sub-contractor are essential.

Waterfront structures must consider impact from docking ships. (Incidentally, in marine work, the outboard bottom soil level, below the water line, is referred to as the "dredge line".)

A generalized force diagram of a sheet pile wall is shown in **Figure 20-1.** Note that by statics the horizontal active pressures and passive resistance must balance. The maximum moment will occur at the point of zero shear (usually about one-third down from the dredge-line, but considerable evidence that 1/6 is adequate) which can be determined by statics. Determining "d" and "D" shown in the illustration can be determined by statics and is an iterative (trial and error) process.

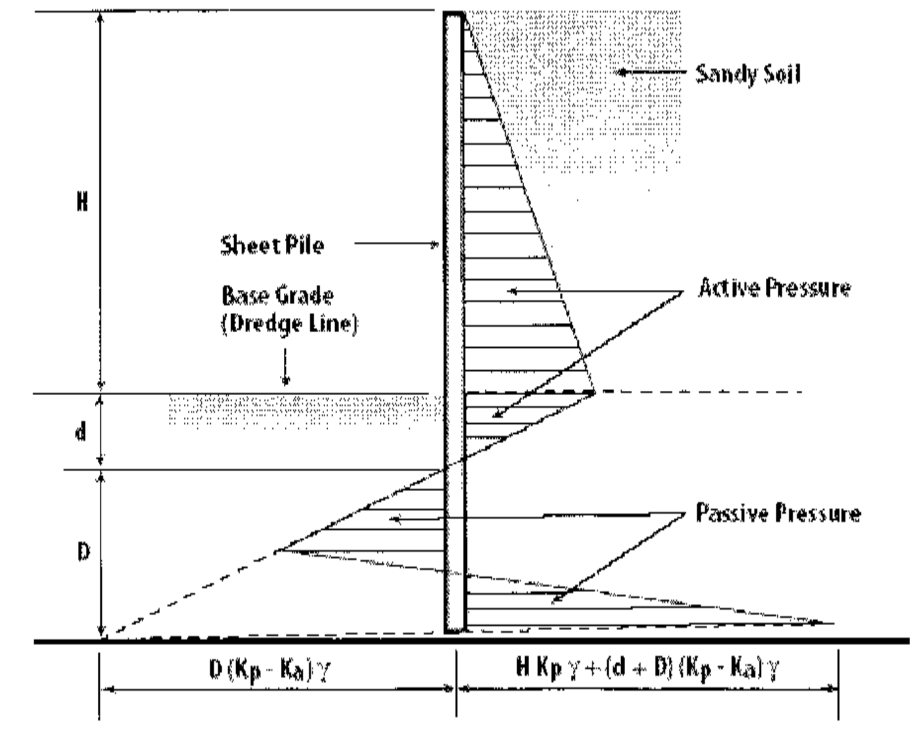


Figure 20-1 ***Cantilevered Sheet Pile Force Diagram – Sandy Soil***

References

The design of sheet pile walls is complex and references should be consulted. One good reference is Das' *Principles of Foundation Engineering, 5th Edition*, Chapter 9. Another: Rayapakse *Pile Design and Construction Guide*, 2003. Teng's *Foundation Design*, Chapter 12, is very good with tables and examples. Contractors specializing in sheet pile installation are the best source for economical design and site-appropriate recommendations and vendors of sheet piling have essential design data.. A Google search for "sheet pile design" will yield valuable sources.

21. SOLDIER BEAM WALLS

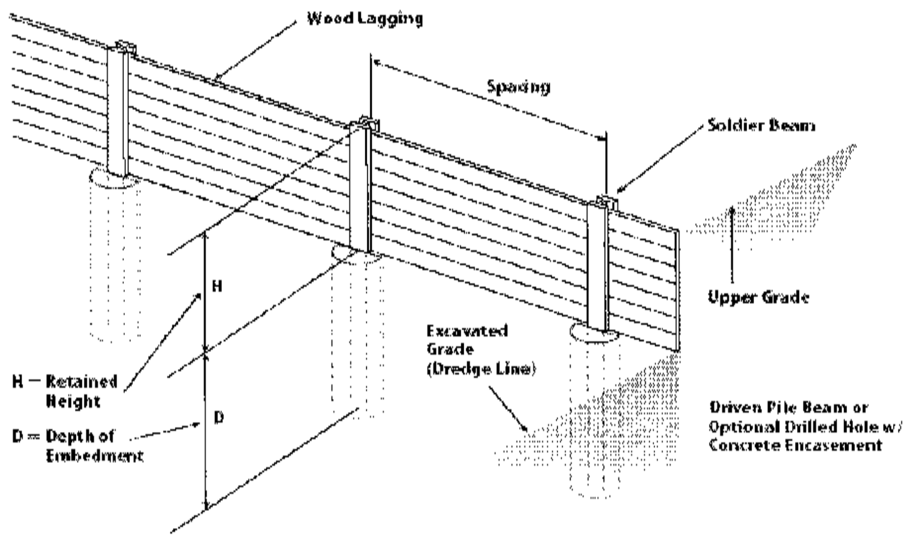


Figure 21-1

Description

Soldier beam retaining walls are used to temporarily retain soil, such as at a construction site. 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 then 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 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 moist 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 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.
Whether to use tiebacks is another decision to be made. The following procedure assumes a cantilevered system in sandy soil.

A basic design requires six steps:

1. Determine the driving forces, that is, forces imposed by any construction surcharges and the active soil pressure tributary to each pile beam. For this use the Rankine equation to calculate K_a. Several designs may be done to optimize the beam spacing based upon lagging selection, embedment depths, and beam sizes.

2. Referring to **Figure 21-1**, after P_a and P_w have been calculated, the depth of embedment must be determined. This will be a function of the allowable passive pressures and arching factor allowed to increase the effective flange width, or hole diameter if pre-drilling is used. The arching factor, f, can be taken as 0.08 * phi, but should not exceed about 2.5. This means that the effective pressure width in front of a 30" diameter drilled and concrete filled beam encasement, with a phi of 32° would be 0.08 * 32 = 2.56, but use 2.5. Thus the effective passive pressure would be a width of 2.0*2.5 = 5.00 feet which will considerably reduce embedment depth and moment applied to the beam.

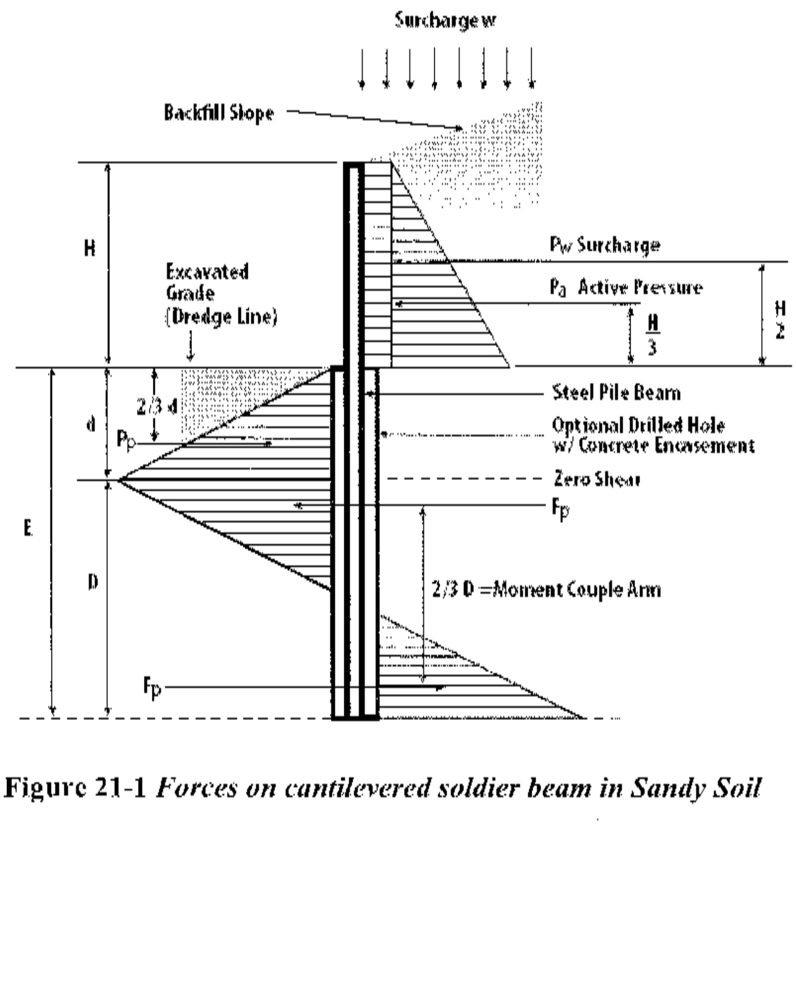


Figure 21-1 ***Forces on cantilevered soldier beam in Sandy Soil***

For determining the embedment depth to zero shear (where beam bending is maximum), designated "d", the following equation can be used:

$$d = \sqrt{\frac{P_P \times SF \times 2.}{p \times DIA \times A}}$$

Where P_p is equal to and counteracting to $P_w + P_a$; SF is the safety factor applied to allowable passive pressure; A is the arching factor multiplier; Dia is the hole diameter or flange width, whichever applicable; and p is the allowable passive pressure in pcf.

3. The maximum beam moment is then determined by summing moments above the point of zero shear. However, mathematically the result is equivalent to:

$$M_{max} = P_w\,(0.50H + 0.67d) + P_a\,(0.33H + 0.67d)$$

4. The maximum moment is resisted by a passive pressure couple consisting of $F_p * 0.67D$. Therefore the required depth D can be determined from the following equation:

$$D = \sqrt{\frac{(\text{max. moment}) \times SF}{(p \times DIA \times A \times d \times 0.25\,)0.67}}$$

The required depth of embedment is then (d + D). As a rule-of-thumb for sandy soils this is usually in the range of 1.3 H to 1.5 H.

5. After the maximum moment has been computed, convert it to LRFD (Load Resistance Factor Design) by multiplying by the usually applicable load factor of 1.6. Then select several beam options from AISC 13th edition, LRFD, Steel Design Handbook. When several beam selections are made it is recommended that you talk with the contractor for his opinion on which is most economical or available.

6. Select the lagging. Treated lumber should be used, and a conservative fiber stress is 900 psi. Calculate the lateral pressure at various depths, H_y,(to determine changing lagging thicknesses) which is $K_a * \gamma * H_y$. When the simple span moment is calculated it is acceptable to multiply by 0.8 because of arching action of the soil between pile beams. It is also customary to limit the active pressure to 40 psf. Lagging is either 3" or 4" by 12" treated wood. These ends should bear against the beam flange a minimum of 3". Allow about 1" between each lagging for drainage

Using tiebacks

If the retained height is over about 15 feet, and space is available, tiebacks can be considered. This use will also reduce embedment depth and beam size.

Usually tiebacks are steel rods inserted into 3" drilled holes into the backfill a sufficient distance beyond the failure plane to provide anchorage after grouting. They are inclined downward at an angle of 15° to improve withdrawal resistance and to facilitate grouting. Their outside ends are welded to the steel beams.
A simplified force diagram is shown in **Figure 21-3**

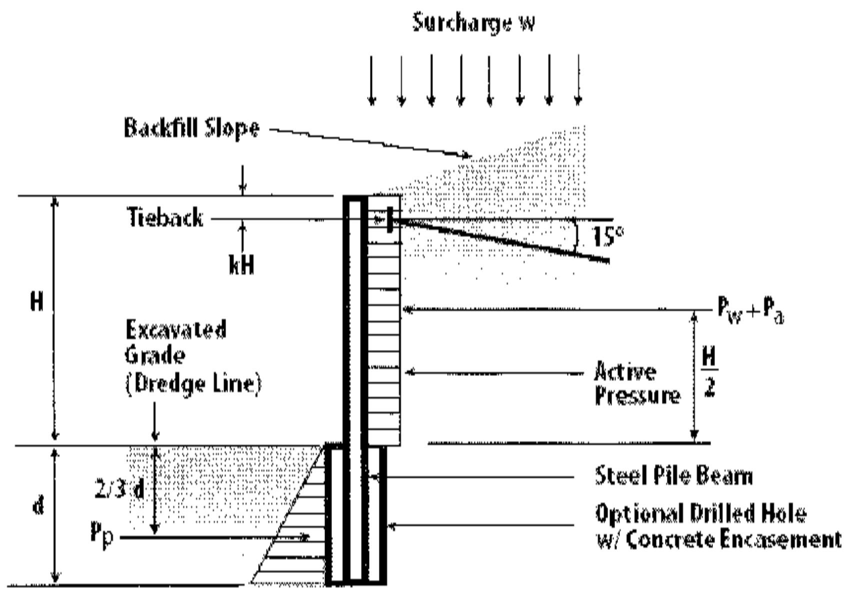


Figure 21-3

The depth of embedment, E, must be sufficient to oppose the applied lateral forces P_a and P_w.

Determining d is an iterative processive to achieve active and passive pressure balance. The maximum bending moment in the beam can then be determined by statics.

An alternate tieback can be anchored beyond the failure plane as shown in **Figure 21-4**.

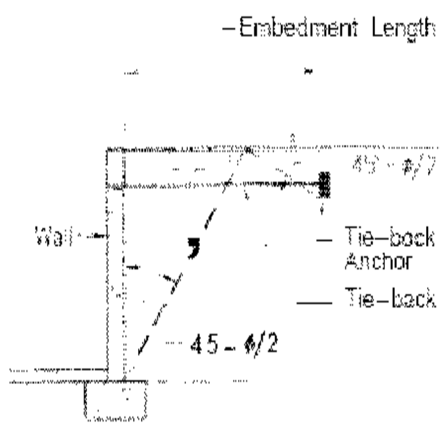
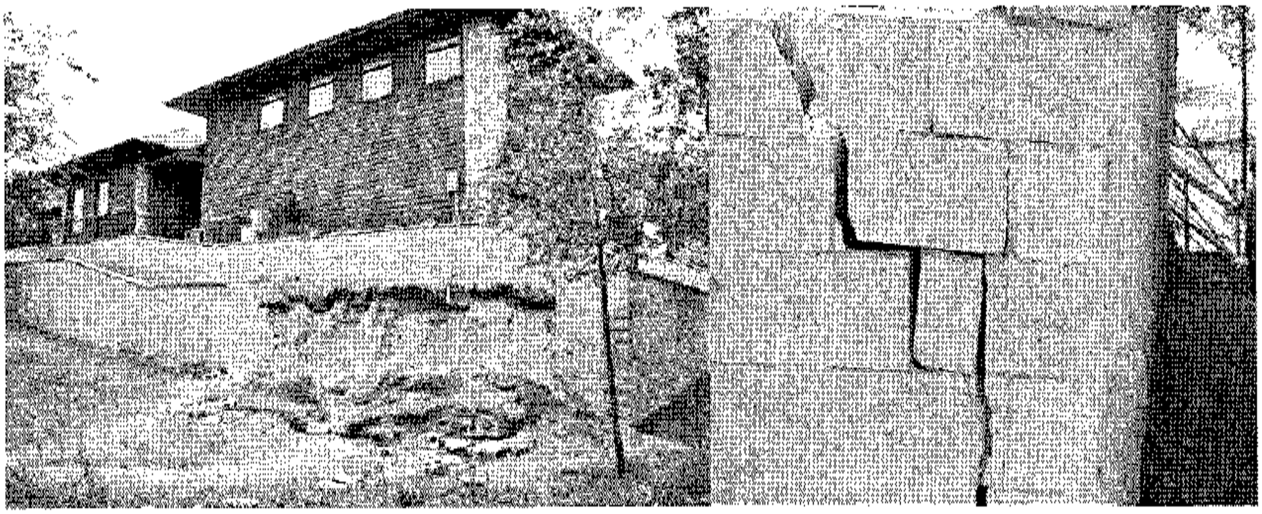


Figure 21-4 – Tie-back Anchorage

22. WHY RETAINING WALLS FAIL & COST EFFECTIVE FIXES



The above photo is a rare occurrence. In this case there was no permit, no engineer, minimal reinforcing ungrouted cells, and other oversights.

"Failure" of a retaining wall does not necessarily mean total collapse, as shown above, but rather signs of impending instability and likelihood of a 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 design a new wall.

Fortunately, retaining walls are quite forgiving, nearly always displaying telltale signs of trouble and 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 1/16th 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.

2. Saturated backfill. Retaining walls are generally designed assuming a well drained granular backfill. If surface drainage is allowed to penetrate and accumulate in the backfill, the pressure against the wall can double. Ponding of water behind the wall not only indicates poor grading, but clayey soil impeding the downward seepage of water. The surface of the backfill should be graded to direct water away from the wall, or by the use of drainage channels adjacent to the wall to intercept surface water and divert it to disposal. Often surface water problems are attributable to a misdirected or poorly timed irrigation system. Poor backfill material, such as containing clay, can swell and increase wall pressure. One contractor always uses crushed rock for backfill; it's less expensive than pea gravel, and the elimination of tamping compaction of granular soil offsets the cost of crushed rock versus the use of materials that require compaction, and assures good drainage. Don't compact backfill by flooding.

3. Weep holes that don't weep. The only thing that comes out of most weep holes is weeds – not water. This becomes clogged when there is no filtering, such as a line of gravel or crushed rock placed along the base to provide a channel for water to drain to weep holes, or to be conducted by an embedded perforated pipe. Commercial filtering fabric is available. Weep holes in masonry are usually made by omitting mortar at the side joints of every other block (32 inches on center). For concrete walls, 3" diameter pipe sleeves are often used, spaced 4' – 6' on center, or as deemed appropriate by the designer. Specifying proper drainage measures (backfill material, surface water control, and base-of-wall drainage) is an important specification task for the EOR (Engineer of Record).

4. Design error because of misinformation Design errors as the cause of failures are relatively rare when prepared by an experienced designer. However, sometimes the designer is given insufficient or erroneous information. For example, "Design the wall to retain eight feet", but later examination of the grading plans, or as-built conditions, shows the wall retaining nine feet, an additional foot, thereby increasing the base moment on the stem by nearly fifty-percent.

5. Calculation errors. An experienced designer can quickly spot a calculation error because it obviously "doesn't look right". New engineers usually lack this experience and in such cases don't let the design leave the office without a check. A 15-minute review could save costly fixes and damage client relations. And don't assume a plan checker will find errors.

6. Unanticipated loads. Again, this is a client-to-designer information problem. Good communication is essential. Is there a surcharge the designer didn't know about? A steeper backfill slope? A beam connection? Wind load? A foundation investigation or memo that wasn't transmitted?

7. Mistakes in using software If software is used as a design aid, it is essential that the designer correctly inputs data and understands the capabilities and limitations of the particular program (Retain Pro advises its users to be licensed civil or structural engineers, or at least have the expertise to design a relatively complex retaining wall by hand calculations). If in doubt of a result, do a quick hand calculation.

8. Detailing errors. The contractor must have clear instructions. Details not conforming to the design, or doubtful of interpretation, must be avoided. Perhaps the biggest source of problems is with reinforcing placement. I recall one case where the designer actually detailed the rebar on the wrong side of the wall! In another case vagueness of details led to dowels from the footing extending only 6" into the stem, rather than the intended 24", because of confusing dimensions. Easy-to-read drawings and careful checking by the designer can eliminate these problems.

9. Foundation problems. When a investigation is provided, there will be guidelines for design (allowable soil bearing, friction factors, seismic if applicable) and any caveats based upon site conditions, such as liquefaction potential or recompaction of the underlying soil. Implementation of such recommendations

should provide a trouble-free foundation. However, often such an investigation is not provided, calling for special care by the designer. Without such a report the soil bearing is limited by code, usually to 1,500 psf, and the coefficient of sliding friction to 0.25, and allowable passive pressure of 150 pcf. Regardless of using more conservative values, the designer should be aware of any adverse conditions, such as fill material, compressible soil, water table, or other factors that could cause excessive settlement or reduce sliding resistance.

10. Inadequate specifications and notes. If you use "boiler plate" notes or specifications, edit these carefully and use a checklist. It's embarrassing to have notes or paragraphs that obviously apply to another project. Here's a note that should keep you out of trouble and avoid problems: "If a discrepancy arises between the drawings and field conditions, or where a detail is doubtful of interpretation or an unanticipated field condition is encountered, the engineer shall be immediately contacted for procedure to be followed. Such instructions shall be confirmed in writing and distributed to all affected parties". And another good one: "Wherever there is a conflict between details and specifications, or between details, or where doubtful of interpretation, the most restrictive shall govern as determined by the Engineer of Record."

11. Shoddy construction. This could be anything from a homeowner having built a wall from a "how to" book, to an inexperienced or unscrupulous contractor building without plans or not following these – inadequate grouting or mortar or improperly placed reinforcing. Retaining walls are quite forgiving and poor construction may not appear as distress for years, or never. I once built a vacation home with a five foot masonry retaining wall intended to be restrained at the top by the floor slab placed on the backfill. I instructed the contractor not to place any fill until the wall was properly braced since there was only a 14 inch wide footing. I went to the site a week later and to my amazement he had placed the backfill the full five feet no floor slab yet – and the wall was perfectly plumb! Another case of practice defying theory – but don't count on it! I had not followed my own rule and advice to others -- to always, when possible, have a pre-construction meeting between the designer and contractor to be sure all conditions and requirements are understood, and jointly review the plans.

12. Age. If a retaining wall has been in place twenty years and shows no sign of distress, the chances are it will remain so for another twenty years, or fifty years. The adage "if it ain't broke, don't fix it" may be prudent advice. However, the caveat is that this precludes any change affecting the wall, such as new surcharges or a change in drainage above the wall. If in a seismic area the chances are it has already successfully withstood several earthquakes, but if the seismic risk is high and its failure could impact another structure, a seismic evaluation would be appropriate

And twelve fixes that could save a wall:

Note that each of the fixes listed below have been successfully used, but it is assumed that the wall is not in such distress that none are viable solutions.

1. Correct surface drainage problems. You can't economically replace the backfill or get to the base-of-wall drainage system, but you can re-grade at the surface so water does not collect behind the wall. Perhaps a small concrete diversion culvert. Often just shutting off an over active irrigation system will mitigate the problem. Additional weep holes can also be cored through the wall, although perhaps visually objectionable.

2. Reduce the retained height. If the soil pressure needs to be reduced, investigate whether re-grading of the surface can reduce the height of earth retained. Sometimes a change in landscaping, or a depressed drainage culvert at the back of the wall may reduce the height to an acceptable level based upon the as-built capabilities.

3. Use tie-backs. If the stem is severely overstressed, an option is to use tie-backs extending back beyond the failure plane. Drill holes through the wall and install conventional tiebacks (also called soil nailing). A downside of this is the appearance of the tie-back anchors on the exposed face of the wall. Or perhaps a tie-back at the surface can be used, with a concrete anchor block, or an added slab-on-grade. Using tie-backs requires re-analyzing the wall moments and shears because of the changed restraints.

4. Extend the footing. You can extend the toe of the footing and thereby substantially reduce the soil bearing pressure. Determine how much you need to extend the footing, then excavate to the bottom of the footing (add deeper for a key if necessary) and place concrete. To transfer shear and moment at the interface, drill holes in the existing footing and epoxy dowels to resist the calculated pullout. It may be prudent to maintain lateral stability by excavating in front of the toe in longitudinal increments, say twenty feet, or less.

5. Remove and replace backfill material. This may be the only solution if saturated backfill is the problem and cannot be controlled at the surface. Use crushed rock backfill, and be sure the base-of-wall drainage is functional.

6. Reinforce the front of the wall. This can be done by forming or pneumatically placing concrete to thicken the base, and tapering to a height where the added strength is no longer needed. This is on the compression side so the only design concern (other than how much thickness to add) is shear transfer at the interface, which can be accomplished by drilled dowel pins. This assumes, of course, that the existing footing will still be adequate.

7. Add a key. If there is a sliding problem you could add a deepened key in front of the existing footing. This will increase passive resistance and may be all you need. See #4 above for incremental excavating during this process.

8. Use cantilevered soldier beams. Drill holes on the heel side of the footing and embed a vertical beam, tied to the wall to transfer load to the beam. Space the beams at a distance the wall will span horizontally. The footing heel will determine how close to the wall the soldier beams (piles?) can be placed.

9. Get a building permit. Quite often there is no apparent distress in a wall, but an observant building inspector discovers that a permit had not been issued. This usually happens when a new building or addition is being constructed on the property. If plans for the wall are found it requires only substantiating calculations with an engineers signature. If it can't be justified, then one of the procedures above are needed to remedy an overstress. If no plans are found it's necessary to determine how the wall was built. This means probing and perhaps testing to determine location and spacing of reinforcing, toe and heel dimensions of the footing, and perhaps core tests of the wall material. The task then is one of working backward to find the capacity of the wall and hence its adequacy. Lesson: Always get a permit; it could save a future expense.

10. Push it back to plumb. Not recommended, but has been successfully done in some cases if the wall is only out of plumb an inch or two, and not all backfill has been placed, and depending upon its height, and in conjunction with the above fixes, the wall can be pushed back to near-plumb. The wall may have been bumped to cause this, or tractor compacting too close to the wall. This is an arguable procedure but has been done successfully with no after effects. Use this with extreme caution! You may want to remove some backfill first.

11. "Tear down that wall". If it's in bad shape, and none of the above make sense, it can be less costly to tear it down and rebuild. Especially valid if new conditions exist, such as need for a higher wall or a preference for a different wall material. See Case D below.

12. An exotic solution? We engineers pride ourselves on innovation. There may be a unique site condition that suggests a cost-effective fix. And you could come up with an ingenious method of saving a wall from reconstruction, and be a hero to a very happy client!

Some actual cases

Here are a few examples (edited) of problems that have occurred:

Case A. A wall was observed to lean excessively and it was found that the reinforcing protruding from the foundation was on the wrong side of the wall. *Solution: Add tie-backs.*

Case B. A wall was observed to lean excessively. Investigation revealed the wall had been designed to retain 12 feet of earth, with an extension of the wall another four feet above grade for screening. The owner and his landscaper arbitrarily added two additional feet of earth, thereby increasing the moment at the base of the stem by 60%! *Solution: Add tie-backs.*

Case C. Again the sign of a problem was leaning of the wall. Investigation discovered the contractor had misinterpreted the plans and halved the number of dowels projecting from the footing. *Solution: Gunite added wall thickness at the base, bonded to the existing wall.*

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23. CONSTRUCTION TOPICS AND CAVEATS

Horizontal Control Joints

Horizontal joints are intended for crack control and are largely a matter of judgment. Shrinkage in a wall cannot be eliminated. As the adage goes, concrete shrinks and ice cream melts. But 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 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

Lateral earth pressure theories are 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 base for any water to freely flow, otherwise the only thing coming out of a weep hole will be inside 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 plastic 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. Preventing water from entering the backfill is critical important because it changes the soil characteristics and increases lateral pressures.

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 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

8The 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 report 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. At an impasse, mediation can be a very effective and less costly (i.e. attorney fees) resolution of a dispute.

24. RETAINING WALL DESIGN EXAMPLES

Description of Design Examples:

These fourteen designs illustrate a variety of design conditions for retaining walls. They are worked by hand - the way you are accustomed to design retaining walls. You may use a different format, and your methodology may be a little different, but the results should be nearly the same. They are intended to show accepted design procedures. They are based on IBC '06 ('09 is similar), ACI 318-08, MSJC '05, and NCMA-SRW.

Following each of the examples is a report printout for the same problem using Retain Pro 9. This allows you to compare results, which should closely agree, given round-offs and shortcuts in hand calculations which most of us do for expediency.

Example #1 - Retaining wall with sloped backfill, and stem of both concrete and masonry. The problem is designed so a key is necessary.

Example #2 - A wall with an adjacent footing, and wind on a projecting stem.

Example #3 - This problem illustrates a heel-side surcharge, and an axial load consisting of both dead and live load, and an eccentricity.

Example #4 - This wall has a fence (zero weight and with wind load) on top of the retaining wall, and a property line condition.

Example #5 - This is a freestanding wall with seismic force due to self-weight applied, and only minor earth retaining. It is set on a property line. Remember that for free-standing walls designed for seismic or wind, these loads can act in each direction, and if the controlling direction is not obvious, you may need to check the reversed too.

Example #6 - This illustrates a concrete stem with the inside face tapered (battered) and with a seismic force due to earth pressure.

Example #7 - Masonry "basement" wall restrained laterally near the top.

Example #8 - Concrete "basement" wall restrained laterally near the top..

Example #9 - A rubble gravity wall design.

Example #10 – A segmental wall (MSE) with geogrids

Example #11 – A segmental gravity wall -- no geogrids.

Example #12 - A pier foundations option for Example #1.

Example #13 – Solder beam design – cantilevered

Example #14 – Gabion Wall (or multi-wythe large blocks)

Design Data

Code: IBC '06

Soil slope = 2:1

Soil density = 110 pcf

Equiv. fluid press. = 45 pcf

Active pressure toe side = 30 pcf

Passive press. = 389 pcf

$\mu = 0.40$

$F_s = 24{,}000$ psi

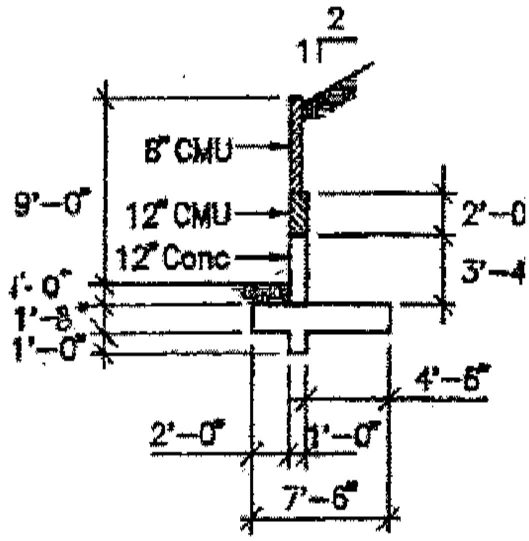
* $f'_m = 1500$ psi

$f'_c = 2500$ psi

$f_y = 60{,}000$ psi

Allow. soil pressure: 3000 psf

Angle of internal friction = 34°



Check by Rankine formula

for $\phi = 34°$ $\quad \beta = \tan^{-1}\left(\frac{1}{2}\right) = 26.6°$

K_a (slope) = 0.406 Pressure = .406 x 110 = 45 pcf

K_a (level) = 0.28 Pressure = .28 x 110 = 30 pcf

K_p (level = $\tan^2\left(45 + \frac{\varphi}{2}\right) = 3.54$ Pressure = 3.54 x 110 = 389 pcf

IBC '06 Load Factors for Strength Design (Concrete)

DL = 1.2

LL = 1.6

H = 1.6

W = 1.6

E = 1.0

Check Stem at Base

(Design Ht. – 0.00')

$$M_u = \frac{45 \, x \, 10^3}{2} \; x \; \frac{10}{3} \; x \; 1.6 = 11{,}999'\#$$

Use t = 12", d = 9.6"

$$A_s \text{ (req'd)} = \frac{1.7 \, f_c' \, bd}{2 \, f_y} - \frac{1}{2} \sqrt{\frac{2.89 \, (f_c' \, bd)^2}{f_y^2} - \frac{6.8 \, f_c' \, bM_u}{\phi f_y^2}} = 0..29 \; sq. \, in. \text{ (per CRSI eq.)}$$

$$\rho_{bal.} = \frac{.85 \, f_c'}{f_y} \; x \; .85 \left(\frac{87{,}000}{87{,}000 + f_y} \right) = 0.018$$

$\rho_{max.} = .75 \, \rho_{bal.} = 0.0106 \; \rho_{min.} = \dfrac{200}{f_y} = .0033$ (or at least 1.33 A_s required.)

Try #6 at 16 $\quad A_s = \dfrac{0.44}{1.33} = 0.33 > 0.31$ OK for strength

But $\rho = \dfrac{0.44}{16 \, x \, 9.6} = 0.0029 < .0033$ N.G.

Try #7 at 16" $\quad \rho = \dfrac{.60}{16 \, x \, 9.6} = .0039 > .0033 \qquad As = \dfrac{.60}{1.33} = .45$ sq. in. OK

$$a = \frac{.45 \, x \, 60{,}000}{.85 \, x \, 2500 \, x \, 12} = 1.06$$

$$\Phi M_u = .9 \, x \, .45 \, x \, 60{,}000 \left(9.6 - \frac{1.06}{2} \right) x \, \frac{1}{12} = 18{,}367'\#$$

$$V_u = \left(\frac{45 \, x \, 10^2}{2} - \frac{30 \, x \, 1}{2} \right) 1.6 = 3576\#$$

$$v_u = \frac{3576}{12 \; x \; 9.6} = 31.0 \quad v_{allow} = \underset{\substack{\uparrow \\ .75}}{\phi} \, 2\sqrt{f_c'} = 75 > 31.0 \text{ OK}$$

Check embedment into footing:

* For hooked bar (ACI 12.5) =

$$\frac{0.02 f_y\, d_b}{\sqrt{f'_c}}\ x\ 0.7 = \frac{.02\ x\ 60{,}000\ x\ 0.875\ x\ 0.7}{\sqrt{2500}} = 14.7'' \quad or\ 8\,d_b \quad or\ 6''$$

* If ACI 318 '08, 12.5.3.4 is deemed applicable, reduce embedment by (A_s req'd) / (A_s provided).

Min. footing thickness required = 14.7 + 3 = 17.7 in. Use 20"

Development length into stem

$$\ell_d = \frac{3\,F_y\ \alpha\beta\lambda\ d_b}{40\sqrt{f'_c}\left(\dfrac{c+K_{tr}}{d_b}\right)} \quad [See\ ACI\ 12.2.3,\ equation\ 12-1]$$

$$\left(\frac{c+K_{tr}}{d_b}\right) = \left(\frac{2.0+.44+0}{.875}\right) = 2.79\ \rangle\ 2.5\ \text{max.}$$

$$\therefore \ell_d = \frac{3\ x\ 60{,}000\ x\ 1\ x\ 1\ x\ 1\ x\ .875}{40\ x\ 50\ x\ 2.5} = 31.5\ in.$$

Note: If lapped with continuing reinf. of same size, splice length (assuming Class B splice) = 1.3 x 31.5 = 41"

<u>Check Masonry Stem</u> - Allowable Stress Design

Switch to 12" masonry at 3'-4" high which is approx. top of dowels.

f'_m = 1500 psi F_s - 24,000 f_b = .33 x 1500 = 500 psi

E_m = 900 f'_m E_m = 1,350,000 psi E_s = 29,000,000 $n = \dfrac{E_s}{E_n}$ 21.5

$$M\ @ +3.33'\ (= H\ of\ 6.67') = \frac{45\ x\ 6.67^2}{2}\ x\ \frac{6.67}{3} = 2225^{'\#}$$

Try #5 at 16" at edge

$A_s = \frac{.31}{1.33} = 0.23$ $\rho = \frac{.31}{16 \, x \, 9} = .0022$

$np = 21.5 \times .0022 = .0473$ $\frac{2}{kj} = 8.33$ $j = 0.91$ « From Amreim, Table E-9

$$M_m = \frac{f_b \, bd^2}{\frac{2}{kj}} = \frac{500 \, x \, 12 \, x \, 9^2}{8.33} \, x \, \frac{1}{12} = 4862'^{\#}$$

$M_s = f_s \, A_s \, jd = 24{,}000 \times .23 \times .91 \times 9 \times \frac{1}{12} = 3767'^{\#}$ « governs

∴ OK for # 5 @ 32 @ edge (overly conservative and could be reduced for final design)

$V = \frac{45 \, x \, 6.67^2}{2} = 1001^{\#}$ $v = \frac{1001}{12 \, x \, .91 \, x \, 9} = 10.2 < 1.0\sqrt{f'_m} = 38.7$ OK

Lap length into concrete below (ACI 12.2.3) –

$$\ell_d \; \frac{3 \, x \, 60{,}000 \, x \, 1 \, x \, 1 \, x \, 0.8 \, x \, 0.625}{40 \, x \, 50 \, x \, 2.5} \, x \, 1.3 = 23.4''$$

Lap length into masonry above – $.002 \, d_b f_s$

$= .002 \times .625 \times 24{,}000 = 30.0''$

Check Stem at + 5.33'

Reduce to 8" masonry, grout reinf. cells only

$f'_m = 1500$ $F_s = 24{,}000$ $F_b = 1500 \times .33 = 500$ psi

$d = 5.25"$ $n = 21.5$

Depth @ + 5.33' = 10.00 - 5.33 = 4.67'

$$M = \frac{45 \, x \, 4.67^2}{2} \, x \, \frac{4.67}{3} = 763'^{\#}$$

Use #5 at 32" at edge

$$np = \frac{21.5 \;\; x \;\; .31}{32 \;\; x \;\; 5.25} = .04$$

$\frac{2}{kj} = 8.87 \quad j = 0.92$

$$M_m = \frac{500 \ x \ 12 \ x \ 5.25^2}{8.87} \ x \frac{1}{12} = 1553'^{\#}$$

$$M_s = 24{,}000 \ x \ \frac{.31}{2.67} \ x \ .92 \ x \ 5.25 \ x \ \frac{1}{12} - 1121'^{\#} \leftarrow \text{governs}$$

OK because $1109 > 763$

$$V = \frac{45 \ x \ 6.67^2}{2} = 490^{\#}$$

$$v - \frac{490}{12 \ x \ .91 \ x \ 5.25} = 8.5 \text{ psi}$$

$v_{allow} = 38.7$ psi $\rangle$ 8.5 OK

Lap embedment below

$= .002 \ d_b \ x \ 24{,}000$

$= .002 \ x \ .625 \ x \ 24{,}000 = 30"$

Stability Check

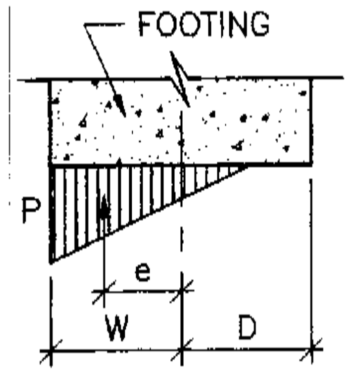
Item	Wt. (lbs.)	*arm (ft.)	+M (ft.-lbs.)	-M (ft.-lbs.)
8" CMU 78 x 4.67	364	2.33	849	
12" CMU 124 x 2.0	248	2.50	620	
12" conc. 150 x 3.33	500	2.50	1250	
earth 4.5 x 10 x 110	4,950	5.25	25,988	
earth .33 x 4.67 x 110	170	2.83	481	
earth 4.5 x 2.25 x 110 x ½	557	6.00	3,341	
earth 2.0 x 1.0 x 110	220	1.0	220	
footing 7.5 x 1.67 x 150	1,875	3.75	7,031	
key 1 x 1 x 150	150	2.5	375	
P_a = $\frac{45\,x\,(10.0+1.67+2.25)^2}{2} = 4360$		$\frac{13.92}{3}$		20,229
P_v = ½ x 4210	2,105	7.5	15,788	
	11,139		55,943	20,229
w/o P_v -	9,034		40,155	

$$\bar{x} = \frac{RM - OTM}{W} = \frac{40{,}155 - 20{,}229}{9034} = 2.21'$$ * About front edge of footing

$$e = \frac{7.5}{2} - 2.21 = 1.54' = 18.5"$$

Middle $\frac{1}{3} = \frac{7.5}{6} = 1.25 < 1.54$ ∴ outside middle third

When resultant outside middle third:



Soil pressure = $\dfrac{W}{.75D - 1.5e}$

$$* \text{ Soil P} = \frac{9034}{.75 \times 7.5 - 1.5 \times 1.54} = 2725 < 3000 \text{ lbs./sq. ft.} \quad \text{OK}$$

$$\text{OTM ratio} = \frac{40{,}155}{20{,}184} = 1.99$$

Check OTM Using P_v

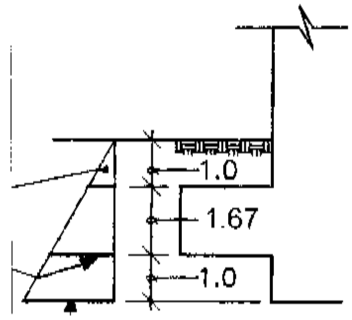
RM = 40,155 + 15,788 55,943 #

OTM 20,158 #

W = 9034 + 2105 = 11,139

$$\text{OTM ratio} = \frac{55{,}943}{20{,}158} = 2.78$$

Check Sliding



Total lateral =

$$\frac{45\ x\ 13.92^2}{2} - \frac{30\ x\ 2.67^2}{2} = 4253^{\#}$$

Friction resistance = 9034 x 0.40 = 3614#

Passive resistance: Neglect

389 x 2.67 = 1039

389 x 3.67 = 1428

Total passive = 1428 x 3.67 x $\frac{1}{2}$ - 389 x 1 x $\frac{1}{2}$ =

$$\text{Factor of safety} = \frac{3614 + 2426}{4253} = 1.42 \approx 1.50$$

Consider OK

Check Key

$$V_u = \frac{1428 + 1039}{2} \text{ x } 1 \text{ x } 1.6 = 1974^{\#}$$

$$v_u = \frac{1974}{12(12-2)} = 16.4 < 2\varphi\sqrt{f_c'} = 58.1$$

$\uparrow \phi = 0.65$

(deduct 2" from footing thickness for plain concrete) [ACI 22.4.8]

$$M_u = \left[1039\ x\ 1\ x\ \frac{12}{2} + \frac{(1428 - 1039)1.0}{2}\ x\ \frac{2\ x\ 12}{3}\right] 1.6 = 12{,}464^{"\#}$$

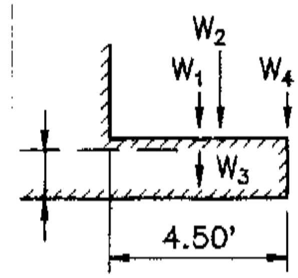
$$S = \frac{12\ (12 - 2)^2}{6} = 200$$

$$f_r = \frac{12{,}464}{200} = 62.3\ psi < 5\varphi\sqrt{f_c'} = 5\ x\ 0.55\ x\ 50 = 137.5 \quad OK$$

No reinf. req'd.

Check Heel

Neglect upward soil pressure



$W_1 = 4950$

$W_2 = 557$ Use d = 20.0 - 2.5 = 17.5 in.

$W_3 = 4.5 \times 1.67 \times 150 = 1127^{\#}$

$W_4 = (P_v$ not used)

$M_u = 4950 \times 2.25 \times 1.2 + 557 \times 3.0 \times 1.2 + 1127 \times 2.25 \times 1.2$

$= 18{,}413^{'\#}$

$Mu_{Stem} = 11{,}992^{'\#}$ ← governs

(Note: By statics, heel moment cannot exceed applied stem moment)

$$*A_s = \frac{1.7\, f_c'\, bd}{2\, f_y'} - \frac{1}{2}\sqrt{\frac{2.89\,(f_c'\, bd)^2}{f_y^2} - \frac{6.8\, f_b'\, M_u}{\varphi\, f_y^2}}$$

*[from C.R.S.I. Handbook, M_u in inch-kips]

$\phi = 0.90$

↓ ACI 10.5.3

$$A_s = 0.154\ {}^{sq.\,in.}\!/_{ft.} \quad or\ \frac{200}{f_y} \times 12 \times 17.5 = .57 \quad \text{or } 1.33 \times 0.154 = 0.20$$

Select #6 at 16 to match stem dowels ($A_s = .33 > .20$ OK)

Embedment length beyond stem face (using stress ratio reduction)

$$\frac{3 \times 60 \times 1.0 \times 1.0 \times 0.8 \times 0.75}{40 \times 50 \times 2.5} \times .80 \times \frac{.154}{.44/1.33} = 10.1'' \langle\ 12''\ \text{min.}$$

Use 12" embedment beyond stem face.

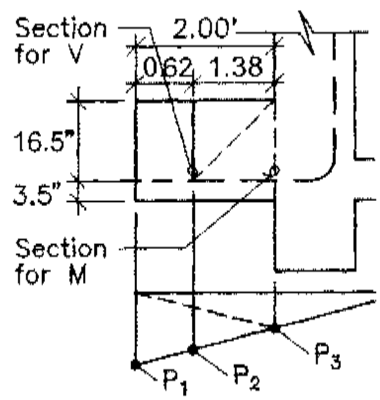
V_u – 4950 x 1.2 + 557 x 1.2 + 1127 x 1.2 + (w/o P_v) 7961#

$$v_u = \frac{7961}{12\,x\,17.5} = 37.9\,psi < 2\varphi\sqrt{f_c'} = 2\,x\,.75\,x\,\sqrt{2500} = 75\,psi \qquad \text{OK}$$

Check Toe

Apply total factored vertical load (w/o p_v) at same eccentricity as service loads:

$= 9034 \text{ x } 1.2 = 10{,}841^{\#}$



Factored soil pressure

$$= \frac{9034\,x\,1.2}{.75\,x\,7.5 - 1.5\,x\,1.54}\,x\ 3270\ \text{psf} = @\ P_1$$

Soil Pressure @ P_2 (for shear)

$$= \frac{3\,x\,2.21 - .62}{3\,x\,2.21}\,x\ 3270 = 2964\ \text{psf}$$

Soil Pressure @ P_3 (Max. M) =

$$\frac{3\,x\,2.21 - 2.0}{3\,x\,2.21}\,x\ 3270 = 2283\ \text{psf}$$

But because stem moment governs

$$V_u = \frac{11{,}992}{4.5/2}\,x\ \frac{1.2}{1.6} = 3997^{\#}$$

v_u 3997 / (12 x 17.5) = 19.0 psi < 75 OK

$$M_u\uparrow = \frac{3270 \times 2}{2} \times .67 \times 2 + \frac{2284 \times 2}{2} \times .33 \times 2 = 5905^{\#}$$

$$M_u\downarrow = (1.67 \times 150 + 1 \times 110) \times \frac{2^2}{2} \times 1.2 = 865^{\#}$$

$$M_{design} = 5905 - 865 = 5040^{\#}$$

$$A_s min = \frac{200}{f_y} \times 12 \times 16.5 = .66 \text{ sq. in.}$$

A_s required (per C.R.S.I formula) = .07 sq. in.

A_s to override A_s min = 1.38 x .07 = 0.09 sq. in.

But also As min = 0.0018 x 12 x 16.5 = 0.36 ← Governs

Use #7 @ 16" ($A_s = \frac{0.60}{1.33} = 0.45 > 0.36$) OK

Shear negligible by inspection because only acts on 0.62 ft.

DESIGN EXAMPLE 1

Report Printout

Hugh Brooks, PE, SE
Retain Pro Software
PO Box 826 Corona del Mar, CA 92625
hbrooks@retainpro.com
949-721-4099

Title: EX-1
Job #:
Description...
Dsgnr:
Date: JUL 18,2006
Page: ____

This Wall in File: c:\program files\rp2007\examples.rpd

Retain Pro 9 © 1989 - 2009 Ver: 9.00 6856
Registration #: RP-1119115 RP6.00
Licensed to: [The company name goes here]

Cantilevered Retaining Wall Design

Code: IBC 2006

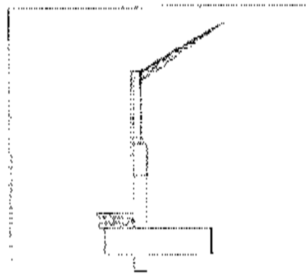
Criteria

Retained Height	=	10.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	2.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Vertical component of active lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	45.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	389.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
FootingIISoil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios		
Overturning	=	2.30 OK
Sliding	=	1.42 Ratio < 1.5!
Total Bearing Load	=	6,005 lbs
...resultant ecc.	=	13.30 in
Soil Pressure @ Toe	=	2,716 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	3,187 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	8.1 psi OK
Footing Shear @ Heel	=	37.9 psi OK
Allowable	=	75.0 psi
Sliding Calcs (Vertical Component NOT Used)		
Lateral Sliding Force	=	4,251.9 lbs
less 100% Passive Force	=	- 2,425.4 lbs
less 100% Friction Force	=	- 3,513.9 lbs
Added Force Req'd	=	0.0 lbs OK
...for 1.5 : 1 Stability	=	342.1 lbs NG
Load Factors		
Building Code		IBC 2006
Dead Load		1.200
Live Load		1.600
Earth, H		1.600
Wind, W		1.600
Seismic, E		1.000

Stem Construction

			Top Stem	2nd	3rd
			Stem OK	Stem OK	Stem OK
Design Height Above Ftg	ft	=	5.33	3.33	0.00
Wall Material Above "Ht"		=	Masonry	Masonry	Concrete
Thickness		=	8.00	12.00	12.00
Rebar Size		=	# 5	# 5	# 7
Rebar Spacing		=	32.00	16.00	16.00
Rebar Placed at		=	Edge	Edge	Edge
Design Data					
fb/FB + fa/Fa		=	0.681	0.483	0.649
Total Force @ Section	lbs	=	400.7	1,001.0	3,576.0
Moment....Actual	ft-#	=	763.9	2,225.6	11,862.0
Moment.....Allowable	ft-#	=	1,121.1	3,620.4	18,280.4
Shear.....Actual	psi	=	7.8	9.1	31.2
Shear.....Allowable	psi	=	38.7	36.7	75.0
Wall Weight	psf	=	78.0	124.0	150.0
Rebar Depth 'd'	in	=	5.25	9.00	9.56
LAP SPLICE IF ABOVE	in	=	30.00	30.00	26.59
LAP SPLICE IF BELOW	in	=	20.00	13.63	
HOOK EMBED INTO FTG	in	=			14.70
Masonry Data					
f'm	psi	=	1,500	1,500	
Fs	psi	=	24,000	24,000	
Solid Grouting		=	Yes	Yes	
Modular Ratio 'n'		=	21.48	21.48	
Short Term Factor		=	1.000	1.000	
Equiv. Solid Thick.	in	=	7.60	11.60	
Masonry Block Type		=	Medium Weight		
Masonry Design Method		=	ASD		
Concrete Data					
f'c	psi	=			2,500.0
Fy	psi	=			60,000.0

DESIGN EXAMPLE 1

Report Printout

Hugh Brooks, PE, SE
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PO Box 826 Corona del Mar, CA 92625
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949-721-4099

Title : EX-1 Page:
Job # : Dsgnr: Date: JUL 16,200
Description...

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 8 © 1989 - 2007 Ver: 9.00 8056
Registration #: RP-1110115 RP8.00
Licensed to: [The company name goes here]

Cantilevered Retaining Wall Design

Code: IBC 2006

Footing Dimensions & Strengths

Toe Width	=	2.00 ft
Heel Width	=	5.50
Total Footing Width	=	7.50
Footing Thickness	=	20.00 in
Key Width	=	12.00 in
Key Depth	=	12.00 in
Key Distance from Toe	=	2.00 ft
f'c = 2,500 psi Fy	=	60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm.=	3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	5,197	0 psf
Mu' : Upward	=	5,753	0 ft-#
Mu' : Downward	=	884	0 ft-#
Mu: Design	=	4,869	11,992 ft-#
Actual 1-Way Shear	=	8.12	37.90 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	#7 @ 18.00 in	
Heel Reinforcing	=	#6 @ 15.00 in	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: #4@ 6.50 in, #5@ 10.00 in, #6@ 14.00 in, #7@ 19.25 in, #8@ 25.25 in, #9@ 31
Key: #4@ 12.50 in, #5@ 19.25 in, #6@ 27.25 in, #7@ 37.25 in,

Summary of Overturning & Resisting Forces & Moments

Item		....OVERTURNING.... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	4,357.7	4.64	20,214.7
Surcharge over Heel	=			
Toe Active Pressure	=	-106.7	0.89	-94.8
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	4,251.0	O.T.M. =	20,119.9

Resisting/Overturning Ratio = 2.00
Vertical Loads used for Soil Pressure = 9,034.9 lbs
Vertical component of active pressure NOT used for soil pressure

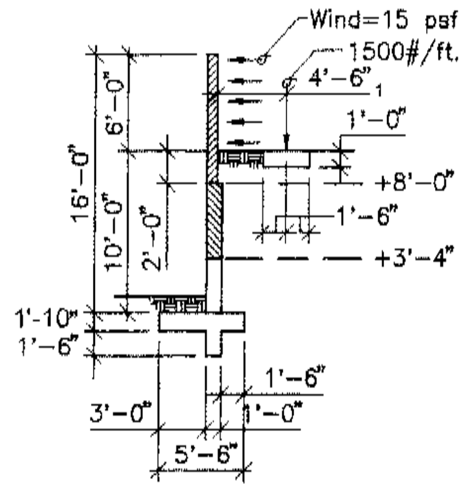
Item		....RESISTING.... Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	4,950.0	5.25	25,987.5
Sloped Soil Over Heel	=	556.9	6.00	3,341.3
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	220.0	1.00	220.0
Surcharge Over Toe	=			
Stem Weight(s)	=	1,111.8	2.45	2,718.7
Earth @ Stem Transitions	=	171.2	2.83	485.2
Footing Weight	=	1,875.0	3.75	7,031.3
Key Weight	=	150.0	2.50	375.0
Vert. Component	=			
Total =		9,034.9 lbs	R.M.=	40,159.9

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

Design Data

Building Code: IBC '06
Soil bearing – 2000 psf
Soil density = 110 pcf
Eq. fluid pressure = 30 pcf
Passive = 300 pcf
μ = 0.40
Poisson's Ratio (for Bousinesq) = 0.50
f'_m = 1,500 psi
f_s = 24,000 psi
f_y = 60,000 psi
f'_c = 2,000 psi



Check Stem at +8.00'

$$M = 15 \times 6\left(\frac{6}{2}+2\right) + \frac{30 \times 2^2}{2} \times \frac{2}{3}$$

$$= 490^{\prime\#}$$

f'_m = 1500 psi $\qquad f_b$ = .33 x 1500 = 500 psi

d = 3.75 (for 6" CMU)

$E_m = 900 \times f'_m - 1{,}350{,}000$ psi

$E_s - 29{,}000{,}000 \qquad n = 21.5$

$$A_s = \frac{490 \times 12}{24{,}000 \times 1.33 \times .90 \times 3.75} = .054$$ Use #4 @ 32

Assume ↑ $\qquad A_s = \dfrac{.20}{2.67} = .075$ OK

$$np - \frac{21.5 \times .20}{32 \times 3.75} = .036 \qquad {}^{2}\!/_{kj} = 9.24 \qquad j - .92$$

M_s = $24{,}000 \times 1.33 \times .075 \times .92 \times 3.75 \times \frac{1}{12} = 690^{\#}$ ← governs

M_M = $500 \times 12 \times 3.75^2 \times \frac{1}{9.24} \times \frac{1}{12} = 761^{\#}$

$V = 6' \times 15 + \frac{30 \times 2^2}{2} = 150^{\#}$

$v = \frac{150}{12 \times .92 \times 3.75} = 3.62 \qquad v_{allow} = \sqrt{f'_c} \times 1.33 = 59$

Rebar embedment below = .002 x .50 x 24,000 = 24"

Check Stem @ ± 3.33'

Change to 12" CMU, d = 9.0"

$M_{wind} = 6' \times 15 \text{ psf} \times \left(\frac{6}{2} + 6.67\right) = 870^{\#}$

$M_{soil} = \frac{30 \times 6.67^2}{2} \times \frac{6.67}{3} = 1484$

$M_{bousinesq}$ (from program) = $\underline{1107}$

3461 $^{\#}$

Total lateral = $6' \times 15 + \frac{30 \times 6.67^2}{2} + 505 = 1262^{\#}$

(from program) ↑

$A_s = \frac{3461 \times 12}{24{,}000 \times .9 \times 9.0} = 0.21$ Use #5 @ 16 @ edge

(Disallow $\frac{1}{3}$ wind stress increase at this level)

$np = \frac{21.5 \times .31}{16 \times 9} = .046 \qquad 2/kj = 8.4 \qquad j = 0.91 \qquad A_s = \frac{.31}{1.33} = .23$

$M_s = .23 \times 24{,}000 \times .91 \times 9 \times \frac{1}{12} = 3767^{\#}$ > 3461 OK

$$M_{ui} = \frac{500 \times 12 \times 9^2}{8.4} x \frac{1}{12} = 4821'\#$$

$$V = 6 \times 15 + \frac{30 \times 6.67^2}{2} + 504 = 1261\#$$

$$v = \frac{1261}{12 \times .91 \times 9} = 12.8 < \sqrt{f_m'} = 38.7 \quad \text{OK}$$

Embedment length = .002 x .625 x 24,000 = 30"

Check Stem @ Base

Use 12" CMU

$$M_{wind} = 6' \times 15 \text{ psf} \left(\frac{6}{2} + 10\right) = 1170'\#$$

$$M_{soil} = \frac{30 \times 10^2}{2} x \frac{10}{3} = 5000$$

$M_{bousiresq.}$ (from program) = $\underline{3245}$

9415 '#

f_m' = 1500 psi.

f_b = 500 psi n = 21.5 d = 9.0 $A_s = \dfrac{9415 \times 12}{24{,}000 \times .90 \times 9} = 0.58$

↑ Assume

Try #8 @ 8"

$$A_s = \frac{0.79}{0.67} = 1.18 \text{ sq. in./ft.}$$

$$np = \frac{21.5 \times 0.79}{8 \times 9} = 0.236 \qquad {}^2/_{kj} = 4.88 \qquad j = .83$$

$$M_s = \frac{1.33 \ x \ 24{,}000 \ x \ .83 \ x \ 9.0}{12} = 19{,}870''\#$$

$$M_m = \frac{500 \ x \ 12 \ x \ 9^2}{4.88} \ x \ \frac{1}{12} = 8299 \approx 9415$$

$$\text{Stress Ratio} = \frac{9415}{8299} = 1.33$$

(13% overstressed optional redesign)

$$V = 6' \text{ x } 15 + \frac{30 \ x \ 10^2}{2} + 737 = 2327^{\#}$$

Bousinesq ↑

$$v = \frac{2327}{12 \ x \ 9} = 21.6 < \sqrt{f'_m} = 38.7 \quad \text{OK}$$

Development length of dowels

ℓ_{db} = .002 x 24,000 x 1.0 x 1.5 = 72.0 in.

Choose not to reduce by stress ratio. Assume continuing bars will be smaller diameter, therefore, 1.3 multiplier for splice lap not required.

$$\text{Embedment for hooked bar into footing} = \frac{0.02 \ x \ f_y \ d_b \ x \ 0.7}{\sqrt{f'_c}}$$

$$= \frac{.02 \ x \ 60{,}000 \ x \ 1.0 \ x \ 0.7}{44.7} = 18.8''$$

Min. ftg. Thickness = 18.8 + 3.0 = 21.5"

Use 22" thick

Stability

Try footing 5'-6" x 1'-10" thick

Item	Wt.		arm		+M	-M
8" stem 8' x 78 psf	624	x	3.33	=	2078	
12" stem 8' x 124 psf	992	x	3.50	=	3472	
Soil @ heel 1.5 x 110 x 10'	1650	x	4.75	=	7838	
Soil @ toe 3 x 1 x 110	330	x	1.50	=	495	
Soil behind stem .33 x 110 x 2	74	x	3.83	=	283	
Footing 5.5 x 1.83 x 150	1510	x	2.75	=	4152	
Key	225	x	3.50	=	788	
Adj. Footing	167	x	4.75	=	795	
OTM Wind = 6' x 15			14.83			1335
OTM soil = $\frac{30\ x\ 11.83^2}{2}$			$11.83/3$			8278
OTM adj. footing 830#			5.7			4731
	5572#		$2.83/3$		19,899#	14,344#

$$\bar{x} = \frac{19{,}889 \quad 14{,}344}{5572} = 1.00$$

$$e = \frac{5.5}{2} - 1.00 = 1.75 > \frac{5.5}{6} = .92$$

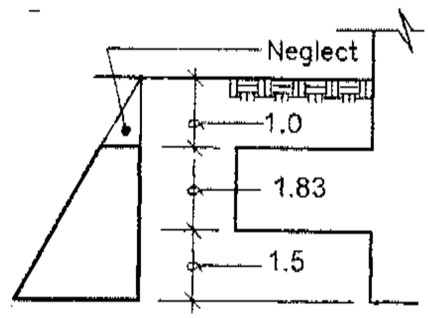
∴ outside middle third – designer may prefer redesign for within middle third.

$$\text{Soil p} = \frac{5572}{.75\ x\ 5.5 - 1.5\ x 1.75} = 3715\ psf \langle 4000 \quad \text{OK}$$

$$\text{OTM ratio} = \frac{19{,}899}{14{,}344} \quad 1.39 \approx 1.50 \quad \text{(consider OK?)}$$

<u>Check Sliding</u>

Lateral force of adj footing
↓



Total lateral = $15 \times 6' + \dfrac{30\,x\,11.83^2}{2} + 830$

= 3019 lbs.

Less friction resistance

= 5572 x .40 = 2229

Less passive

$$= \frac{300\;x\;4.33^2}{2} - \frac{300\;x\;1^2}{2} = 2662$$

Sliding ratio = $\dfrac{2229 + 2662}{3020}$ = 1.62

<u>Check Key</u>

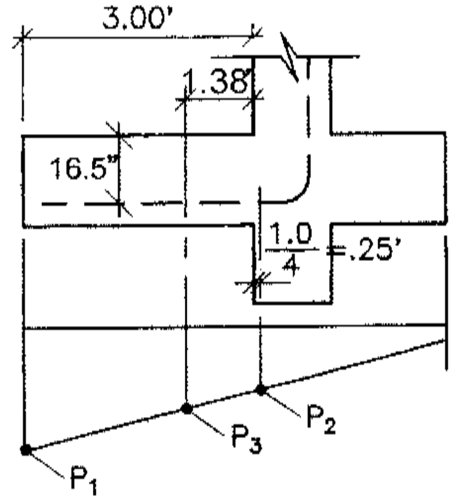
$$\text{Force} = \frac{300\;x\;4.33^2}{2} - \frac{300\;x\;2.83^2}{2} = 1611^{\#}$$

Effective width = 12" - 2" = 10"

$$v = \frac{1.6\;x\;1611}{12\;x\;9} = 23.9 < 2\varphi\sqrt{f_c'} = 85 \qquad \text{OK}$$

$$M_u\ (\text{approx.}) = 1611 \times \frac{18''}{2} \times 1.6 = 23{,}198''^{\#}$$

$$S = \frac{12 \ x \ (12-2)^2}{6} = 200$$



$$f_t = \frac{23,198}{200} = 116 < 5\varphi\sqrt{f_c'} = 137$$

OK

$\therefore$ No reinforcing required

Check Toe

Total vert. factored load

$= 5572 \times 1.2 = 6686^{\#}$

Factored soil pressure

$$= \frac{6686}{.75 \ x \ 5.5 - 1.5 \ x \ 1.73} = 4370 \text{ psf } @$$

P_1

$$P_2 = \frac{3 \ x \ 1.02 - 3.0 - .25}{3 \ x \ 1.02} \times 4370 \simeq 0.0$$

$$P_3 = \frac{3 \ x \ 1.02 - 3.0 + 1.54}{3 \ x \ 1.02} \times 4370 = 2285 \text{ psf}$$

$M_u\uparrow = 4370 \times 3.25 \times \frac{1}{2} \times .67 \times 3.25 = 15,394^{\#}$

$M_u\downarrow = [\ (1.83 \times 150 + 1 \times 110) \times 3 \times \left(\frac{3}{2} + .25\right)\ ] \times 1.2 = 2,422^{\#}$

$M_{design} = 15,394 - 2,422 = 12,972^{\#}$

A_s (Per CRSI formula - See Example #1) = 0.17

or $\left(\frac{200}{f_y}\right)$ 10 x 10.5 = 0.35 sq. in./ft.

Use 1.33 x 0.17 = 0.23

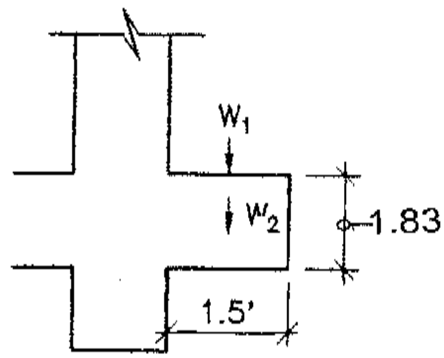
Use dowel bars : #8 @ 8" $A_s = \frac{.79}{.67} = 1.18$ OK

$$V_u = \frac{4370 + 2285}{2} \times (3 - 1.54) - (1.83 \times 150 + 1 \times 110)(3 - .1.54)\ 1.2$$

= 4996#

$$v_u = \frac{4996}{12 \ x \ 18.5} = 22.5$$

$v_{allow} = 2 \times .85 \sqrt{2500} = 85$ OK



<u>Check Heel</u>

Ignore upward soil pressure

W_1 =	992 x 1.2 =	1190 #
	+ 624 x 1.2 =	749
	+ 74 x 1.2 =	89
	+ 167 x 1.2 =	200
W^2 =	1.83 x 1.5 x 150 x 1.2 =	<u>494</u>
		2722 #

$$M_u = 3163 \times \frac{1.5}{2} = 2372'^{\#}$$

Effective thickness of footing = 20 - 2 = 18"

$$S = \frac{12 \ x \ 18^2}{6} = 648$$

$$f_r = \frac{2372 \ x \ 12}{648} = 43.9 < 5\phi\sqrt{f_c'} = 137 \quad \text{OK}$$

∴ No reinforcement required

V_u = 2722 #

$$v_u = \frac{2722}{12 \ x \ 20} = 11.3 < 2\phi\sqrt{f_c'} = 67 \quad \text{OK}$$

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Title : EX-2
Job # : 702
Description...
Dsgnr
Page: ____
Date: JUL 11,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8058
Registration #: RP-1164-5 RP9.00
Licensed to: [The company name goes here].

Cantilevered Retaining Wall Design

Code: IBC 2006

Criteria

Retained Height	=	10.00 ft
Wall height above soil	=	6.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Vertical component of active
lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	4,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe		110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Stem	=	15.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	1,500.0 lbs
Footing Width	=	3.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	4.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	-1.0 ft
Poisson's Ratio	=	0.500

Design Summary

Wall Stability Ratios		
Overturning	=	1.39 Ratio < 1.5!
Sliding	=	1.51 OK
Total Bearing Load	=	5,549 lbs
...resultant ecc.	=	20.07 in
Soil Pressure @ Toe	=	3,698 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	4,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	4,376 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	23.7 psi OK
Footing Shear @ Heel	=	10.9 psi OK
Allowable	=	87.1 psi
Sliding Calcs (Vertical Component NOT Used)		
Lateral Sliding Force	=	2,980.7 lbs
less 100% Passive Force	= -	2,404.2 lbs
less 100% Friction Force	= -	2,219.4 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

			Top Stem	2nd	3rd
			Stem OK	Stem OK	Stem < 1.5
Design Height Above Ftg	ft	=	6.00	3.33	0.00
Wall Material Above "Ht"		=	Masonry	Masonry	Masonry
Thickness		=	8.00	12.00	12.00
Rebar Size		=	# 4	# 5	# 6
Rebar Spacing		=	32.00	16.00	8.00
Rebar Placed at		=	Center	Edge	Edge
Design Data					
fb/FB + fa/Fa		=	0.897	0.986	1.148
Total Force @ Section	lbs	=	182.1	1,291.5	2,326.9
Moment....Actual	ft-#	=	465.1	3,481.5	9,410.0
Moment.....Allowable	ft-#	=	518.6	3,520.4	8,201.7
Shear.....Actual	psi	=	2.6	11.5	20.1
Shear.....Allowable	psi	=	38.7	38.7	38.7
Wall Weight	psf	=	78.0	124.0	124.0
Rebar Depth 'd'	in	=	3.75	9.00	9.00
LAP SPLICE IF ABOVE	in	=	36.00	45.00	72.00
LAP SPLICE IF BELOW	in	=	36.00	45.00	
HOOK EMBED INTO FTG	in	=			18.75
Masonry Data					
f'm	psi	=	1,500	1,500	1,500
Fs	psi	=	24,000	24,000	24,000
Solid Grouting		=	Yes	Yes	Yes
Modular Ratio 'n'		=	21.48	21.48	21.48
Short Term Factor		=	1.000	1.000	1.000
Equiv. Solid Thick.	in	=	7.60	11.60	11.60
Masonry Block Type		=	Medium Weight		
Masonry Design Method		=	ASD		
Concrete Data					
f'c	psi	=			
Fy	psi	=			

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Title : EX-2
Job # : 702
Description...

Dsgnr:

Page: ___
Date: JUL 11,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.20 8054
Registration #: RP-1110t 5 RP9/06
Licensed to: (The company name goes here)

Cantilevered Retaining Wall Design

Code: IBC 2006

Footing Dimensions & Strengths

Toe Width	=	3.00 ft
Heel Width	=	2.50
Total Footing Width	=	5.50
Footing Thickness	=	22.00 in
Key Width	=	12.00 in
Key Depth	=	16.00 in
Key Distance from Toe	=	3.00 ft
f'c = 2,000 psi Fy	=	60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm.=	3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	4,375	0 psf
Mu' : Upward	=	15,407	0 ft-#
Mu' : Downward	=	2,440	2,527 ft-#
Mu: Design	=	12,967	2,527 ft-#
Actual 1-Way Shear	=	23.85	10.59 psi
Allow 1-Way Shear	=	67.08	67.08 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: #4@ 6.25 in, #5@ 9.50 in, #6@ 13.25 in, #7@ 18.25 in, #8@ 23.75 in, #9@ 30.
Heel: Not req'd, Mu < S * Fr
Key: #4@ 12.50 in, #5@ 19.25 in, #6@ 27.25 in, #7@ 37.25 in,

Summary of Overturning & Resisting Forces & Moments

Item		OVERTURNING Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	2,100.4	3.94	8,265.8
Surcharge over Heel	=			
Toe Active Pressure	=	-120.4	0.94	-113.7
Surcharge Over Toe	=			
Adjacent Footing Load	=	830.7	5.66	4,700.1
Added Lateral Load	=			
Load @ Stem Above Soil	=	90.0	14.83	1,335.0
Total	=	2,900.7	O.T.M. =	14,206.4

Resisting/Overturning Ratio = 1.30
Vertical Loads used for Soil Pressure = 5,548.5 lbs

Item		RESISTING Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	1,650.0	4.75	7,837.5
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=	166.7	4.75	791.7
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	330.0	1.50	495.0
Surcharge Over Toe	=			
Stem Weight(s)	=	1,610.0	3.44	5,892.6
Earth @ Stem Transitions	=	73.3	3.83	281.1
Footing Weight	=	1,512.5	2.75	4,159.4
Key Weight	=	200.0	3.50	700.0
Vert. Component	=			
Total =		5,548.5 lbs	R.M.=	18,516.7

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

Design Data

Code: IBC '06

Eq. Fluid Press – 30 pcf

Soil Bearing = 3000 psf

Soil Density = 110 pcf

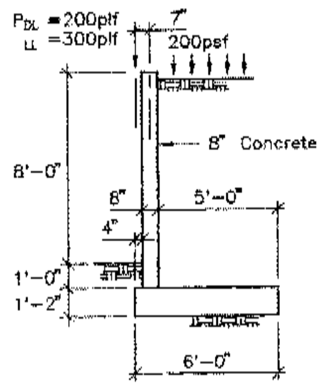
Passive = 300 pcf

Surcharge / Axial as shown

$f_c' \;=\; 2500$ psi

$f_y \;=\; 60{,}000$ psi

Neglect soil over toe for passive



Stem @ Base (= 10.0)

M_u =

$$\frac{30\,x\,9^2}{2}\,x\,\frac{9}{3}\,x\,1.6+200\,x\,\frac{30}{110}\,x\,\frac{9^2}{2}\,x\,1.6+(200\,x\,1.2+300\,x\,1.6)\frac{7}{12}=9{,}787\,'\#$$

$$d = 8 - 2 - \frac{.75}{2} = 5.63''$$

The general solution for A_s (per CRSI)

$$= \frac{1.7\,f_c'\,bd}{2\,f_y}-\frac{1}{2}\sqrt{\frac{2.89\,(f_c'\,bd)^2}{f_y{}^2}-\frac{6.8\,f_c'\,bM_u}{\phi\,f_y{}^2}}$$

For b (unit stem width) = 12", f_y' = 60 ksi: this reduces to:

$$A_s = 0.17 f_c'\,d - \sqrt{.029\;(f_c'\,d)^2 - .0063 f_c'\,M_u} \quad [M_u \text{ in in-kips}]$$

$$= .17 \times 2.5 \times 5.63 - \sqrt{.029\;(2.5\,x\,5.63)^2 - .0063\,x\,2.5\,x\,9.787\,x\,12}$$

$$= 0.42 \text{ sq. in./ft.}$$

A_s (Per above CRSI) = 0.42 sq. in./ft.

$$\rho_{min} = \frac{200}{f_y} = .0033$$

or 1.33 x 0.42 = 0.56 (See ACI 10.5.3)

Use # 6 @ 9" $\quad A_s = \frac{.44}{.75} = .59 \quad \rho = \frac{.44}{9\;x\;5.63} = .0087 \rangle .0033$

$$a = \frac{.44\;x\;60}{.85\;x\;2.5\;x\;9} = 1.38''$$

$$M_n = .59 \times 60{,}000 \left(5.63 - \frac{1.38}{2}\right) x\ .90\ x\ \frac{1}{12} = 13{,}115^{\#} > 9{,}787 \quad \text{OK}$$

$\phi\uparrow$

$$\text{Stress ratio} = \frac{9787}{13{,}115} = 0.746$$

<u>Check Stem Shear</u>

$$V_u = \frac{30\ x\ 9^2}{2}\ x\ 1.6 + 200\ x\ \frac{30}{110} \text{ x } 9' \text{ x } 1.6 = 2729^{\#}$$

$$v_u = \frac{2729}{12\ x\ 5.63} = 40.4 \text{ psi}$$

$$v_{allow} = \varphi 2\sqrt{f_c'} = .75\ x\ 2\sqrt{2500} = 75 \text{ psi} \quad \text{OK}$$

<u>Check embedment into footing</u>

$$\text{For hooked bar} = \frac{.02\ x\ f_y\ d_b\ x\ 0.7}{\sqrt{f_c'}}$$

or $8d_b$ or 6"

$$= \frac{0.02\ x\ 60{,}000\ \ x\ \ .75\ x\ 0.7}{50} = 12.6"$$

Choose to reduce by stress level per ACI 12.5.3.4

$\therefore$ embedment $12.6 \times .746 = 9.40"$

<u>Footing thickness required</u>

= 9.4" + 3" clear = 12.4"

<u>use 14" thick</u> (d = 14 - 3 - .50 = 10.5"
arbitrary ↑

<u>Lap length above footing</u>

$$\ell d_b = \frac{0.024\ x\ 0.75\ x\ 60{,}000}{\sqrt{2500}} \times 0.746 = 16.1"$$

Assume Class B splice w/continuing. #6 bars above, then lap = 16.1 x 1.3 = 20.94".

Check Stem @ + 1.5 Above Footing

$$M_u = \frac{30 \ x\ 7.5^2}{2}\ x\ \frac{7.5}{3}\ x\ 1.6 + 200\frac{30}{110}\ \ x\ \frac{7.5^2}{2}\ x\ 1.6 + (200\ x\ 1.2 + 300\ x\ 1.6)\ \frac{7}{12}$$

$$= 6250^{\#}$$

$A_s = 0.257$ [CRSI formula] Use #6 @ 18" $A_S = \frac{.44}{1.5} = 0.293 > 0.257$

$\underline{M_n\ @\ +\ 1.5'}$ $a = \frac{.44\ x\ 60}{.85\ x\ 2.5\ x\ 18} = 0.69$

$$M_n = \frac{.44}{1.5}\ x\ 60{,}000\ x\left(5.63 - \frac{.69}{2}\right)\ x\ .9\ x\ \frac{1}{12} = 6976'^{\#} > 6250 \quad \text{OK}$$

Lap length over dowels = $\frac{.024\ x\ .75\ x\ 60{,}000}{\sqrt{2500}}$ x 1.3 = 28.1" (assuming Class B splice)

Extend ftg. dowels 30" high.

Check Shear

$$V_u = \frac{30\ x\ 7.5^2}{2}\ x\ 1.6 + 200\frac{30}{110}\ \ x\ 7.5\ x\ 1.6 = 2005^{\#}$$

$$v = \frac{2005}{12\ x\ 5.63} = 29.7 < 75\ psi$$

Stability Check

Item	Wt.		arm		+M	-M
Stem $\frac{8}{12}$ x 150 x 9	900	x	.67	=	600	
Earth 5 x 110 x 9	4950	x	3.5	=	17325	
Surcharge 5 x 200	1000	x	3.5	=	3500	
Axial DL	200	x	$\frac{1}{12}$	=	17	
Axial LL	300	x	$\frac{1}{12}$	=	25	
Footing 6 x 1.17 x 150	1053	x	3.0	=	3159	
OTM earth = $\frac{30 \times 10.17^2}{2}$			$\frac{10.17}{3}$	=		5259
Surcharge = $200 \times \frac{30}{110} \times 10.17$			$\frac{10.17}{2}$	=		2821
	8403				24,626	8080

w/o axial LL = 8103

$\bar{x}$ (from front edge of footing) = $\frac{24{,}626 - 8080}{8403} = 1.97'$

e = $\frac{6}{2}$ - 1.97 = 1.03' = 12.36" Middle $\frac{1}{3} = \frac{6}{6} = 1.00$ ft. = 12.0 in.

∴ Slightly outside middle third

Soil $\rho = \frac{8403}{.75 \times 6 - 1.5 \times 1.03} = 2829$ psf

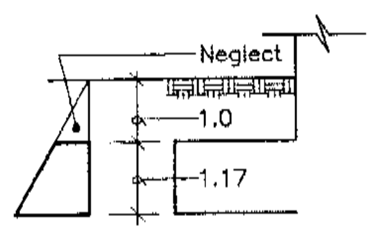
OTM ratio (w/o axial LL) = $\frac{24{,}626 - 25}{8080} = 3.04$

NOTE: If surcharge is live load it should be excluded from overturning and sliding resistance and lateral pressure reduced accordingly.

<u>Check Sliding</u>

<u>Total lateral</u> = $\frac{30 \ x \ 10.17^2}{2} + 200 \ x \frac{30}{110} \ x \ 10.17 = 2106$

Passive resistance



$= \frac{300 \text{ x } 2.17^2}{2} - \frac{300 \text{ x } 1^2}{2} = 556 \text{ lbs.}$

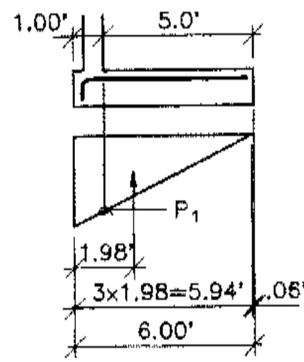
Friction resistance (w/o axial LL)

$= \ 8103 \text{ x } 0.40 = 3241^{\#}$

Sliding factor of safety = $\frac{556 + 3241}{2106} = 1.80$

OK > 1.5

<u>Check Heel</u>



Factored total vert. load = 8403 x 1.2 + 300 x 1.6 = 10,204#

Soil p (using e = 1.02') = $\frac{10,204}{.75 \ x \ 6 - 1.5 \ x \ 1.02}$

$= \ 3456$ psf

Soil p @ $P_1 = \frac{4.94}{5.94}$ x 3456 = 2874 psf

$M_u\downarrow \ = \ (1.17 \text{ x } 150 \text{ x } 1.2 + 110 \text{ x } 9 \text{ x } 1.2 + 200 \text{ x } 1.2) \text{ x } \frac{5^2}{2} = 20{,}483^{\#}$

$M_u\uparrow \ = \ \frac{2874 \ x \ 4.94}{2} \ x \ \frac{4.94}{3} = 11{,}689^{\#}$

$M_{design} \ = \ 20{,}483 - 11{,}689 = 8794^{\#}$

A_s (w/ d = 11.5") = 0.164 sq. in. $\rho_{min} = \frac{200}{f_y} = .0033$

A_s min required = .0033 x 12 x 11.5 = 0.455 sq. in. or 1.33 x .164 = 0.219

Use #5 e 9" to match stem dowels.

$A_s = \frac{0.31}{.75} = 0.41 > 0.219$

Check Shear @ Heel

V_u @ face of stem = (1.17 x 150 x1.2 + 110 x 9 x 1.2 + 200 x 1) x 5

= 8593#

$$v = \frac{8593}{12 \ x \ 11.5} = 62.2 \text{ psi} < 2\varphi\sqrt{f_c'} = 75 \quad \text{OK}$$

Extension of top bar from stem face

$$= \frac{0.02 \, x \, 0.625 \, x \, 60{,}000 \, x \, 0.7}{\sqrt{2500}} = 10.5$$

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Title EX-3
Job # :
Description...

Dsgnr:

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Date: JUL 12,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8056
Registration #: RP-11101 5 RP9.00
Licensed to: [The company name goes here]

Cantilevered Retaining Wall Design

Code: IBC 2006

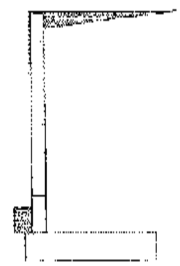
Criteria

Retained Height	=	9.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Vertical component of active
lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe		110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Thumbnail

Surcharge Loads

Surcharge Over Heel	=	200.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
NOT Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.000

Axial Load Applied to Stem

Axial Dead Load	=	200.0 lbs
Axial Live Load	=	300.0 lbs
Axial Load Eccentricity	=	7.0 in

Design Summary

Wall Stability Ratios			
Overturning	=	3.04	OK
Sliding	=	1.81	OK
Total Bearing Load	=	8,427 lbs	
...resultant ecc.	=	12.74 in	
Soil Pressure @ Toe	=	2,905 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	3,528 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	62.2 psi	OK
Allowable	=	75.0 psi	
Sliding Calcs (Vertical Component NOT Used)			
Lateral Sliding Force	=	2,105.0 lbs	
less 100% Passive Force	= -	554.2 lbs	
less 100% Friction Force	= -	3,253.6 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd
		Stem OK	Bar Lap/Emb
Design Height Above Ftg	ft =	1.50	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	=	8.00	8.00
Rebar Size	=	# 6	# 6
Rebar Spacing	=	18.00	9.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.897	0.751
Total Force @ Section	lbs =	2,004.5	2,705.5
Moment....Actual	ft-# =	6,249.5	9,778.5
Moment.....Allowable	ft-# =	6,968.1	13,022.4
Shear.....Actual	psi =	29.7	40.1
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	5.63	5.63
LAP SPLICE IF ABOVE	in =	25.18	21.09
LAP SPLICE IF BELOW	in =	25.18	
HOOK EMBED INTO FTG	in =		7.56
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

DESIGN EXAMPLE 3

Report Printout

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Title : EX-3
Job # :
Description...

Dsgnr:

Page: ______
Date: JUL 12,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Cantilevered Retaining Wall Design

Code: IBC 2006

Footing Dimensions & Strengths

Toe Width	=	0.33 ft
Heel Width	=	5.66
Total Footing Width	=	5.99
Footing Thickness	=	14.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
fc = 2,500 psi	Fy =	60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm=	3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	3,528	0 psf
Mu' : Upward	=	188	0 ft-#
Mu' : Downward	=	19	0 ft-#
Mu: Design	=	170	9,779 ft-#
Actual 1-Way Shear	=	0.00	62.16 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 5 @ 18.00 in	
Heel Reinforcing	=	# 5 @ 18.00 in	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: #4@ 9.50 in, #5@ 14.75 in, #6@ 20.75 in, #7@ 28.25 in, #8@ 37.00 in, #9@ 47
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		OVERTURNING Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	1,550.4	3.39	5,254.2
Surcharge over Heel	=	554.5	5.08	2,818.9
Toe Active Pressure	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	2,105.0	O.T.M. =	8,073.1

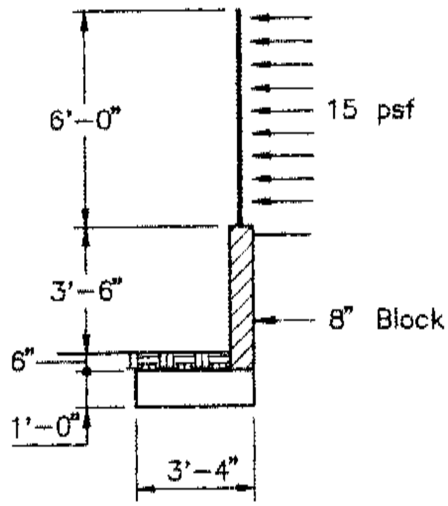
Resisting/Overturning Ratio = 3.04
Vertical Loads used for Soil Pressure = 8,425.6 lbs

		RESISTING Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	4,943.4	3.49	17,268.8
Sloped Soil Over Heel	=			
Surcharge Over Heel	=	998.7	3.49	3,488.7
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	200.0	0.08	16.0
* Axial Live Load on Stem	=	300.0	0.08	24.0
Soil Over Toe	=	36.3	0.17	6.0
Surcharge Over Toe	=			
Stem Weight(s)	=	900.0	0.66	597.0
Earth @ Stem Transitions	=			
Footing Weight	=	1,048.8	3.00	3,139.5
Key Weight	=			
Vert. Component	=			
Total	=	8,128.0 lbs	R.M.=	24,516.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

Design Data



Code: IBC '06

Equivalent fluid press. = 30 pcf

Wind on fence = 15 psf

Soil bearing = 1500 psf

Passive – 350 pcf

Soil density = 110 pcf

μ = 0.45 – Footing friction coeff.

f'_m = 1500

Use LRFD method

f_y = 60,000

f'_c = 2500

f'_y = 60,000

Neglect soil over toe for passive

Check Fence

$$M \text{ @ base} = \frac{15 \times 6^2}{2} = 270'^{\#} \text{ (for design of fence connection to wall)}$$

Lateral @ bott. of fence = 15 x 6 = 90 plf

Check stem @ base (Ht = 0.0)

Use LRFD Design Method

(Load factor wind and earth pressure = 1.6)

$$M_u = \left[15 \times 6\left(\frac{6}{2} + 4\right) + \frac{30 \times 4^2}{2} \times \frac{4}{3}\right] \times 1.6 = 1520 \text{ ft-lbs.}$$

Use 8" block solid grouted

f_b = 1500 x .33 = 500 psi

Try #5 @ 24 in. (A_s = 0.31/2.0 = 0.155)

d = 5.25"

$$a = \frac{0.155 \, x \, 60{,}000}{0.80 \, x \, 1500 \, x \, 0.80 \, x \, 12}$$

$\phi M_n = 0.90 \text{ x } 0.155 \text{ x } 60{,}000 \, [5.25 - (0.81/2)] \text{ x } \underline{1/12} = 3379$ ft-lbs.

Stress ratio $= \dfrac{1520}{3379} = 0.45$

<u>Stem Shear</u>

Lateral @ base $= \underline{\left(15 \, psf \; x \; 6' + \dfrac{30 \;\; x \;\; 4^2}{2} \right) x \, 1.6 = 528 \text{ lbs.}}$

$v_u = \dfrac{528}{12 \;\; x \;\; 5.25} = 8.38 \text{ psi} < \sqrt{750} \quad 27.4 \quad \text{OK}$

$\phi \upsilon_n \text{ x } 0.80 \, (4.0 - 1.75) \, \underline{\sqrt{1500}} = 69.7$ psi

<u>Embedment into ftg. w/ std. hook</u>

$$= \frac{.02 \, x \, 60{,}000 \, x \, 0.625 \, x \, 0.7}{\sqrt{2500}} \quad 10.5'' \quad \text{Use 6'' min.}$$

Min. ftg. Thickness = 10.5 + 3.0 = 13.5" <u>Use 14</u>"

Stability

Item		Wt.	arm	+M	-M
Fence		-0-			
8" stem	4 x 78 psf	312	3.0	936	
Earth over toe	0.5 x 110 x 2.67	147	1.33	195	
Footing	1.17 x 150 x 3.33	584	1.67	975	
Wind OTM	15 x 6'		8.17		735
Soil OTM	$= \dfrac{30 \ x \ 5.17^2}{2}$		1.72		690
		1043		2106	1425

$\bar{x} = \dfrac{2106 - 1425}{1043} = 0.66$ $\qquad e = \dfrac{3.33}{2} - .66 = 1.0' = 12"$

Middle third $e = \dfrac{3.33}{6} = .55$ $\qquad \therefore$ outside middle third

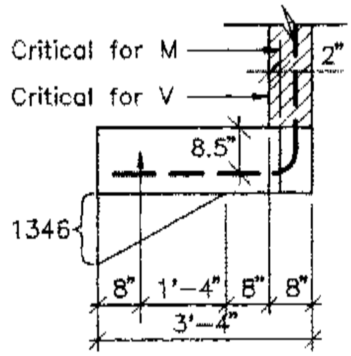
Soil p= $\dfrac{1043}{.75 \ x \ 3.33 - 1.5 \ x \ 1.0} = 1043$ psf

Overturning ratio = $\dfrac{2106}{1425}$ x 1.48 (consider OK because of wind load)

Check Sliding

Total lateral = 6' x 15 psf + $\dfrac{30 \ x \ 5.17^2}{2}$ = 490 lbs.

Friction resistance = 1043 x .45 = 469 lbs.



Passive resistance = $\frac{350\ x\ 1.67^2}{2} - \frac{350\ x\ .5^2}{2}$

= 444 lbs.

Sliding ratio = $\frac{444\ +\ 469}{490}$ = 1.86 > 1.5 OK

Check Toe

Total factored vertical load = 1067 x 1.2 = 1280 lbs.

Factored soil p = $\frac{1280}{.75\ x\ 3.33 - 1.5\ x\ 1}$ = 1280 psf

$M_u\uparrow$ = $\frac{1280\ x\ 2}{2}$ (3.33 - .67 - .50) = 2765 ft-lbs.

$M_u\downarrow$ = (150 x 1 + 110 x .5) 2.67 x 1.33 x 1.2 = 874 ft-lbs.

M_{design} = 2765 - 874 = 1891 ft-lbs.

V_u = 1280 x 2 x $\frac{1}{2}$ - (150 x 1 + 110 x .5) 2.67 x 1.2 = 623 lbs.

v_u = $\frac{623}{12\ x\ 10.5}$ = 4.95 v_{allow} = 76 OK

Toe Reinforcing

M_u = 874 ft-lbs.

d = 14 - 3 - .5 = 10.5 in.

A_s required = .02 A_s min = $\frac{200}{f y}$ = .0033

Min. reinf. = .0033 x 12 x 10.5 = .0.416

or = 1.33 x .042 = .03

or = .0018 x 12 x 10.5 = 0.227

Select #5 @ 16 (to match stem dowel bars bent to toe)

$A_s = \frac{0.31}{1.33}$ = 0.23 > 0.227 OK

DESIGN EXAMPLE 4

Report Printout

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Title : EX-4
Job # : 704
Description....

Dsgnr:

Page: ______
Date: JUL 12,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8056
Registration #: RP-1110115 RP9.00
Licensed to: [The company name goes here]

Cantilevered Retaining Wall Design

Code: IBC 2006

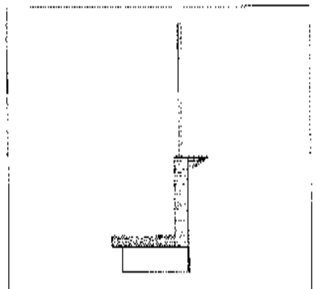
Criteria

Retained Height	=	4.00 ft
Wall height above soil	=	5.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Vertical component of active lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	350.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe		110.00 pcf
Footing||Soil Friction	=	0.450
Soil height to ignore for passive pressure	=	0.00 in



Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Stem	=	15.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	1.48 Ratio < 1.5!
Sliding	=	1.86 OK
Total Bearing Load	=	1,043 lbs
...resultant ecc.	=	12.16 in
Soil Pressure @ Toe	=	1,067 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,280 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	3.6 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	75.0 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	490.4 lbs
less 100% Passive Force	=	442.4 lbs
less 100% Friction Force	=	469.1 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

			Top Stem	2nd Stem OK
Design Height Above Ftg	ft =		4.00	0.00
Wall Material Above "Ht"	=		Fence	Masonry
Thickness	=			8.00
Rebar Size	=			# 5
Rebar Spacing	=			24.00
Rebar Placed at	=			Edge
Design Data				
fb/FB + fa/Fa	=			0.643
Total Force @ Section	lbs =		90.0	326.3
Moment....Actual	ft-# =		270.0	949.4
Moment.....Allowable	ft-# =			1,477.5
Shear.....Actual	psi =			5.1
Shear.....Allowable	=			38.7
Wall Weight	psf =			78.0
Rebar Depth 'd'	in =			5.25
LAP SPLICE IF ABOVE	in =			45.00
LAP SPLICE IF BELOW	in =			
HOOK EMBED INTO FTG	in =			5.75
Masonry Data				
f'm	psi =			1,500
Fy	psi =			60,000
Solid Grouting	=			Yes
Modular Ratio 'n'	=			21.48
Equiv. Solid Thick.	=			7.60
Masonry Block Type	=	Medium Weight		
Masonry Design Method	=	LRFD		
Concrete Data				
f'c	psi =			
Fy	psi =			

DESIGN EXAMPLE 4 — Report Printout

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949-721-4099

Title: EX-4
Job #: 704
Description....
Dsgnr:
Page: ____
Date: JUL 12,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Cantilevered Retaining Wall Design

Code: IBC 2006

Footing Dimensions & Strengths

Toe Width	=	2.66 ft
Heel Width	=	0.67
Total Footing Width	=	3.33
Footing Thickness	=	14.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
fc = 2,500 psi	Fy =	50,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm.=	3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,280	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	11 ft-#
Mu: Design	=	1,519	11 ft-#
Actual 1-Way Shear	=	3.64	0.02 psi
Allow 1 Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 5 @ 16.00 in	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

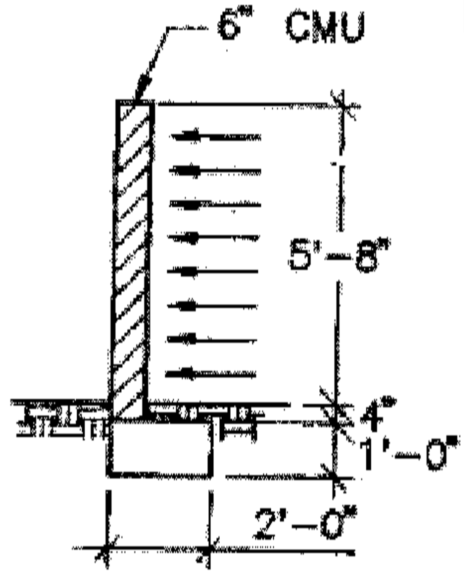
Item		....OVERTURNING.... Force lbs	Distance ft	Moment ft-#			....RESISTING.... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	400.4	1.72	689.6	Soil Over Heel	=	1.5	3.33	4.9
Surcharge over Heel	=				Sloped Soil Over Heel	=			
Toe Active Pressure	=				Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=				* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=	90.0	8.17	735.0	Soil Over Toe	=	146.3	1.33	194.6
					Surcharge Over Toe	=			
					Stem Weight(s)	=	312.0	2.99	933.9
					Earth @ Stem Transitions	=			
Total	=	490.4	O.T.M. =	1,424.6	Footing Weight	=	582.8	1.67	970.3
Resisting/Overturning Ratio	=	1.48			Key Weight	=			
Vertical Loads used for Soil Pressure	=	1,042.5 lbs			Vert. Component	=			
					Total	=	1,042.5 lbs	R.M.=	2,103.7

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

DESIGN EXAMPLE 5	**Page 1 of 3**

Freestanding Yard Wall



Design Data

Code: CBC 2007 (= IBC '06)

Assume California so seismic governs over wind.

Soil bearing = 2000 psf

Use 6" CMU solid grout

Equiv. fluid pressure = 30 pcf

Passive = 400 pcf

$\mu = 0.40$

$f'_m = 1500$ $\quad F_s = 24{,}000$

$f'_c = 2000$ $\quad f_y = 60{,}000$

Determine seismic force factor F_P/W_P

Per ASCE 7-05, 15.4.2 (Rigid nonbuilding structures)

From Hazard Maps for short period assume $S_s = 1.10$

Seismic Design Category D

$F_a = 1.0$

$S_{MS} = F_a\, S_s = 1.0 \times 1.10 = 1.10$

$S_{DS} = 0.67\ (1.10) = 0.73$ (Eq. 16-38)

Equation (15.4.5), rewritten

$F_p/W_p = 0.30\, S_{DS}\, I = 0.30 \times 0.73 \times 1.0 = 0.22$ (Assume $I = 1.0$)

Input $F_p/W_p = 0.22 \times \dfrac{1}{1.4} = 0.16$ for masonry ASD and overturning

Convert to ASD $\uparrow$

One-third stress increase permitted per IBC '09, 1605.3.2

Stem Design, Masonry

$$M = (0.16 \times 58 \text{ psf}) \frac{6^2}{2} = 167 \text{ ft-lbs.}$$

Use #4 c 48" @ center (d – 2.75")

$$A_s = \frac{.20}{4} = .05$$

$$np = \frac{21.5 \times .20}{48 \times 2.75} = .033 \qquad 2/kj = 9.57 \qquad j = .92$$

$$M_s = 24{,}000 \times 1.33 \times .92 \times 2.75 \times .05 \times \frac{1}{12} = 338^{\#} > 167 \quad \text{OK}$$

$$M_m = \frac{1500 \times .33 \times 12 \times 2.75^2}{9.57} \times 1.33 \times \frac{1}{12} = 525^{\#}$$

$$V = 0.16 \times 58 \times 6 = 55.7 \text{ lbs.}$$

$$v = \frac{55.7}{12'' \times 2.75} = 1.68 \text{ psi} \qquad \text{OK}$$

$$v\ (\text{Allow}) = \sqrt{1500} = 38.7 > 1.69 \quad \text{OK}$$

Check Stability

$$\text{Overturning} = 6.0 \times .16 \times 58 \left(\frac{6}{2} + 1\right) + \frac{30 \times 1.33^2}{2} \times \frac{1.33}{3} = 234 \text{ ft- lbs.}$$

Resisting moment = $6' \times 58 \text{ psf} \times .25 + 110 \times .33 \times 1.5 \times 1.25 + 1.0 \times 150 \times 2.0 \times 1.0$

$= 455^{\#}$

$$\bar{x} = \frac{455 - 234}{6 \times 58 + .33 \times 110 \times 1.5 + 2.0 \times 1 \times 150} = 0.32$$

$$W = 702 \text{ lbs.}$$

$$e = 2.0/2 - 0.32 = 0.68 \qquad \text{middle third} = 2.0/6 = .0.33$$

$\therefore$ outside middle third

Soil p= $\dfrac{702}{.75 \; x \; 2.0 - 1.5 \; x \; .68} = 1463$ psf < 3500 psf OK

Overturning ratio = $\dfrac{455}{234} = 1.94$ OK w/seismic

Lateral force @ base of ftg. = $.16 \times 58 \times 6 + (30 \times 1.33^2)/2 = 82.2$ lbs.

$$\text{Sliding ratio} = \frac{702 \; x \; .40 + \left(\dfrac{400 \; x \; 1.33^2}{2} - \dfrac{400 \; x \; .33^2}{2} \right)}{82.2} = 7.45$$

Check Heel Reinforcing

Neglect upward soil p

$M_u \downarrow$ = $(1.0 \times 150 + .33 \times 110) \dfrac{1.5^2}{2} \times 1.2 = 252^{\#}$

M_u stem = $167 \times 1.6 = 267$ ft-lbs. ← governs

d = 12 - 3 - .5 = 8.5"

A_s req'd = $.04 \; {}^{sq.in.}\!/_{ft.}$ (per CRSI equation)

$\rho_{min} = \dfrac{200}{fy} = .0033$

$\therefore A_s \min = .0033 \; x \; 12 \; x \; 8.5 = .34$ or $1.33 \; x \; .05 = .067$

Use #4 @ 48 (to match stem dowels) $A_s = \dfrac{.20}{4} = .05 < .067$ Consider OK

V_u = $(1.0 \; x \; 150 \; x \; 1.5 + .33 \; x \; 110 \; x \; 1.5) \; 1.2 = 335^{\#}$

v_u = $\dfrac{335}{12 \;\; x \;\; 8.5} = 3.3 < 76$ OK

DESIGN EXAMPLE 5

Report Printout

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Title : EX-5
Job # :
Description....

Dsgnr:

Page: ______
Date: JUL 24,2005

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 8 © 1989 - 2007 Ver: 8.00 8058
Registration #: RP-1110118 RP8.00
Licensed to: [The company name goes here]

Cantilevered Retaining Wall Design

Code: CBC 2007

Criteria

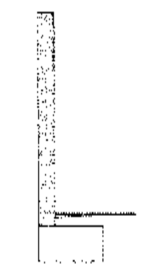
Retained Height	=	0.33 ft
Wall height above soil	=	5.67 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	4.00 in
Water height over heel	=	0.0 ft

Vertical component of active lateral soil pressure options:

NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	400.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe		110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	4.00 in



Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Stem Weight Seismic Load

F_p / W_p Weight Multiplier = 0.220 g

Added seismic base force 54.4 lbs

Design Summary

Wall Stability Ratios			
Overturning	=	1.99	OK
Sliding	=	7.59	OK
Total Bearing Load	=	702 lbs	
...resultant ecc.	=	8.14 in	
Soil Pressure @ Toe	=	1,456 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	2,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,745 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	2.9 psi	OK
Allowable	=	87.1 psi	
Sliding Calcs (Vertical Component NOT Used)			
Lateral Sliding Force	=	80.9 lbs	
less 100% Passive Force	= -	383.3 lbs	
less 100% Friction Force	= -	281.0 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Building Code	CBC 2007
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

			Top Stem
			Stem OK
Design Height Above Ftg	ft	=	0.00
Wall Material Above "Ht"		=	Masonry
Thickness		=	6.00
Rebar Size		=	# 4
Rebar Spacing		=	48.00
Rebar Placed at		=	Center
Design Data			
fb/FB + fa/Fa		=	0.481
Total Force @ Section	lbs	=	54.3
Moment....Actual	ft-#	=	163.1
Moment.....Allowable		=	339.1
Shear.....Actual	psi	=	1.6
Shear.....Allowable	psi	=	51.6
Wall Weight		=	58.0
Rebar Depth 'd'	in	=	2.75
LAP SPLICE IF ABOVE	in	=	24.00
LAP SPLICE IF BELOW	in	=	
HOOK EMBED INTO FTG	in	=	9.39
Masonry Data			
f'm	psi	=	1,500
Fs	psi	=	24,000
Solid Grouting		=	Yes
Modular Ratio 'n'		=	21.48
Short Term Factor		=	1.333
Equiv. Solid Thick.	in	=	5.60
Masonry Block Type		=	Medium Weight
Masonry Design Method		=	ASD
Concrete Data			
f'c	psi	=	
Fy	psi	=	

DESIGN EXAMPLE 5

Report Printout

Hugh Brooks, PE, SE
Retain Pro Software
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Title : EX-5
Job # :
Description....

Dsgnr:

Page: ______
Date: JUL 24,2009

This Wall in File: c:\program files\rp2007\examples.rp5

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Registration #: RP-1110115 RP9.00
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Cantilevered Retaining Wall Design

Code: CBC 2007

Footing Dimensions & Strengths

Toe Width	=	0.00 ft
Heel Width	=	2.00
Total Footing Width	=	2.00
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 2,000 psi   Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0010
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,748	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	2.94 psi
Allow 1-Way Shear	=	0.00	67.08 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings
Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		....OVERTURNING.... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	26.5	0.44	11.8
Surcharge over Heel	=			
Toe Active Pressure	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Seismic Stem Self Wt	=	54.4	4.00	217.4
Total	=	80.8	O.T.M. =	229.2

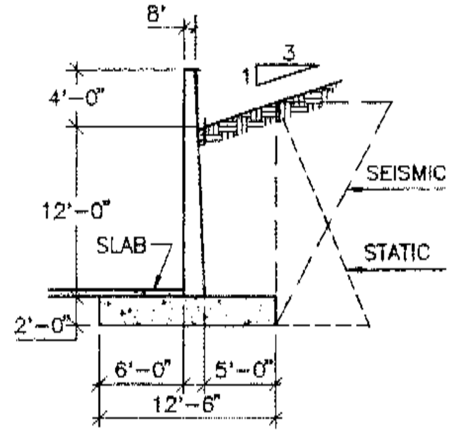
Resisting/Overturning Ratio = 1.99
Vertical Loads used for Soil Pressure = 702.5 lbs

		....RESISTING.... Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	54.5	1.25	68.1
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=			
Surcharge Over Toe	=			
Stem Weight(s)	=	348.0	0.25	87.0
Earth @ Stem Transitions	=			
Footing Weight	=	300.0	1.00	300.0
Key Weight	=			
Vert. Component	=			
Total =		702.5 lbs	R.M.=	455.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

DESIGN EXAMPLE 6



TAPERED CONCRETE STEM -

Include seismic effect

Assume lateral support at top of footing

Angle of internal friction

$\phi = 34°$

Wall friction angle

$= \delta = \varphi/2 = 17°$

Soil density = 110 pcf

f'_c = 3,000 psi f_y = 60,000 psi

Backfill slope = 3:1 = β = 18.4°

Wall friction angle assumed: $\delta = 17°$

Determine seismic factor, K_h

Assume high-seismic California

From charts in IBC for "short period", select $S_s = 1.274$

Then $S_{MS} = F_a S_s$

$F_a = 1.0$ (This is a function of soil characteristics and value of S_s. See Table 1615.1.2 in IBC).

$\therefore \quad S_{MS} = 1.0 \times 1.274 = 1.274$

$$S_{DS} = \frac{2}{3} S_{MS} = 0.667 \times 1.274 = 0.85$$

Per FEMA/NEHRP (Commentary) '03, Section 5.3.1

$$K_h = \frac{S_{DS}}{2.5} = 0.40 \times 0.85 = 0.34$$

Use $K_h = 0.34$ $\tan^{-1}\theta = 0.34 = 18.8°$

Determine static and seismic lateral pressures

Use Coulomb/Monokobe-Okabe equations

$$K_A = \frac{\sin^2(\phi+90)}{\sin(90-\delta\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(90-\delta)\sin(\beta+90)}}\right]^2}$$

$$= \frac{\sin^2(34+90)}{\sin(90\text{-}17)\left[1+\sqrt{\dfrac{\sin(34+17)\sin(34-18.4)}{\sin(90-17)\sin(18.4+90)}}\right]^2} = 0.328$$

K_A (horizontal) = $\cos\delta\, K_A = 0.96 \times .328 = 0.313$

$$K_{AE} = \frac{\sin^2(\phi + 90 - \theta)}{\cos\theta \sin(90 - \theta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta - \beta)}{\sin(90 - \delta - \theta)\sin(\beta + 90)}}\right]^2}$$

$$= \frac{\sin^2(34 + 90\text{-}18.8)}{\cos 18.8 \ \sin(90\text{-}18.8\text{-}17)\left[1 + \sqrt{\frac{\sin(34 + 17)\sin(34 - 18.8 - 18.4)}{\sin(90 - 17 - 18.8)\sin(18.4 + 90)}}\right]^2}$$

Because term under radical is zero

$K_{AE} = 1.21$

K_{AE} (horizontal) = $\cos\delta \, K_{AE} = 0.96 \times 1.21 = 1.16$

Static lateral at base of stem = $\frac{0.313 \, x \, 110 \, x \, 12^2}{2} \times 1.6 = 3966^{\#}$

Seismic portion of lateral = $\frac{(1.16 - .313)110 \, x \, 12^2}{2} = 6732^{\#}$

Static + seismic lateral at base of stem = 3966 + = 6732 = 10,698 #

$M_{static\ @\ base} = \frac{0.313 \, x \, 110 \, x \, 12^2}{2} \times \frac{12}{3} \times 1.6 = 15{,}865^{\prime\#}$

$M_{seismic}$ @ stem base assuming point of application = 0.6H

6732 x 0.60 x 12 = 48,470 '#

Mu @ stem base = 15,865 + 48,470 = 64,335 '#

Height to resultant of static and seismic forces

$$= \frac{15865 + 48{,}470}{10698} = 6.01 \text{ ft.} \cong \text{mid-height}$$

Check Base of Stem

M_{static} = 15,865 '#

$M_{seismic}$ = 48,470 x 1.0 = 48,470

Design factored moment = 15,865 + 48,470 = 64,335'#

Try 18" stem   d = 18.0 -2.5 = 15.5"

A_s required $= 0.17 \, f_c' d - \sqrt{.029\left(f_c' d\right)^2 - .0063 \, f_c' \, M_u}$

$= .17 \times 3.0 \times 15.5 - \sqrt{.029\,(3 \, x \, 15.5)^2 - .0063 \, x \, 3 \, x \, 64.335 \;\; x \, 12}$

$= 0.965 \ \frac{sq.\,in.}{ft.}$

Check for A_s min. · .0033 x 12 x 15.5 = 0.61

Use #8 @ 9" $A_s = \frac{1.0}{0.75} = 1.33 > 0.965$

V_u factored = 10,698#

$$v = \frac{10{,}698^{\#}}{12 \times 15.5} = 57.5 < .75 \times 2 \times \sqrt{3000} = 82.2 \text{ psi}$$

(Note: This example does not include seismic due to stem self-weight. If desired, this can be added as an "additional lateral load," using the appropriate seismic factor.)

Check stem at 4.0 ft. above ftg. Retained ht. = 8.0 ft.

t = 15.5" (by interpolation) d say 13.0"

Use same procedure as base of stem

M_u = 19,010'#

$$A_s \text{ req'd. @ 4' high} = 0.17 \times 3.0 \times 13.0 - \sqrt{.029\ (3 \times 13)^2 - .0063 \times 3 \times 19 \times 12}$$

$$= 0.0.32\ \frac{sq.\ in.}{ft.}$$

Use #7 @ 18" $A_s = \frac{0.61}{1.5} = 0.41 > 0.32$ OK

Stability and footing design:

Total ht. @ back of heel = 12.0 + 2.0 + (5./3) = 15.67'

(Assume slope starts aligned with stem at bottom).

$$\text{Static lateral @ bott. of footing} = \frac{.313 \times 110 \times 15.67^2}{2} = 4229\#$$

$$\text{Seismic lateral @ bott. of footing} = \frac{(1.16 - 0.31)\ 110 \times 15.67^2}{2} \times 0.71 = 8150^{\#}$$

Total lateral = 4229 + 8150 = 12,379 # (Converted to ASD)

Try 6'-0" toe, 6'-6" heel (incl. stem) = 12'-6" total width

$$\text{Overturning moment} = \frac{(1.16 - .313)\,110 \times 15.67^2}{2} \times .6 \times 15.67 \times 0.71 = 76{,}310 \text{ ft-lbs.}$$

To convert to ASD. ↑

$$+ \frac{.313 \times 110 \times 15.67^3}{2 \times 3} = 76{,}310 + 22{,}067 = 98{,}377 \text{ ft-lbs.}$$

Resisting Moment

(Use vertical comp. to resist overturning but not to reduce soil pressure).

Resisting moment = (soil over heel) (arm) + (sloped soil over heel) (arm) + (stem wt.) (arm) + (earth @ stem) (arm) + (ftg. wt.) (arm) + (vert. comp. @ back of heel) (arm)

= (5 x 110 x 12) (10) + 5.63 x 1.88 x .5 x 110) (10.63) + (16 x 1.08 x 150) x (6.57)

+ (12 x .63 x .5 x 110) (7.29) + (12.5 x 2.0 x 150) (6.25)

+ 4229 tan 17°) (12.5) 131,543 ft-lbs. (Vert. comp. =.4229 tan 17° 1293#

Total vert. load = (5 x 110 x 12) + (5.63 x 1.88 x .5 x 110) + (16 x 1.08 x 150)

+ (12 x .83 x .5 x 110) + (12.5 x 2 x 150) + 1293# = 18,365#

$$\text{Overturning ratio} = \frac{131{,}543}{98{,}377} = 1.34$$

Soil pressure: (Vert. Component not used)

$$\bar{x} = \frac{131{,}543 - 98{,}377 - (1293 \; x \; 12.5)}{15{,}365 - 1293} = 1.21 \text{ ft.}$$

Eccentricity = (12.5/2) - 1.21 = 5.04' = 60.5"

e for inside middle third = $^{12.5}\!/_{6}$ = 2.08 ∴ Outside middle third

$$\text{Soil pressure} = \frac{15{,}365 - 1293}{.75 \; x \; 12.5 \quad 1.5 \; x \; 5.04} = 7753 \text{ psf}$$

$$\text{Sliding S.F.} = \frac{0.4 \; x \; (15{,}365 - 1293)}{12{,}379} = 0.45$$

However, not applicable since slab is present and to be designed to resist lateral.

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Job # : 706
Description....

Dsgnr.

Page: ____
Date: JUL 24,20

This Wall in File: c:\program files\rp2007\examples.rp5

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Registration #: RP-1110 8 RP9.00
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Tapered Stem Concrete Retaining Wall Design

Code: CBC 200

Criteria

Retained Height	=	12.00 ft
Wall height above soil	=	4.00 ft
Slope Behind Wall	=	3.00 : 1
Height of Soil over Toe	=	0.00 in
Wind on Stem	=	0.0 psf

Vertical component of active lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	8,000.0 psf
Coulomb Soil Pressure calculation		
Soil Friction Angle	=	34.0 deg
Active Pressure:Ka*Gamma	=	34.5 psf/ft
Passive Pressure:Kp*Gamma		389.1 psf/ft
Soil Density	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	0.00 in

Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Earth Pressure Seismic Load

Design Kh	=	0.340 g

Kae for seismic earth pressure	=	1.159
Ka for static earth pressure	=	0.314
Difference, Kae - Ka	=	0.845

Added seismic base force 8,100.8 lbs

<<—— Note! These are horizontal components

Using Mononobe-Okabe / Seed-Whitman procedure

Stem Weight Seismic Load

F_p / W_p Weight Multiplier = 0.000 g

Added seismic base force 0.0 lbs

Design Summary

Total Bearing Load	=	13,943 lbs	
...resultant ecc.	=	61.05 in	
Soil Pressure @ Toe	=	7,998 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	8,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	9,313 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	57.2 psi	OK
Footing Shear @ Heel	=	47.8 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs Slab Resists All Sliding !
Lateral Sliding Force = 12,491.4 lbs

Tapered Concrete Stem Design Data

Thickness at TOP	=	8.00 in	Fy	=	60,000 psi
Thickness at BOTTOM	=	18.00 in	f'c	=	3,000 psi

Rebar Cover (rebar center to concrete face) 2.00 in

		@ Height #1	@ Height #2	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design Height Above Ftg	=	8.00 ft	4.00 ft	0.00 ft
Rebar Size	=	# 5	# 9	# 9
Rebar Spacing	=	12.00 in	12.00 in	6.00 in
Rebar Depth 'd'	=	10.50 in	13.00 in	15.50 in
Design Data				
Mu. ..Actual	=	2,374.5 ft-#	18,996.1 ft-#	64,112.0 ft-#
Mn * Phi.....Allowable	=	14,222.3 ft-#	54,075.0 ft-#	121800.0 ft-#
Shear Force @ this height	=	1,185.9 lbs	4,743.6 lbs	10,673.0 lbs
Vu.....Actual	=	9.41 psi	30.41 psi	57.38 psi
Vn * Phi.....Allowable	=	82.16 psi	82.16 psi	82.16 psi
Rebar Lap Required	=	21.36 in	48.05 in	
Hooked embedment into footing			=	17.25 in

Load Factors

Building Code	CBC 2007
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

DESIGN EXAMPLE 6 — Report Printout

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Title : EX-6
Job # : 706
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Page: ____
Date: JUL 24,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Tapered Stem Concrete Retaining Wall Design

Code: CBC 2007

Footing Strengths & Dimensions

Toe Width	=	6.00 ft
Heel Width	=	6.50
Total Footing Width	=	12.50
Footing Thickness	=	24.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 2.00 in @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	9,313	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	64	64 ft-#
Actual 1-Way Shear	=	57.22	47.84 psi
Allow 1-Way Shear	=	82.16	82.16 psi

Toe Reinforcing = # 4 @ 12.00 in
Heel Reinforcing = # 6 @ 12.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings
Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		OVERTURNING Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	4,390.7	5.31	23,336.0
Toe Active Pressure	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Seismic Load	=	8,100.6	9.40	76,145.8
Total	=	12,491.4	O.T.M. =	99,481.8

Resisting/Overturning Ratio = 1.33
Vertical Loads used for Soil Pressure = 13,942.6 lbs

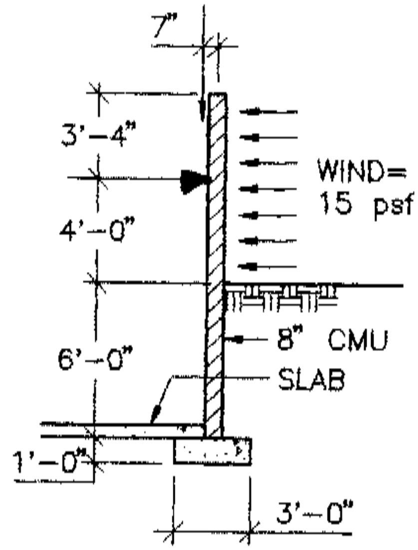
Item		RESISTING Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	6,600.0	10.00	66,000.0
Sloped Soil Over Heel	=	580.1	10.63	6,163.3
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=		0.00	
Soil Over Toe	=			
Surcharge Over Toe	=			
Stem Weight	=	2,600.0	6.57	17,077.8
Earth above Sloping Stem	=	412.5	7.29	3,007.6
Footing Weight	=	3,750.0	6.25	23,437.5
Key Weight	=			
Vert. Component	=	1,296.0	12.50	16,200.3
Total =		15,238.9 lbs	R.M.=	131,886.7

Vertical component of active pressure NOT used for soil pressure

DESIGNER NOTES:

RESTRAINED CMU WALL

Design Data



Code: IBC '06

Soil bearing = 2000 psf

Soil density = 110 pcf

EFP = 30 psf/ft.

Passive - (not appl. because of floor slab)

f'_m = 1500 psi

f_s = 24,000 psi

f_y = 60,000 psi

f'_c = 2500 psi

w = wind = 15 psf

P = 144 plf DL

e = 7.0 in.

Assume 100% fixity at base

Assume lateral restraint at top of footing to resist sliding

*Moments, Shear, and Reaction

R = reaction @ top restraint = 160 lbs. V at base = 685 lbs.

M @ top restraint = 144 (7 / 12) + 15 x $(3.33)^2$ / 2 = 167 '#

M @ base = 7488 in. lbs. = 624'#

+M Max. = 2592 in. lbs. = 216'# @ h = 5.84 ft.

Check stem @ base

M = 7488 in. lbs. = 624 ft. lbs.

* (Obtained from Single Span Beam Analysis program in Enercalc's Structural Engineering Library, Version 5.8.)

Try 8" CMU, #5 @ 32", d = 5.3", solid grouted, n = 21.5

f'_m = 1500 psi f_s = 24,000 f_b = .33 x 1500 = 500 psi

Use #5 @ 32" o.c. @ edge $np = \frac{21.5 \times .31}{32 \times 5.25} = 0.040$

$2/kj = 8.9$ j = 0.92

$$M_m = \frac{500 \times 12 \times 5.25^2}{8.9} \times \frac{1}{12} = 1548 \text{ ft. lbs.} > 624 \quad \text{OK}$$

$M_s = 24{,}000 \times \frac{.31}{2.67} x .92 x 5.25 x \frac{1}{12} = 1122'^{\#} > 624$

$V = 490$ lbs. $v = \frac{490}{12 x 5.25} = 7.78 < v_{allow} = 38.7$

Check @ Max. Positive Moment

+M = 2592 in. lbs. = 216 ft. lbs.

Use #5 @ 32" o.c. @ center

$np = \frac{21.5 x 0.31}{32 x 3.75} = 0.056$ $\quad 2/kj = 7.80 \quad j = 0.91$

$M_m = \frac{500 x 12 x 3.75^2}{7.80} x \frac{1}{12} = 901$ ft. lbs. > 216 OK

$M_s = 24{,}000 \times \frac{0.31}{2.67} \times 0.91 \times 3.75 \times \frac{1}{12} = 792$ ft. lbs. > 216

Check Moment @ Lateral Support

M = 167 '#

Use #5 @ 32" o.c. @ center

OK per above analysis for positive mid-height moment.

Soil Bearing

Embedment of hooked bar in footing $= \frac{0.02 x 60{,}000 x .625 x 0.7}{\sqrt{2000}} x \frac{624}{1122} = 6.5''$

Min. ftg. thickness = 6.6 + 3.0 = 9.6 in.

Try 3'-0" wide ftg. Centered under stem Use 12" thick > 9.6"

Total vertical load = 13.33' x 78 psf + 6' x 1.17 x 110 + 3.0 x 1.0 x 150 + 144 (DL only)

= $2406^{\#}$ M @ base = $624'^{\#}$

Distance from toe to centroid of soil pressure

$$= \frac{13.33' x 78 x 1.5 + 1.17 x 6 x 110 x 2.42 + 3' x 1' x 150 x 1.5 + 144 x .92 - 624}{2406}$$

= 1.50 ft. $\quad e = \frac{3.0}{2} - 1.50 = 0.0$ ft. (in middle third)

Soil pressure $= \frac{2406}{3.0} \pm \frac{2406 x .00 x 6}{3^2} = 802 \pm 0 = 802$ psf max. (uniform)

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Date: AUG

This Wall in File: c:\program files\rp2007\examples.r

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Registration #: RP-11101 5 RP9.03
Licensed to: [The company name goes here]

Restrained Retaining Wall Design

Code: IBC

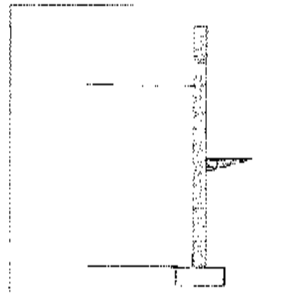
Criteria

Retained Height	=	6.00 ft
Wall height above soil	=	7.33 ft
Total Wall Height	=	13.33 ft
Top Support Height	=	10.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Water height over heel	=	0.0 ft
Wind on Stem	=	16.0 psf

Vertical component of active lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	250.0 psf/ft
Soil Density	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	0.00 in



Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	144.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	7.0 in

Uniform Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Earth Pressure Seismic Load K_h Soil Density Multiplier = 0.200 g Added seismic per unit area = 0.0 psf

Stem Weight Seismic Load F_p / W_p Weight Multiplier = 0.000 g Added seismic per unit area = 0.0 psf

Design Summary

Total Bearing Load	=	2,402 lbs
...resultant ecc.	=	0.01 in
Soil Pressure @ Toe	=	800 psf OK
Soil Pressure @ Heel	=	801 psf OK
Allowable	=	2,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	960 psf
ACI Factored @ Heel	=	961 psf
Footing Shear @ Toe	=	6.9 psi OK
Footing Shear @ Heel	=	0.1 psi OK
Allowable	=	67.1 psi
Reaction at Top	=	160.3 lbs
Reaction at Bottom	=	684.7 lbs

Sliding Calcs Slab Resists All Sliding !
Lateral Sliding Force = 684.7 lbs

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Masonry Stem Construction

Thickness = 8.00 in fm = 1,500 psi Short Term Factor = 1.0
Wall Weight = 78.0 psf Fs = 24,000 psi Equiv. Solid Thick. = 7.6
Stem is FIXED to top of footing n Ratio (Es/Em) = 21.4
Block Type = Medium Weight
Design Method = ASD
Solid Grouted

		@ Top Support	Mmax Between Top & Base	@ Base of
		Stem OK	Stem OK	Stem O
Design Height Above Ftg	=	10.00 ft	4.18 ft	0.00
Rebar Size	=	# 5	# 5	# 5
Rebar Spacing	=	32.00 in	32.00 in	32.00
Rebar Placed at	=	Center	Center	Edge
Rebar Depth 'd'	=	3.75 in	3.75 in	5.25
Design Data				
fb/FB + fa/Fa	=	0.212	0.273	0.557
Moment....Actual	=	167.2 ft-#	215.6 ft-#	623.9
Moment.....Allowable	=	789.8 ft-#	789.8 ft-#	1,121.1
Shear Force @ this height	=	110.3 lbs		459.7
Shear.....Actual	=	2.17 psi		6.81
Shear.....Allowable	=	38.73 psi		38.73
Rebar Lap Required	=	25.00 in	25.00 in	

Hooked embedment into footing (w/ stress level reduction) = 6.00 in

Other Acceptable Sizes & Spacings:
Toe: # 7 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: # 8 @ 18.00 in -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No Key defined

DESIGN EXAMPLE 7

Report Printout

Hugh Brooks, PE, SE
Retain Pro Software
PO Box 826 Corona del Mar, CA 92625
hbrooks@retainpro.com
949-721-4099

Title : EX-7
Job # : 707
Description....
Dsgnr:
Date AUG 8,2009
Page ____

This Wall in File: c:\program files\rp2007\examples.rp5

Restrained Retaining Wall Design

Code: IBC 2006

Footing Strengths & Dimensions

Toe Width	=	1.17 ft
Heel Width	=	1.83
Total Footing Width	=	3.35
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 2,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 2.00 in @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	960	961 psf
Mu' : Upward	=	0	850 ft-#
Mu' : Downward	=	0	860 ft-#
Mu: Design	=	64	10 ft-#
Actual 1-Way Shear	=	8.95	0.11 psi
Allow 1-Way Shear	=	67.08	67.08 psi

Summary of Forces on Footing : Slab RESISTS sliding, stem is FIXED at footing

Forces acting on footing for soil pressure

>>> Sliding Forces are restrained by the adjacent slab

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem	=			-623.9 ft-#
Surcharge Over Heel	=	lbs	ft	ft-#
Adjacent Footing Load	=	lbs	ft	ft-#
Axial Dead Load on Stem	=	144.0 lbs	0.92 ft	132.5 ft-#
Soil Over Toe	=	lbs	ft	ft-#
Surcharge Over Toe	=	lbs	ft	ft-#
Stem Weight	=	1,039.7 lbs	1.50 ft	1,563.1 ft-#
Soil Over Heel	=	757.8 lbs	2.42 ft	1,856.8 ft-#
Footing Weight	=	450.0 lbs	1.50 ft	675.0 ft-#
Total Vertical Force	=	2,401.5 lbs	Base Moment =	3,603.4 ft-#

Soil Pressure Resulting Moment = -4. ft-#

DESIGNER NOTES:

RESTRAINED CONCRETE WALL

Code: IBC '06

Tie-back @ 16 ft. high

Use EFP = 40 pcf

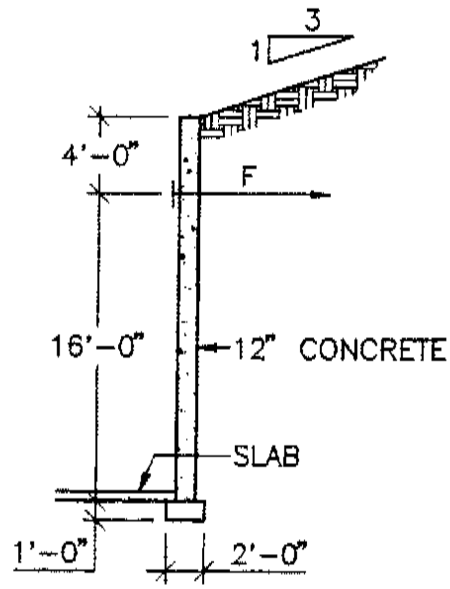
Backfill slope = 3:1

Soil bearing = 3,000 psf

f'_c = 3,000 psi f_y = 60,000 psi

Slab lateral restraint @ base

Assume stem "pinned" at footing



Reactions:

$W = \dfrac{40 \, x \, 20^2}{2} = 8{,}000$ lbs.

R @ tie-back = $\dfrac{8000 \, x \left(\frac{20}{3}\right)}{16}$

= 3333 $lbs./ft.$

R @ base = $[40 \times (20 + 1)^2 / 2] - 3333$
= 5487 lbs.

Moments:

Dist. to max mom where V = 0: = 7.09' by statics

Max. pos. moment = 15,250'# x 1.6 = M_u = 24,400'#

Design @ max. positive moment

M_u = 24,400'#   Try 12"   (d = 10")

A_s required = $.17 \, x \, 3 \, x \, 10 - \sqrt{.029 \, (3 \, x \, 10)^2 \quad .0063 \, x \, 3 \, x \, 24.4 \, x \, 12}$

= 0.57 sq. in./ft.

<u>Use #7 @ 9" o.c.</u> $A_s = \dfrac{.60}{.75} = 0.80 > 0.57$

Design moment at support = 426'# x 1.6 = 681'#

Use min. vert. reinf. @ center throughout   O.K.B.I.

Min. vert. reinf. = $200/f_y$ = .0033 sq. in.

#6 @ 18" o.c. @ center vert. $A_s = .44/(6 \, x \, 18) = .0041 > .0033$

Total vert. load to ftg.

= 20 x 150 + .50 x 20 x 110 = 4100 lbs.

Try 2'-0" wide ftg. Assume "pin" connection wall to footing.

Moments about front edge of ftg.

$$= (20 \text{ x } 150)\ 1.0 + (.50 \text{ x } 20 \text{ x } 110)\ 1.75 + (2 \text{ x } 150 \text{ x } 1.0)$$

$$= 5225^{\#}$$

$$\text{Ecc.} = \left[(5225/4400) - (2/2)\right] \text{ x } 12 = 2.25''$$

$$\text{Mom. @ stem-ftg. interface due to ecc.} = 4400 \text{ x } \frac{2.25}{12} = 825 \text{ ft. lbs.}$$

Allow. Mom. @ stems - ftg. interface:

$$a = \frac{.31 \, x \, 60{,}000}{1.5 \, x \, 0.85 \, x \, 3000 \, x \, 12} = 0.41$$

$$M_u = \frac{.31}{1.5} \text{ x } 60{,}000 \left(6 - \frac{0.40}{2}\right) \text{x} \frac{1}{12} = 5993 \text{ ft. lbs.}$$

Since base of stem allow. moment exceeds mom. due to ftg. eccentricity, soil pressure is uniform $= \dfrac{4400}{2} + 1 \text{ x } 150 = 2350$ psf.

Note: If stem ftg. mom. < 825 ft. lbs., then:

$$\text{Soil pressure} = \frac{4400}{2} + \frac{4400 \ x \ \left(2.25/12\right) \ x \ 6}{2^2}$$

$$= 2200 + 1238$$

$$= 962 \text{ psf @ toe}$$

$$= 3438 \text{ psf @ heel}$$

Note: Check slab for this lateral force of 5487 lbs. – usually resisted by sliding friction.

DESIGN EXAMPLE 8 — Report Printout

Hugh Brooks, PE, SE
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949-721-4099

Title : EX-8
Job # : 708
Dsgnr:
Description....
Page: ____
Date: AUG 8,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 5.00 8058
Registration #: RP-1110116 RP9.00
Licensed to: [The company name goes here]

Restrained Retaining Wall Design

Code: IBC 2006

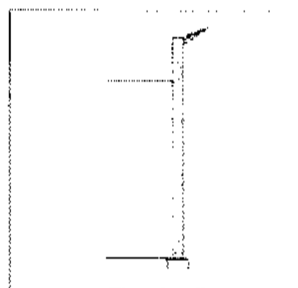
Criteria

Retained Height	=	20.00 ft
Wall height above soil	=	0.00 ft
Total Wall Height	=	20.00 ft
Top Support Height	=	16.00 ft
Slope Behind Wall	=	3.00 : 1
Height of Soil over Toe	=	0.00 in
Water height over heel	=	0.0 ft
Wind on Stem	=	0.0 psf

Vertical component of active
Lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	3,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	389.0 psf/ft
Soil Density	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Earth Pressure Seismic Load

K_h Soil Density Multiplier = 0.200 g   Added seismic per unit area = 0.0 psf

Stem Weight Seismic Load

F_p / W_p Weight Multiplier = 0.000 g   Added seismic per unit area = 0.0 psf

Design Summary

Total Bearing Load	=	4,400 lbs
...resultant ecc.	=	2.25 in
Soil Pressure @ Toe	=	2,200 psf OK
Soil Pressure @ Heel	=	2,200 psf OK
Allowable	=	3,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,155 psf
ACI Factored @ Heel	=	4,125 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	4.0 psi OK
Allowable	=	82.2 psi
Reaction at Top	=	3,333.3 lbs
Reaction at Bottom	=	5,486.7 lbs

Sliding Calcs Slab Resists All Sliding !
Lateral Sliding Force = 5,486.7 lbs

Concrete Stem Construction

Thickness = 12.00 in   Fy = 60,000 psi
Wall Weight = 150.0 psf   f'c = 3,000 psi
Stem is FREE to rotate at top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design Height Above Ftg	=	16.00 ft	7.07 ft	0.00 ft
Rebar Size	=	# 5	# 7	# 5
Rebar Spacing	=	18.00 in	9.00 in	18.00 in
Rebar Placed at	=	Center	Edge	Center
Rebar Depth 'd'	=	6.00 in	10.00 in	6.00 in
Design Data				
fb/FB + fa/Fa	=	0.127	0.741	0.000
Mu....Actual	=	682.7 ft-#	24,568.7 ft-#	0.0 ft-#
Mn * Phi.... Allowable	=	5,391.0 ft-#	33,168.8 ft-#	5,391.0 ft-#
Shear Force @ this height	=	4,821.3 lbs		7,466.7 lbs
Shear....Actual	=	66.96 psi		103.70 psi
Shear .... Allowable	=	82.16 psi		82.16 psi
Rebar Lap Required	=	21.33 in	37.38 in	
Hooked embedment into footing			=	9.69 in

Other Acceptable Sizes & Spacings:
Toe: None Spec'd -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.300
Seismic, E	1.000

DESIGN EXAMPLE 8 — Report Printout

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949-721-4099

Title : EX-8
Job # : 708
Dsgnr:
Description....

Page: ____
Date: AUG 8,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Restrained Retaining Wall Design

Code: IBC 2006

Footing Strengths & Dimensions

Toe Width	=	0.50 ft
Heel Width	=	1.50
Total Footing Width	=	2.00
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 3,000psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 2.00 in @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,155	4,125 psf
Mu' : Upward	=	175	485 ft-#
Mu' : Downward	=	23	354 ft-#
Mu: Design	=	153	139 ft-#
Actual 1-Way Shear	=	0.00	4.05 psi
Allow 1-Way Shear	=	82.16	82.16 psi

Summary of Forces on Footing : Slab RESISTS sliding, stem is PINNED at footing

Forces acting on footing soil pressure

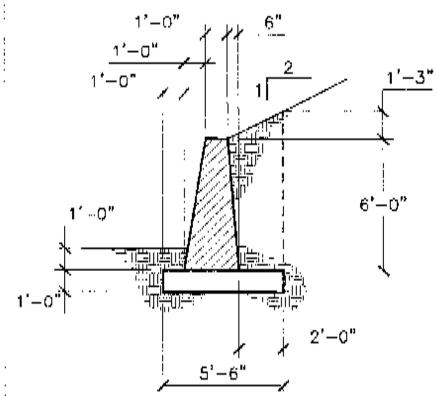
(taking moments about front of footing to find eccentricity)

Surcharge Over Heel	=	lbs	ft	ft-#
Axial Dead Load on Stem	=	lbs	ft	ft-#
Soil Over Toe	=	lbs	ft	ft-#
Adjacent Footing Load	=	lbs	ft	ft-#
Surcharge Over Toe	=	lbs	ft	ft-#
Stem Weight	=	3,000.0lbs	1.00 ft	3,000.0ft-#
Soil Over Heel	=	1,100.0lbs	1.75 ft	1,925.0ft-#
Footing Weight	=	300.0lbs	1.00 ft	300.0ft-#
Total Vertical Force	=	4,400.0lbs	Moment =	5,225.0ft-#

Net Mom. at Stem/Ftg Interface = -825.0 ft-#
Allow. Mom. @ Stem/Ftg Interface = 3,363.4 ft-#
Allow. Mom. Exceeds Applied Mom.? Yes
Therefore Uniform Soil Pressure = 2,200.0 psf

DESIGNER NOTES:

Gravity Wall:



Retained height = 6.0 ft.
Wt. of rubble masonry = 145 pcf
Allow. comp. = 100 psi no tension > 10 psi
Allow. passive = 300 pcf
Soil bearing = 2000 psf
Cohesion = 200 psf
Width of stem base = 30 in.

$$S \text{ @ base} = \frac{12 \, x \, 30^2}{6} = 1800 \, in.^3$$

Backfill slope = 2:1
Soil density = 110 pcf
EFP = 43 pcf

$$M_{base} = \frac{43 \, x \, 6^3}{2 \, x \, 3} = 1548 \, ft - lbs.$$

Wt. of stem above base (rubble + earth over back face)

= 145 (1 x 6 + 1 x 6 x 0.5 + 0.5 x 6 x 0.5) + 0.5 x 6.0 x 0.5 x 110
= 1688 lbs.

Stress @ base: $= \dfrac{1688}{12 \, x \, 30} \pm \dfrac{1548 \, x \, 12}{1800}$

$= 4.69 \pm 10.32$

Max. comp = 15.01 psi
Max. tension = 5.63 psi OK

$$v \text{ @ base} = \frac{43 \, x \, 6^2 \, x \, 0.5}{12 \, x \, 30} = 2.15 \, psi \quad \text{OK}$$

Check @ 2'-0" height above base
Thickness = 24" S = 1152 in^2

$$M \text{ @ 2' high} = \frac{43 \, x \, 4^3}{2 \, x \, 3} = 459 \, ft - lbs.$$

Wt. above 2' = 145 (1 x 4 + .67 x 4 x 0.5 + .33 x 4 x 0.5) = 870 lbs.

$$\text{Stress @ 2'} = \frac{870}{12 \, x \, 24} \pm \frac{459 \, x \, 12}{1152}$$

$= 3.02 \pm 4.78$

Max. comp. = 7.8 max. tension = 1.76 OK

DESIGN EXAMPLE 9	**Page 2 of 3**

Resisting Moments:

		W		X		M	
Stem:							
1 x 6 x 145	=	870	x	2.50	=	2,175	ft. lbs.
+ 1 x 6 x 0.5 x 145	=	435	x	1.67	=	726	
+ 0.5 x 6 x 0.5 x 145	=	218	x	3.17	=	691	
Soil:							
1 x 1 x 110	=	110	x	0.50	=	55	
+ 0. 5 x6 x 0.5 x 110	=	165	x	3.33	=	549	
+ 2 x 6 x 110	=	1,320	x	4.50	=	5,940	
+ 1.25 x 2.5 x 0.5 x 110 =172			x	4.67	=	803	
Footing:							
1 x 5.5 x 150		825	x	2.75	=	2,269	
		4,115 lbs.				3,208	ft.-lbs.

$$\text{Overturning M} = \frac{43\,(1.25 + 6.0 + 1.0)^3}{2\,x\,3} - \frac{30\,x\,2^3}{2\,x\,3} = 3984\ ft - lbs.$$

$$\text{Overturning S. F.} = \frac{13{,}208}{3984}\ 3.32 > 1.5 \text{ OK}$$

Dist. toe to soil pressure c.g.

$$\frac{13{,}208 \quad 3984}{4115} = 2.24 \text{ ft.}$$

$$e = \frac{5.50}{2} - 2.24 = 0.50$$

within middle third

$$\text{Soil pressure} = \frac{4115}{5.5} \pm \frac{6 \, x \, 4115 \, x \, 0.50}{5.5^2}$$

$$= 748 + 408 = 1156 \text{ psf} < 3500 \text{ OK}$$

Check sliding:

$$\text{Net lateral force} = \frac{43\,(1.25 + 6.0 + 1.0)^2}{2} - \frac{30 \, x \, 2^2}{2} = 1403 \, lbs.$$

Cohesion resistance - $5.5 \, x \, 200 = 1100$ lbs.

$$\text{Passive resistance} = \frac{300 \, x \, 2^2}{2} = 600$$

$$\text{Sliding S.F.} = \frac{600 + 1100}{1403} = 1.21 < 1.5 \text{ Consider key}$$

DESIGN EXAMPLE 9

Report Printout

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949-721-4099

Title : EX-9
Job # : 709
Description...

Dsgnr:

Page: ____
Date: AUG 8,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8056
Registration #: RP-11101 5 RP9.00
Licensed to: [The company name goes here]

Gravity Stem Retaining Wall Design

Code: IBC 2006

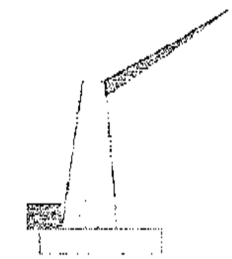
Criteria

Retained Height	=	6.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	2.00 : 1
Height of Soil over Toe	=	12.00 in
Soil Density	=	110.00 pcf
Wind on Stem	=	0.0 psf

Vertical component of active lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	3,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	43.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Water height over heel	=	0.0 ft
Cohesion value	=	200.0 psf
Soil height to ignore for passive pressure	=	0.00 in



Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	4,124 lbs
resultant ecc.	=	6.14 in
Soil Pressure @ Toe	=	1,158 psf OK
Soil Pressure @ Heel	=	331 psf OK
Allowable	=	3,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,346 psf
ACI Factored @ Heel	=	382 psf
Footing Shear @ Toe	=	8.5 psi OK
Footing Shear @ Heel	=	5.4 psi OK
Allowable	=	67.1 psi
Sliding Stability Ratio	=	1.21 Ratio < 1.5!
Sliding Calcs (Vertical Component NOT Used)		
Lateral Sliding Force	=	1,403.3 lbs
less Passive Force	= -	600.0 lbs
less Cohesion Force	= -	1,100.0 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	144.4 lbs OK

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.300
Seismic, E	1.000

Gravity Stem Analysis Data *(Unreinforced material)*

Wall Material Weight	=	145.00 pcf
Front Batter Distance	=	12.00 in
Thickness @ Top of Stem	=	12.00 in
Back Batter Distance	=	6.00 in
Fc : Max. Allow. Compression	=	100.0 psi
Ft : Max. Allow. Tension	=	10.0 psi
Lateral Load Factor	=	1.0

		@ Height #1	@ Height #2	@ Height #3
Height above Footing	=	4.00 ft	2.00 ft	0.00 ft
Wall Thick. @ Height	=	18.00 in	24.00 in	30.00 in
Section Modulus	=	648.00 in^3	1,152.00 in^3	1,800.00 in^3
Moment @ Height	=	57.3 ft-#	458.7 ft-#	1,543.0 ft-#
Vertical Load @ Height	=	362.5 lbs	870.0 lbs	1,522.5 lbs
Actual Unit Tension	=	0.6 psi	1.8 psi	6.1 psi
Actual Unit Compression	=	2.7 psi	7.8 psi	14.5 psi
Shear @ Section	=	86.0 lbs	344.0 lbs	759.0 lbs
Actual Unit Shear	=	0.4 psi	1.2 psi	2.1 psi

DESIGN EXAMPLE 9 — Report Printout

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Title : EX-9
Job # : 709
Description....

Dsgnr:

Page: ____
Date: AUG 8,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8036
Registration #: RP-1110116 RP9.00
Licensed to: [The company name goes here]

Gravity Stem Retaining Wall Design

Code: IBC 2006

Footing Strengths & Dimensions

Toe Width	=	1.00 ft
Heel Width	=	4.50
Total Footing Width	=	5.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 2,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 2.00 in @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,346	302 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu : Design	=	64	64 ft-#
Actual 1-Way Shear	=	8.84	8.44 psi
Allow 1-Way Shear	=	87.08	87.08 psi

Toe Reinforcing = # 7 @ 16.05 in
Heel Reinforcing = # 8 @ 16.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		OVERTURNING Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	1,463.3	2.75	4,024.2
Toe Active Pressure	=	-60.0	0.67	-40.0
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Seismic Load	=	8,100.6	0.00	0.3
Seismic Stem Self Wt	=			
Total	=	1,403.3	O.T.M. =	3,984.2

Resisting/Overturning Ratio = 3.37
Vertical Loads used for Soil Pressure = 4,123.5 lbs

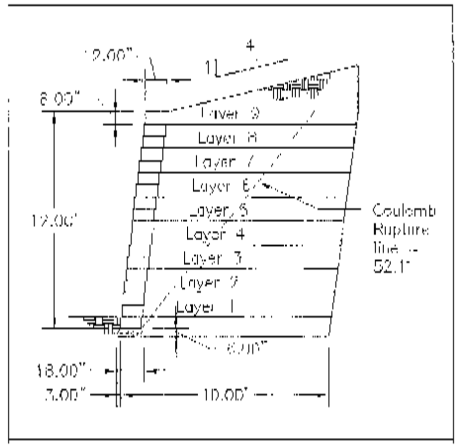
Item		RESISTING Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	1,320.0	4.50	5,940.0
Sloped Soil Over Heel	=	171.9	4.67	802.1
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=		0.00	
Soil Over Toe	=	119.2	0.54	64.7
Surcharge Over Toe	=			
Stem Weight	=	1,522.5	2.36	3,588.8
Earth above Sloping Stem	=	165.0	3.33	550.0
Footing Weight	=	825.0	2.75	2,268.8
Key Weight	=			
Vert. Component	=			
Total =		4,123.5 lbs	R.M.=	13,214.3

Vertical component of active pressure NOT used for soil pressure

DESIGNER NOTES:

Segmental Retaining Wall – Geogrids

Wall height = 12.0 ft.



Embedment - 1.0 ft.

Backfill slope = 4:1 – 14°

Backfill soil: $\phi_b = 33^\circ$ $\gamma = 120$ pcf

$\delta = \frac{2}{3}\phi = 22^\circ$

In situ soil: $\phi_i = 32^\circ$ $\gamma = 110$ pcf

$\delta = \phi_i = 32^\circ$

Use Coulomb method

DL = 50 psf LL = 100 psf

Base width (trial) = 75% of 12 ft. say

= 10.00 ft.

Block selection: Keystone Compac

1" offsets, Height = 8.0 in.,

Depth = 12.0 in.

Wt. = 120 psf.

Batter = 7.1° $\alpha = 90 + 7.1 = 97.1^\circ$

Active earth pressure – backfill zone

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

$$= \frac{\sin^2(97.1 + 33)}{\sin^2 97.1 \sin(97.1 - 22)\left[1 + \sqrt{\dfrac{\sin(33 + 22)\sin(33 - 14.0)}{\sin(97.1 - 22)\sin(97.1 - 14.0)}}\right]^2}$$

= 0.26

K_a (horiz) = 0.26 cos (22 - 7.1) = 0.25

Geogrid Placement:

Lowest layer at 1st block joint = 0.67 ft.

Space subsequent layers every 2nd block = 1.33 ft. o.c.

<u>Tension to Bottom Layer:</u>

$$T_u = 0.25 \times 120 \left(\frac{2.0-0}{2}\right) x\ (12.0-0.67) = 340$$

$$+ \text{ surcharge: } 0.25\ (50+100) \times \left(\frac{2.0-0}{2}\right) = \underline{38}$$

(Sloped backfill neglected) T_u = 378 lbs.

<u>Select Geogrid</u>

Try Strata Systems Stratagrid SG200

Long-term design strength (LTDS) = 1613

S.F. = 1.5

Allowable (LTADS) $= \dfrac{1643}{1.5} = 1085 > 378$ @ bott. layer OK

<u>Check Connection Strength</u>

Equations: Peak connect $= 889 + 0.31\ N$ but < 1624

¾" Serviceability $= 519 + 0.14\ N$ but < 767

$N = (12.0 - 0.67)\ 120 = 1360$ lbs.

$$\text{Peak connect value} = \frac{(889 + .31\,x\,1360)}{1.5} = 874 > 378 \quad \text{OK}$$

$$\text{¾" Serviceability} = \frac{(519 + 0.14\,x\,1360)}{1.0} = 709 > 378 \quad \text{OK}$$

Safety Factor

$$\frac{\textit{Lesser of Peak, Service, or LTADS}}{T_u} = \frac{709}{378} = 1.88 > 1.50 \quad \text{OK}$$

<u>Check Embedment Depth, L_e</u>

Bottom Layer:

$$L_e = \frac{T_u}{2H_{ov} \tan\phi_i\ x\ C_i} \qquad (H_{ov} = \text{overlay soil} + \text{surcharge})$$

Assume $C_i = 0.90$ $H_{ov} = (12.0 - 0.67)\ 120 + (50 + 100) = 1510$ lbs.

$$L_e = \frac{378}{2 \; x \; 1510 \; x \; \tan 33° \; x \; 0.90} = 0.22 \text{ ft.}$$

Add 1.0 ft. per NCMA = 1.22 ft. < 7.00 (AASHTO requires 3.0 ft. added = 3.22 ft.)

Tension to Layer #2:

$$T_u = 0.25 \text{ x } 120 \left(\frac{3.33 - 0.67}{2}\right) x \; (12.0 - 2.0) \quad = \quad 399$$

$$+ \text{ surcharge: } 0.25 \; (50 + 100) \text{ x } \left(\frac{3.33 - 0.67}{2}\right) \quad = \quad \underline{50}$$

(Sloped backfill neglected) T_u = 449 lbs.

Check Connection Strength

Equations: Peak connect = 889 + 0.31 N but < 1624

¾" Serviceability = 519 + 0.14 N but < 767

N = (12.0 – 2.0) 120 = 1200 lbs.

$$\text{Peak connect value} = \frac{(889 + .31 \, x \, 1200)}{1.5} = 841 > 449 \qquad \text{OK}$$

$$\text{¾" Serviceability} = \frac{(519 + 0.14 \, x \, 1200)}{1.0} = 687 > 449 \qquad \text{OK}$$

Tension to Top Layer:

$$T_u = 0.25 \text{ x } 120 \left(\frac{12.0 - 10.0}{2}\right) x \; (12.0 - 11.33) \quad = \quad 20$$

$$+ \text{ surcharge: } 0.25 \; (50 + 100) \text{ x } \left(\frac{12.0 - 10.0}{2}\right) \quad = \quad \underline{38}$$

(Sloped backfill neglected) T_u = 58 lbs.

Check Geogrid

Strata Systems Stratagrid SG200

Long-term design strength (LTDS) = 1643

S.F. = 1.5

$$\text{Allowable (LTADS)} = \frac{1643}{1.5} = 1095 > 58 \qquad \text{OK}$$

Check Connection Strength

Equations: Peak connect $= 889 + 0.31\ N$ but < 1624

¾" Serviceability $= 519 + 0.14\ N$ but < 767

$N = (12.0 - 11.33)\ 120 = 80$ lbs.

Peak connect value $= \dfrac{(889 + .31\ x\ 80)}{1.5} = 609 > 58$ OK

¾" Serviceability $\dfrac{(519 + 0.14\ x\ 80)}{1.0} = 530 > 58$ OK

Check Embedment Depth, L_e

$$L_e = \frac{T_u}{2H_{ov} \tan \phi_i\ x\ C_i}$$ (H_{ov} = overlay soil + surcharge)

Assume $C_i = 0.90$ $H_{ov} = (12.0 - 11.33)\ 120 + (50 + 100) = 230.4$ lbs.

$$L_e = \frac{58}{2\ x\ 230\ x\ \tan\ 33°\ x\ 0.90} = 0.22$$

Add 1.0 ft. per NCMA = 1.22 ft (AASHTO requires 3.0 ft. added = 3.22 ft.)

Check available embedment depth based on base = 10.0 ft.

Coulomb rupture angle = 52.1°

L_e avail.: $(10.0 - 1.0) + 11.33 \tan 7.1° - \dfrac{11.33}{\tan 52.1} = 1.59 > 1.22$ OK

Overturning Moments

NOTE: Earth pressure applied to back of reinforced zone, assuming Vertical Plane (90°)
effective ht. = 12.00 + 9.00 tan 14 = 14.25 ft.

K_a [external (in-situ)]

$\phi_e = 32°$

$\alpha = 90°$

$\delta = \phi = 32°$

$\beta = 14°$

$\gamma = 110$

K_a = 0.34

K_e (horiz) = 0.29

		Force		Distance	Moment
Earth pressure	$0.29 \times 110 \times \frac{14.25^2}{2} =$	3239	$\frac{14.25}{3} = 4.75$		15,385 ft.-lbs.
Surcharge	$0.29\,(50 + 100) \times 14.25 =$	620	$\frac{14.25}{2} = 7.1$		4,418
Sliding Force Total =		3859 lbs.			19,803 ft. lbs.

Resisting Moments

		Force	Distance	Moment
W_4 Wall	12 x 120 =	1440	1.25	1,800
W_1 Earth	9 x 12 x 120 =	12,960	5.5	71,280
W_2 Sloped	9 x 2.25 x 120 x ½ =	1215	$\frac{2}{3} \times 9 + 2.49 = 8.5$	10,328
W_3 Surcharge	9 x (50 + 100) =	1350	$\frac{9}{2} + 2.49 =$ 7.0	9,436
Total vert. force =		16,965 lbs.		92,844 ft. lbs.

Overturning safety factor ratio = $\frac{92{,}844}{19{,}803} = 4.68 > 2.0$ OK

Check Sliding at Base

Lateral force on reinforced soil = 3859 lbs.

Sliding resistance = 16,965 tan ϕ_e = 10,600 lbs.

Total vertical force = 16,965 lbs.

Sliding safety factor = $\frac{10{,}600}{3859} = 2.75$ OK

Check Soil Pressure

Use Meyerhoff Method

Eccentricity, e = $\frac{(B)}{2} - \frac{M_R - M_{OT}}{V_T} = \frac{(10)}{2} - \frac{92{,}84 - 19{,}803}{16{,}965} = 5.0 - 4.31 = 0.69\ ft.$

Effective bearing length = (B) – 2e = (10) – 2 x 0.69 = 8.62 ft.

Bearing pressure = $\frac{V_T}{8.62} = \frac{16{,}965}{8.62}$ = 1968 psf

Allowable Bearing Pressure

Assume no cohesion (c = 0)

$= \gamma D N_q + 0.5\gamma$ [Eff. Bearing length] N_γ γ = in situ soil density = 110 pcf.

= 110 x 1.0 x 23.2 + .5 x 110 (8.62) 30.22 D = Depth of embedment

= 1.0 ft.

= 16,879 psf N_q for 32° = 23.2

N_γ for 32° = 30.22

Soil bearing ratio = $\frac{16{,}879}{1968}$ = 8.58 OK

Values for N_q and N_γ

ϕ_i	N_q	N_γ
31	20.63	26.0
32	23.2	30.2
33	26.1	35.2
34	29.4	41.1
35	33.3	48.0
36	37.8	56.3

Check for Added Seismic

Added seismic has three components:

Seismic force of self-weight of wall

Seismic force from reinforced zone

Seismic force acting on reinforced zone

k_h = 0.15 θ = $\tan^{-1} 0.15$ = 8.53°

K_{AEH} = 0.55 [α = 97.1°, ϕ_i = 33°, δ = 22°, β = 14°]

K_{AH} = 0.25 $\Delta K_{AEH} = K_{AEH} - K_{AH}$ = .55 - .25 = 0.30

Total seismic lateral force:

Wall: K_h x W x H – 0.15 x 120 x 12 = 216 lbs.

Reinf. Zone: K_h x H (.5 H – t) γ_i = 0.15 x 12 (.5 x 12 – 1.0) 120 = 1080 lbs.

+ sloped soil = k_h [(.5 x 12 – 1.0)^2 x γ_i x tan β x 0.5]

= .15 [(.5 x 12 – 1.0)^2 x 120 x tan 14° x .5] = 56

* Exterior of zone: – 0.5 x ΔK_{AEH} [H + (B-t) tan β x 0.5]^2 x 0.5 γ_e

= .5 x 0.30 [12 + (10 – 1) tan 14°]^2 x 0.5 x 110 = <u>1675</u> lbs.

* Use 50% of seismic per NCMA

Total seismic = 3027 lbs.

** Total sliding force = 0.5 K_{AH} [H + (B – t) tanβ]^2 γ_e + seismic

= 0.5 x 0.29 x [12 + (10-1) tan 14°]^2 x 110 + 3027 = 6266 lbs.

** K_{AH} for back of reinf. zone based on α= 90° and $\delta = \phi_e$ ∴ K_{AH} = 0.29

+ (DL + LL) = 0.29 (50 + 100) x [12 + (10 – 1) tan 14°] = 620

Total sliding = 6266 + 620 = 6886 lbs.

Sliding resistance = 10,660 lbs.

Sliding ratio $= \dfrac{10{,}660}{6886} = 1.55$ OK

<u>Added Seismic Overturning</u>

Wall: OTM $= 216 \times \dfrac{12}{2}$ = 1296

Reinf. Zone $= 1080 \times \dfrac{12}{2}$ = 6480

Exterior: = 1675 x [12 + (10 - 1) tanβ] x 0.6 = 14,321

56 x [12 + (.5H tan 14° x 0.33)] = 700

Total added seismic overturning = 22,797 ft. lbs.

Total overturning = 22,799 + 19,803 = 42,782 ft. lbs.

Overturning ratio w/seismic = $\frac{92,852}{42,782} = 2.17$

<u>Seismic Tension to Layer #1</u>

$$k_h \left(\frac{h_2 - h_o}{2}\right) w + \Delta K_{AEH} \gamma_i H \left(\frac{h_2 - h_o}{2}\right) \left[0.8 - 0.6\left(\frac{H - h_o}{H}\right)\right]$$

$$= \quad 0.15 \left(\frac{2.0 - 0}{2}\right) 120 + 0.30 \text{ x } 120 \text{ x } 12 \left(\frac{2.0 - 0}{2}\right) \left[0.8 - 0.6\left(\frac{12 - 0.67}{12}\right)\right]$$

= 119 lbs.

Pullout safety factor = $\frac{1064}{378 + 119} = 2.14$

<u>Seismic tension to top layer #9</u>

$$= \quad 0.15 \left(\frac{12 - 10}{2}\right) 120 + 0.30 \text{ x } 120 \text{ x } 12 \left(\frac{12 - 10}{2}\right) \left[0.8 - 0.6\left(\frac{11.33}{12}\right)\right]$$

= 349 lbs.

Pullout S.F. = $\frac{540}{58 + 349} = 1.28 > 1.1$ (for seismic) OK

$$L_e = \frac{58 + 349}{2\,x\left[(0.67 + say\ 2.0)\,x\,120 + (50 + 100)\right]\tan 33^\circ\,x\,0.90} + 1.0 = 1.66 \text{ ft.}$$

Available embedment = (10-1) + 11.33 tan 7.1° - $\frac{11.33}{\tan 52.1^\circ}$ = 1.59

Consider OK w/seismic

DESIGN EXAMPLE 10 — Report Printout

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949-721-4099

Title : EX-10
Job # :
Description...

Dsgnr:

Page: ____
Date: SEP 1,2005

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8066
Registration #: RP-1110.5 RP9.00
Licensed to: (The company name goes here)

Segmental Retaining Wall Design with Geogrids

Code: NCMA

Criteria	
Wall height (retained height), ft	12.00
Backfill slope	4:1
Backfill angle	14.0
Embedment	1.0

Soil data	
External Soil, Phi_e	32
External soil density (in situ), pcf	110
Internal Soil, Phi_i	33
Internal soil density, pcf	120
Wall Soil Friction Angle	32
K_a(Horz)	0.25

Thumbnail

Stability	
Overturning ratio	4.74
Sliding ratio	2.77
Overturning moment, ft-lbs	19,594
Resisting moment, ft-lbs	92,866
Total lateral/sliding force, lbs	3,822
Sliding Resistance, ft	10,598.87
Total vertical force, lbs	16,952
Base length, ft	10.00
Eccentricity on base, ft	0.68
Effective base length, ft	8.64
Soil bearing pressure, psf	1,963.24
Allowable soil bearing, psf	16,902.46
Soil Bearing Ratio	8.61

Segmental block data	
Vendor selection	Keystone Retaining Wall Systems
Vendor web address	www.keystonewalls.com
Block selection type	Compac
Block height, in	8.00
Block depth, in	12.00
Offset per block, in	1.00
Batter angle	7.13
Wall weight, psf	120.00
Hinge height, ft	8.00

Loading	
Dead load, psf	50
Live load, psf	100
Seismic Design Kh	0.00

Geogrid material	
Vendor Selection	Strata Systems, Inc.
Vendor web address	www.geogrid.com
Geogrid type	Stratagrid SG 200
LTDS	1643.00
Factor of safety	1.50
LTADS	1095.33
Peak connection equation	889 + 0.31N
Peak connection maximum	1674
Serviceability connection equation	519 + 0.14N
Serviceability connection maximum	767

Wall Analysis Table:

Block	Layer	Height above base Ft	In	Dec	Tension Static	Tension Seismic	Connection Peak	Connection Serv	Embed Le	Vert N	S. F.
18		12'	0"	12.00							
17	9	11'	4"	11.33	58		609.2	530.2	1.21	80	9.22
15	8	10'	0"	10.00	130		642.3	552.6	1.29	240	4.25
13	7	9'	8"	8.67	163		675.3	575.0	1.29	400	3.13
11	6	7'	4"	7.33	237		708.4	597.4	1.29	560	2.52
9	5	6'	0"	6.00	290		741.5	619.8	1.29	720	2.14
7	4	4'	8"	4.67	343		774.5	642.2	1.29	880	1.87
5	3	3'	4"	3.33	397		807.6	664.6	1.29	1,040	1.67
3	2	2'	0"	2.00	450		840.7	687.0	1.29	1,200	1.53
1	1	0'	8"	0.67	378		873.7	709.4	1.21	1,360	1.88
Base		0'	0"	0.00						1,440	

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Title : EX-10
Job # :
Description....

Dsgnr.

Page: ____
Date: SEP 1,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8056
Registration #: RP-11101 5 RP9.00
Licensed to: [The company name goes here]

Segmental Retaining Wall Design with Geogrids Code: NCMA

Summary: Resisting / Overturning

Resisting Moments

Item	Force, lbs	Distance, ft	Moment, ft-lbs
Wall	1,440	1.25	1,800
Reinf. earth	12,960	5.50	71,280
Sloped	1,212	8.53	10,336
Dead load	450	7.00	3,150
Live load	900	7.00	6,300
Total	16,962		92,866

Overturning Moments

Item	Force, lbs	Distance, ft	Moment, ft-lbs
Earth	3,211	4.75	15,245
Surcharge, DL	204	7.12	1,450
Surcharge, LL	407	7.12	2,900
Seismic, Wall	0	0.00	0
Seismic, Reinf.	0	0.00	0
Seismic, Sloped soil	0	0.00	0
Seismic, Exterior	0	0.00	0
Total	3,822		19,594

Overturning Ratio 4.74

ASSUMPTIONS AND CRITERIA USED

1. References used include *Design Manual for Segmental Retaining Walls*, 2nd Edition, and *Segmental Retaining Walls – Seismic Design Manual*, 1st Edition, both by NCMA
2. Blocks are all same size and uniform offsets (batter) for full wall height.
3. Coulomb earth pressure theory used for earth pressures and failure plane angle.
4. Refer to geotechnical report for backfill material, compaction, and other design data and recommendations.
5. Cap blocks if used are above the retained height and are neglected in this design.
6. Geogrid LTDS and connection values for block vendors obtained from ICC Evaluation Service (ES Legacy Reports) or as provided by vendors. Since these may change or be updated, verification of values is recommended.
7. Block sizes obtained from vendors' literature and may vary with locality.
8. Geogrid layers are equally spaced vertically, all same length, and laid horizontally.
9. Average weight of block and cell infill assumed to be 120 pcf.
10. See vendor web sites (on input screen) for more information and specifications.
11. Design height is limited to 18 feet or 24 blocks, whichever is less. Contact vendor for higher designs or special conditions.
12. Seismic design is per *Seismic Design Manual* cited above. Also see *Methodology/Seismic Design* in *User's Manual*.
13. Vendor specifications or project specifications, whichever is most restrictive, to be followed for construction procedures.
14. Add notes and details for proper drainage.
15. See *User's Manual* Design Example #10 for methodology and sample verification calculations.
16. Final design responsibility is with the project Engineer-of-Record.

DESIGN EXAMPLE 10 **Report Printout**

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Title : RX-10-w/seismic
Job # 710 Dsgnr:
Description....

Page: ___
Date: SEP 1,2

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8956
Registration #: RP-1110 1 & RP9.00
Licensed to: [The company name goes here]

Segmental Retaining Wall Design with Geogrids

Code: NCM

Criteria	
Wall height (retained height), ft	12.00
Backfill slope	4:1
Backfill angle	14.0
Embedment	1.0

Soil data	
External Soil, Phi_e	32
External soil density (in situ), pcf	110
Internal Soil, Phi_i	33
Internal soil density, pcf	120
Wall Soil Friction Angle	22
K_a(Horiz)	0.25
K_AE(Horiz)	0.55

Thumbnail

Stability	
Overturning ratio	2.18
Sliding ratio	1.54
Overturning moment, ft-lbs	42,586
Resisting moment, ft-lbs	92,866
Total lateral/sliding force, lbs	6,872
Sliding Resistance, ft	10,596.87
Total vertical force, lbs	15,962
Base length, ft	10.00
Eccentricity on base, ft	2.04
Effective base length, ft	5.93
Soil bearing pressure, psf	2,663.97
Allowable soil bearing, psf	12,399.52
Soil Bearing Ratio	4.33

Loading	
Dead load, psf	50
Live load, psf	100
Seismic Design Kh	0.15

Segmental block data	
Vendor selection	Keystone Retaining Wall Systems
Vendor web address	www.keystonewalls.com
Block selection type	Compac
Block height, in	8.00
Block depth, in	12.00
Offset per block, in	1.00
Batter angle	7.13
Wall weight, psf	120.00
Hinge height, ft	8.00

Geogrid material	
Vendor Selection	Strata Systems, Inc.
Vendor web address	www.geogrid.com
Geogrid type	Stratagrid SG 200
LTDS	1643.00
Factor of safety	1.50
LTADS	1095.33
Peak connection equation	889 + 0.31N
Peak connection maximum	1624
Serviceability connection equation	519 + 0.14N
Serviceability connection maximum	767

Wall Analysis Table:

Block	Layer	Height above base Ft	in	Dec	Tension Static	Seismic	Connection Peak	Serv	Embed Le	Vert N	S.F.
18		12'	0"	12.00							
17	9	11'	4"	11.33	56	354	605.2	530.2	2.58	80	1.29
15	8	10'	0"	10.00	130	423	642.3	552.6	2.24	240	0.98
13	7	8'	8"	8.67	183	394	675.3	575.0	1.90	400	1.00
11	6	7'	4"	7.33	237	355	708.4	597.4	1.71	560	1.01
9	5	6'	0"	6.00	290	316	741.5	619.8	1.60	720	1.02
7	4	4'	8"	4.67	343	277	774.5	642.2	1.52	880	1.03
5	3	3'	4"	3.33	397	238	807.6	664.6	1.46	1,040	1.05
3	2	2'	0"	2.00	450	199	840.7	687.0	1.41	1,200	1.06
1	1	0'	8"	0.67	378	120	873.7	709.4	1.28	1,360	1.42
Base		0'	0"	0.00						1,440	

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Title : EX-10-w/seismic
Job # 710 Dsgnr:
Description....

Page: ____
Date: SEP 1,2009

This Wall in File: c:\program files\rp2007\examples.rp5

Retain Pro 9 © 1989 - 2009 Ver: 9.00 8956
Registration #: RP-1110[1] 5 RP9.00 **Segmental Retaining Wall Design with Geogrids** Code: NCMA
Licensed to: [The company name goes here]

Summary: Resisting / Overturning

Resisting Moments

Item	Force, lbs	Distance, ft	Moment, ft-lbs
Wall	1,440	1.25	1,800
Reinf. earth	12,960	5.50	71,280
Sloped	1,212	8.53	10,336
Dead load	450	7.00	3,150
Live load	900	7.00	6,300
Total	16,962		92,866

Overturning Moments

Item	Force, lbs	Distance, ft	Moment, ft-lbs
Earth	3,210	4.75	16,245
Surcharge, DL	204	7.12	1,450
Surcharge, LL	407	7.12	2,900
Seismic, Wall	216	6.00	1,296
Seismic, Reinf.	1,080	6.00	6,480
Seismic, Sloped soil	56	12.41	698
Seismic, Exterior	1,699	8.55	14,519
Total	6,872		42,586

Overturning Ratio 2.18

ASSUMPTIONS AND CRITERIA USED

1. References used include *Design Manual for Segmental Retaining Walls*, 2nd Edition, and *Segmental Retaining Walls – Seismic Design Manual*, 1st Edition, both by NCMA.
2. Blocks are all same size and uniform offsets (batter) for full wall height.
3. Coulomb earth pressure theory used for earth pressures and failure plane angle.
4. Refer to geotechnical report for backfill material, compaction, and other design data and recommendations.
5. Cap blocks if used are above the retained height and are neglected in this design.
6. Geogrid LTDS and connection values for block vendors obtained from ICC Evaluation Service (ES Legacy Reports) or as provided by vendors. Since these may change or be updated, verification of values is recommended.
7. Block sizes obtained from vendors' literature and may vary with locality.
8. Geogrid layers are equally spaced vertically, all same length, and laid horizontally.
9. Average weight of block and soil infill assumed to be 120 pcf.
10. See vendor web sites (on input screen) for more information and specifications.
11. Design height is limited to 16 feet or 24 blocks, whichever is less. Contact vendor for higher designs or special conditions.
12. Seismic design is per *Seismic Design Manual* cited above. Also see *Methodology/Seismic Design* in *User's Manual*.
13. Vendor specifications or project specifications, whichever is most restrictive, to be followed for construction procedures.
14. Add notes and details for proper drainage.
15. See *User's Manual* Design Example #10 for methodology and sample verification calculations.
16. Final design responsibility is with the project Engineer-of-Record.

DESIGN EXAMPLE 11 **Page 1 of 3**

Segmental Gravity Wall Design

Criteria:

Retained height (trial) = 4.0 ft.

Slope: Level

Soil density, in situ – 110 pcf

Soil density, backfill – 120 pcf

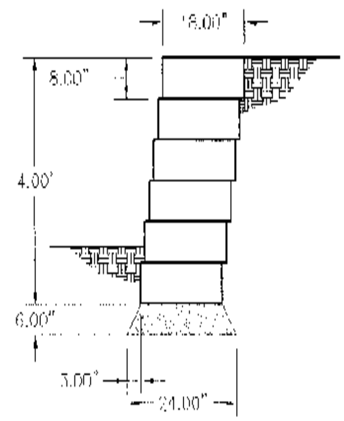
Soil ϕ, backfill 33°

Soil ϕ, in-situ 34°

Soil/wall friction angle = ⅔ϕ backfill

= 22°

Embedment = 1.0 ft.



Block Data:

Try Keystone, Standard

Height – 8.00 in. Depth = 18.0"

Wt. = 120 x 1.5 180 psf

Offset: ½ in. per block

Batter = $\tan^{-1}(0.5/8.0) = 3.6° = \omega$

$$\text{Hinge height} = \frac{(blockdepth)}{\tan \omega} = \frac{1.50}{\tan 3.6°} = \frac{1.50}{.063} = 24\ ft. > 4.0\ ft.$$

Lateral Soil Pressure:

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

K_a (horiz.) = $K_a \cos(90 + \delta - \alpha)$

$$\frac{\sin^2(93.60 + 33.0)}{\sin^2 93.60 \sin(93.60 - 22.0)\left[1 + \sqrt{\dfrac{\sin(33.0 + 22.0)\sin(33.0 - 0}{\sin(93.60 - 22.0)\sin(93.60 - 0)}}\right]^2} = 0.24$$

K_a(horiz.) = 0.24 cos (90 + 22.0 – 93.60) = 0.23

$$\text{Total lateral force} = \frac{0.23 \times 120 \times 4^2}{2} = 221\ lbs.$$

DESIGN EXAMPLE 11

<u>Stability:</u>

Overturning moment = $221 \times \frac{4.0}{3} = 294$ ft. lbs.

Total vertical force = 4.0 x 180 pcf = 720 lbs.

Resisting moment = $720 \left[\left(\frac{1.5}{2}\right) + \left(\frac{4.0}{2}\tan 3.6^\circ\right)\right] = 630$ ft. lbs.

Overturning ratio = $\frac{630}{294} = 2.14 > 1.50$ OK

<u>Check Sliding</u>

Sliding force = 221 lbs.

Resistanse = 720 (tan ϕ in situ) = 720 x tan 34° = 486 lbs.

Sliding safety factor = $\frac{486}{221} = 2.20 > 2.0$ OK

<u>Soil Bearing Pressure</u>

Base width = 1.0 + 0.5 = 1.5 ft. = B

$$e = \frac{B}{2} - \left(\frac{Resisting - Overturning}{Total\ Vert.}\right) = \frac{1.5}{2} - \frac{630 - 294}{720} = 0.28$$

Effective bearing area = (B – 2e) + [(1.5 – 2 x 0.28) +0.50] = 1.44 ft.

Bearing pressure = $\frac{720}{1.44} = 500$ psf.

<u>Allowable Bearing Capacity:</u>

Effective bearing area = 1.44 ft.

Depth to bottom of 6 in. pad = 1.0 + 0.5 = 1.5 ft.

$N_q = 29.4$ $N_\gamma = 41.1$

Bearing capacity = $\gamma D N_q + 0.5\gamma (B - 2_e) N_\gamma$ (N_q and N_γ from Table in NCMA Handbook)

= 110 x 1.5 x 29.4 + 0.5 x 110 x [(1.5 – 2 x 0.28) + 0.5] 41.1

= 8106 psf OK

Bearing safety factor = $\frac{8106}{500} = 16.2$

<u>If Design for Seismic</u>:

Assume $k_h = 0.05$ (If greater, overturning would exceed resisting moment!)

K_a (horiz.) = 0.23

$$K_{AE} = \frac{\sin^2(\alpha + \theta - \phi)}{\cos\theta \sin^2\alpha \sin(\alpha + \theta + \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \beta - \theta)}{\sin(\alpha + \theta + \delta)\sin(\alpha - \beta)}}\right]^2}$$

$\theta = \tan^{-1}(0.05) = 2.9°$

$$\frac{\sin^2(93.60 + 2.9 - 33.0)}{\cos 2.9 \sin^2 93.60 \sin(93.60 + 2.9 + 22.0)\left[1 + \sqrt{\frac{\sin(33.0 + 22.0)\sin(33.0 - 0.0 - 2.9)}{\sin(93.60 + 2.9 + 22.0)\sin(93.60 - 0.0)}}\right]^2}$$

K_{AE} (horiz.) = 0.32 cos (90 + 22.0 − 93.60) = 0.30

$K_{AH} = 0.23$

$\Delta K_{AEH} = 0.30 - 0.23 = 0.07$

Added seismic = 0.07 $(4)^2$ x 120 x ½ = 67 lbs. + 0.05 x 4.0 x 180 = 36 lbs.

Total sliding = 221 + 67 + 36 = 324 lbs.

Overturning moment = 294 + 67 x 0.60 x 4 + 36 x 2 = 527 ft. lbs.

Overturning ratio = $\frac{630}{547} = 1.2 < 1.1$

(Note: Safety factor when seismic included = 0.75 x 1.5 = 1.1 per IBC '09)

This design example used a very low seismic factor for illustration. A higher seismic factor would require a revised design.

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Title : EX-11
Job # : 710
Description....

Dsgnr:

Page: ____
Date: MAR 10,2009

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Segmental Gravity Retaining Wall Design

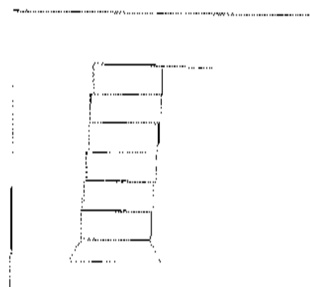
Code: NCMA

Criteria

Wall height (retained height), ft	4.00
Backfill slope	Level
Backfill angle	0.0
Embedment	1.0

Soil data

External Soil, Phi_e	34
External soil density (in situ), pcf	110
Internal Soil, Phi_i	33
Internal soil density, pcf	120
Wall Soil Friction Angle	22
K_a(Horiz)	0.23



Thumbnail

Stability

Overturning ratio	2.16
Sliding ratio	2.22
Overturning moment, ft-lbs	291
Resisting moment, ft-lbs	630
Total lateral/sliding force, lbs	219
Sliding Resistance, ft	485.65
Total vertical force, lbs	720
Base length, ft	1.50
Eccentricity on base, ft	0.28
Effective base length, ft	1.44
Soil bearing pressure, psf	499.80
Allowable soil bearing, psf	8,107.43
Soil Bearing Ratio	16.22

Segmental block data

Vendor selection	Keystone Retaining Wall Systems
Vendor web address	www.keystonewalls.com
Block selection type	Standard
Block height, in	8.00
Block depth, in	18.00
Offset per block, in	0.50
Batter angle	3.58
Wall weight, psf	180.00
Hinge height, ft	24.00

Loading

Dead load, psf	0
Live load, psf	0
Seismic Design Kh	0.00

Wall Analysis Table:

Block	Height above base Ft	In	Dec	Vert N	Lateral Static	Seismic	Shear Interface	S. F.
6	4'	0"	4.00				1,548.00	
5	3'	4"	3.33	120	6		1,585.20	261.13
4	2'	8"	2.67	240	24		1,622.40	56.81
3	2'	0"	2.00	360	55		1,659.60	30.38
2	1'	4"	1.33	480	97		1,696.80	17.47
1	0'	8"	0.67	600	152		1,734.00	11.43
Base	0'	0"	0.00	720	219		1,771.20	8.10

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Segmental Gravity Retaining Wall Design

Code: NCMA

ASSUMPTIONS AND CRITERIA USED

1. References used include *Design Manual for Segmental Retaining Walls*, 2nd Edition, and *Segmental Retaining Walls – Seismic Design Manual*, 1st Edition, both by NCMA.
2. Blocks are all same size and uniform offsets (batter) for full wall height.
3. Coulomb earth pressure theory used for earth pressures and failure plane angle.
4. Refer to geotechnical report for backfill material, compaction, and other design data and recommendations.
5. Cap blocks if used are above the retained height and neglected in this design.
6. Block sizes obtained from vendors' literature and may vary with locality.
7. Average weight of block and cell infill assumed to be 120 pcf.
8. See vendor web sites (on input screen) for more information and specifications.
9. Design height is limited to 12 feet or 18 blocks, whichever is less. Contact vendor for higher designs or special conditions.
10. Seismic design is per *Seismic Design Manual* cited above. Also see *Methodology/Seismic Design* in *User's Manual*.
11. Vendor specifications or project specifications, whichever is more restrictive, to be followed for construction procedures.
12. Add notes and details for proper drainage.
13. See *User's Manual* Design Example #11 for methodology and sample verification calculations.
14. Final design responsibility is with the project Engineer-of-Record.

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Segmental Gravity Retaining Wall Design

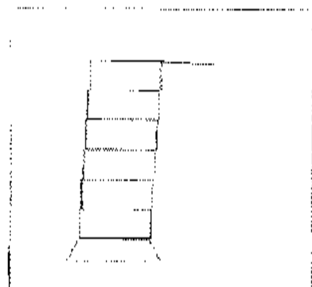
Code: NCMA

Criteria	
Wall height (retained height), ft	4.00
Backfill slope	Level
Backfill angle	0.0
Embedment	1.0

Soil data	
External Soil, Phi_e	34
External soil density (in situ), pcf	110
Internal Soil, Phi_i	33
Internal soil density, pcf	120
Wall Soil Friction Angle	22
K_a(Horz)	0.23
K_AE(Horz)	0.31

Stability	
Overturning ratio	1.15
Sliding ratio	1.47
Overturning moment, ft-lbs	546
Resisting moment, ft-lbs	630
Total lateral/sliding force, lbs	331
Sliding Resistance, ft	485.65
Total vertical force, lbs	720
Base length, ft	1.50
Eccentricity on base, ft	0.63
Effective base length, ft	0.73
Soil bearing pressure, psf	981.49
Allowable soil bearing, psf	6,509.25
Soil Bearing Ratio	6.63

Loading	
Dead load, psf	0
Live load, psf	0
Seismic Design Kh	0.05



Thumbnail

Segmental block data	
Vendor selection	Keystone Retaining Wall Systems
Vendor web address	www.keystonewalls.com
Block selection type	Standard
Block height, in	8.00
Block depth, in	18.00
Offset per block, in	0.50
Batter angle	3.58
Wall weight, psf	180.00
Hinge height, ft	24.00

Wall Analysis Table:

Block	Height above base Ft	In	Dec	Vert N	Lateral Static	Lateral Seismic	Shear Interface	S. F.
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5	3'	4"	3.33	120	6	25	1,585.20	51.00
4	2'	8"	2.67	240	24	47	1,622.40	22.60
3	2	0"	2.00	360	55	67	1,659.60	13.60
2	1'	4"	1.33	480	97	85	1,696.80	9.32
1	0'	8"	0.67	600	152	100	1,734.00	6.90
Base	0'	0"	0.00	720	219	112	1,771.20	5.36

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Job # : Dsgnr: Page: ______
Description.... Date SEP 1,2009

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Segmental Gravity Retaining Wall Design Code: NCMA

ASSUMPTIONS AND CRITERIA USED

1. References used include *Design Manual for Segmental Retaining Walls*, 2nd Edition, and *Segmental Retaining Walls – Seismic Design Manual* 1st Edition, both by NCMA.
2. Blocks are all same size and uniform offsets (batter) for full wall height
3. Coulomb earth pressure theory used for earth pressures and failure plane angle
4. Refer to geotechnical report for backfill material, compaction, and other design data and recommendations
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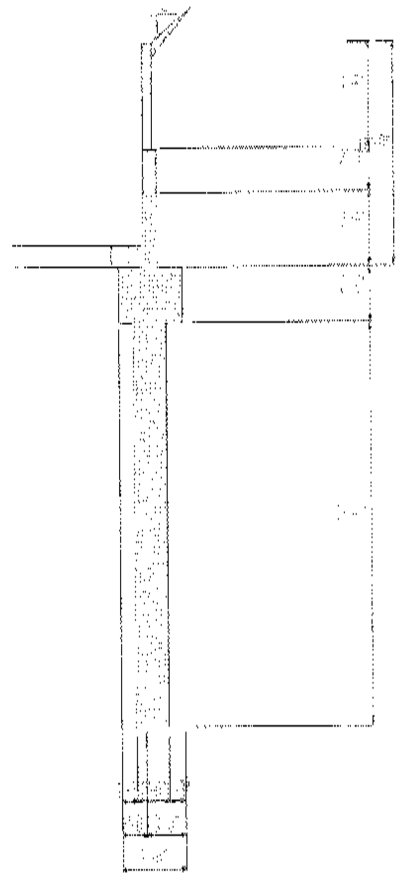
6. Block sizes obtained from vendors' literature and may vary with locality.
7. Average weight of block and cell infill assumed to be 120 pcf.
8. See vendor web sites (on input screen) for more information and specifications.
9. Design height is limited to 12 feet or 18 blocks, whichever is less. Contact vendor for higher designs or special conditions.
10. Seismic design is per *Seismic Design Manual* cited above. Also see *Methodology/Seismic Design* in *User's Manual*.
11. Vendor specifications or project specifications, whichever is more restrictive, to be followed for construction procedures.
12. Add notes and details for proper drainage.
13. See *User's Manual Design Example #11* for methodology and sample verification calculations.
14. Final design responsibility is with the project Engineer-of-Record

DESIGN EXAMPLE 12



Cantilevered wall with pier foundation Code: IBC '06

Use wall design Example #1
for forces imposed to pier

Revise footing/pier cap to 30" deep
x 5.0 ft. wide

Try pier spacing = 6.0 ft.

Vertical load from wall
= 5688 x 6.0 = 34,128 lbs.

Total lateral load at top of pier from wall

$$= \frac{45\,(10.0 + 2.5 + 1.0)^2}{2} \times 6.0 - \frac{30\,x\,(2.5 + 1.0)^2}{2}$$

x 6.0 = 23,502 lbs.

Assume no lateral support at top of pier

Pier f'_c = 3000 psi $\qquad$ f_y = 60,000 psi

Soil bearing at pier tip = 6,000 psf

Use skin friction = 100 psf, neglect top 2.0 ft.

Assume pier fixity at $\frac{1}{6}$ pier depth

Allow passive for pier: 300 pcf.

Load factor for pier concrete design = 1.6

Assume added lateral load at pier top (creep effect) = 500 lbs.

Assume diameter effectiveness multiplier for passive = 1.0

Center the pier under footing/cap

Moment applied at top of pier (unfactored) = 18,238 x 6.0 = 109,428 ft-lbs.

Try 30" diameter pier spaced 6.0 ft. on center

Trial embedment depth (IBC '06 equation 18.1)

$P = 23{,}502 \qquad \text{Equiv "h"} = \dfrac{109{,}428}{23{,}502} = 4.66'$

Embedment = 16.88 ft.

Use 18.0 ft. embedment

Total bearing capacity (neglect pier weight)

$$= \frac{2.5^2 \; x \, 3.14}{4} \text{ x } 6000 + 2.5 \text{ x } 3.14 \text{ x } 100\ (18\text{-}2) = 41{,}998^{\#} > 34{,}128.$$

Determine eccentricity of vert. load on pier

Dist front edge ftg to vertical resultant

$$= \frac{17{,}388}{5688} = 3.06 \text{ ft.}$$

$$e = \frac{5.0}{2} - 3.04 = -0.56 \text{ ft.}$$

Pier M_u = [23,502 (4.66 + 18/6) + 500 (18/6) – 34,128 x 0.56] 1.6 = 259,862 ft-lbs.

Check pier ϕM_n per *Whitney Approximation Method

*ASCE transactions paper 1942 by Charles Whitney

Use 30" diameter with 8 - #8 bars, circular pattern

$f_c' = 3000$ psi, $f_y = 60{,}000$ psi, $\phi = 0.90$, clearance = 3"

Gross area of circular column = $\dfrac{3.14 \; x \, 30^2}{4} = 707$ sq. in.

Whitney equivalent rectangular width = 707 / (0.80 x 30) = 29.5 in.

Whitney equivalent "d" = $\frac{2}{3}$ (30) = 20 in.

Per ACI equations:

$$a = \frac{A_s\ f_y}{.85\ f_c' b} = (8 \text{ x } 0.5 \text{ x } 0.79 \text{ x } 60{,}000) / (0.85 \text{ x } 3000 \text{ x } 29.5) = 2.52''$$

$$\phi M_n = 0.90\ A_s f_y \left(d - \frac{a}{2} \right) = (0.90 \text{ x } 8 \text{ x } 0.5 \text{ x } .79 \text{ x } 60{,}000)$$

$$\left[20 - \left(\frac{2.52}{2} \right) \right] = 3198 \text{ in.-kips} = 266{,}500 \text{ ft-lbs.}$$

∴ Pier design OK

DESIGN EXAMPLE 12 — Report Printout

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Title : EX-12
Job # : 712
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Date: FEB 9,2008

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Cantilevered Retaining Wall Design

Code: IBC 2006

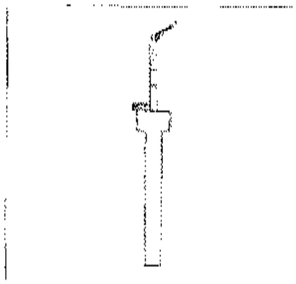
Criteria

Retained Height	=	10.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	2.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Vertical component of active lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	45.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	389.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe		110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Thumbnail

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Pier Foundation
Not restrained at top

Spacing	=	5.00	ft
Diameter	=	30.00	in
Pier Reinforcing	=	8 - #8	
Embedment	=	16.00	ft
fc	=	3,000	psi
fy	=	60,000	psi
Added Lat. at pier top	=	500.0	lbs
End soil bearing allow.	=	6,000	psf
Skin friction	=	100.0	psf
Allow. passive press.	=	300	pcf
Mu	=	256,263	ft-lbs
phi*Mn	=	265,568	ft-lbs
Vu	=	54.35	psi
phi*Vn	=	82.16	psi
Total Vertical Load	=	34,128	lbs
Vertical Capacity	=	42,054	lbs

Sliding Stability
Factor of Safety applied to reduce Allow. Passive Press = 0.0

Stem Construction

		Top Stem	2nd	3rd
		Stem OK	Stem OK	Stem OK
Design Height Above Ftg	ft =	6.33	3.33	0.00
Wall Material Above "Ht"	=	Masonry	Masonry	Concrete
Thickness	=	8.00	12.00	12.00
Rebar Size	=	# 5	# 5	# 7
Rebar Spacing	=	32.00	16.00	16.00
Rebar Placed at	=	Edge	Edge	Edge
Design Data				
fb/FB + fa/Fa	=	0.681	0.583	0.656
Total Force @ Section	lbs =	490.7	1,001.0	3,576.0
Moment....Actual	ft-# =	783.5	2,229.5	11,990.0
Moment.....Allowable	ft-# =	1,121.1	3,820.4	18,288.0
Shear.....Actual	psi =	7.5	9.1	31.2
Shear.....Allowable	psi =	38.7	38.7	75.0
Wall Weight	psf =	78.0	124.0	150.0
Rebar Depth 'd'	in =	5.25	9.00	9.56
LAP SPLICE IF ABOVE	in =	30.00	30.00	41.16
LAP SPLICE IF BELOW	in =	30.00	23.40	
HOOK EMBED INTO FTG	in =			14.70
Masonry Data				
f'm	psi =	1,500	1,500	
Fs	psi =	24,000	24,000	
Solid Grouting	=	Yes	Yes	
Modular Ratio 'n'	=	21.48	21.48	
Short Term Factor	=	1.000	1.000	
Equiv. Solid Thick.	in =	7.60	11.60	
Masonry Block Type	=	Medium Weight		
Masonry Design Method	=	ASD		
Concrete Data				
f'c	psi =			2,500.0
Fy	psi =			60,000.0

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

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Registration #: RP-1110 5 RP9.00
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Cantilevered Retaining Wall Design

Code: IBC 2006

Footing Design Results

Footing Torsion, Tu = 87,644.90 ft-lbs
Footing Allow. Torsion, phi Tu = 61,618.79 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Summary of Overturning & Resisting Forces & Moments

Item		OVERTURNING Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	4,100.6	4.50	18,452.8
Surcharge over Heel	=			
Toe Active Pressure	=	-183.8	1.17	-214.4
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	3,916.8	O.T.M. =	18,238.4

Pier foundation used. Forces and moments displayed are applied at top of pier.

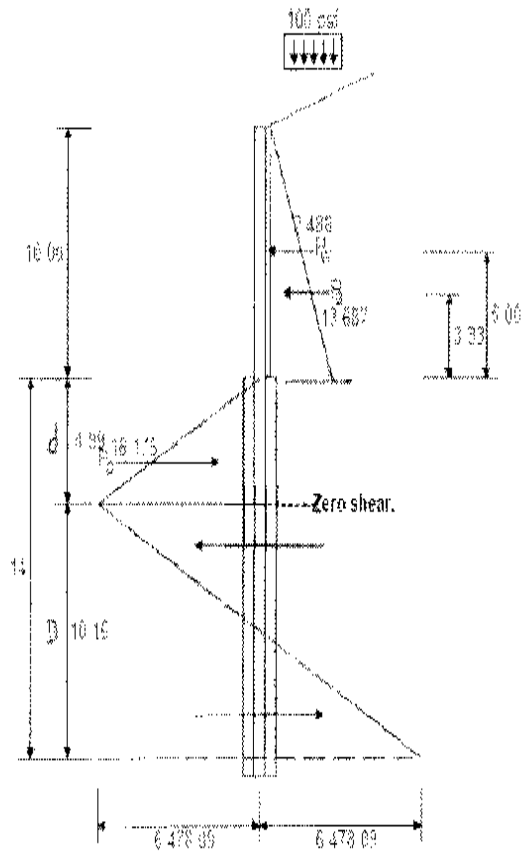
Item		RESISTING Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	2,200.0	4.00	8,800.0
Sloped Soil Over Heel	=	110.0	4.33	476.7
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	220.0	1.00	220.0
Surcharge Over Toe	=			
Stem Weight(s)	=	1,111.6	2.45	2,718.7
Earth @ Stem Transitions	=	171.2	2.83	485.2
Footing Weight	=	1,875.0	2.50	4,687.5
Key Weight	=			
Vert. Component	=			
Total	=	5,687.8 lbs	R.M. =	17,388.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

DESIGN EXAMPLE 13

Design Data



Soldier pile, cantilevered

Retained ht. = 10.0 ft.

Backfill slope = 18.4° (3:1)

Soil density = 110 pcf

Soil friction, $\phi = 34°$

G194Surcharge = 100 psf

Pile spacing = 8.0 ft.

K_a (horiz) = $\cos^2 18.4$

$$\frac{\cos 18.4 - \sqrt{\cos^2 - \cos^2 34}}{\cos 18.4 + \sqrt{\cos^2 - \cos^2 34}} = 0.31$$

$K_P = \tan^2\left(45 + \frac{34}{2}\right) - 3.54$

Passive = 3.54 x 110 = 389 psf

$P_A = \frac{0.31 \times 110 \times 10^2}{2} \times 8.0 \;\; 13{,}640 \text{ lbs}$

$P_w = 0.31 \times 100 \times 10 \times 8.0 = 2{,}480$ lbs.

Safety factor to apply to passive – 1.5

Drill hole diameter = 24"

Multiplier for passive wedge = 2.5

$P_p = P_A = P_w$ -- 13,6404 +2,480 = 16,120 lbs. $= \dfrac{389 \times 0.67 \times 2.0 \times 2.5 \times d^2}{2}$

$d = \sqrt{\dfrac{16{,}120 x 2}{389 x 0.67 \times 2.0 \times 2.5}}$ - 4.89 ft.

$M_{max} = P_A\left(\frac{H}{3} + \frac{2d}{3}\right) + P_w\left(\frac{H}{2} + \frac{2d}{3}\right)$

$= 13{,}640\,(3.33 + 3.33) + 2480\,(5 + 3.33) = 111{,}637$ ft-lbs.

Factored moment for LRFD = 111,174 x 1.6 – 177,878 ft-lbs.

Per AISC 13th Edition, 2007, LRFD

Assume lateral support (designer to assess)

Select W10 x 49

C_{mn} = 227,000 ft-lbs. > 177,878 ft-lbs.

M_{max} = resisted by moment couple = $F_p \frac{2}{3} x\, D$ $D = \frac{M_{max}}{0.67\, F_p}$

Mom. Resisting couple = (0.67D) $\left(\frac{2}{3} \times 389 \times 0.67 \times 2.0 \times 2.5 \times 4.98 \times 0.5\right) = D^2\ (1089)$

$$D = \sqrt{\frac{111{,}637}{1089}} = 10.12$$

Total embedment required = 4.99 + 10.12 = 15.1 ft.

Check Lagging at, say, 8 ft. depth

Assume $M = \frac{WL^2}{10} = \frac{[(8 \times 0.31 \times 110) + (0.31 \times 100)]8^2}{10} = 1944$ ft-lbs.

$S_{req'd} = \frac{1944 \times 12}{900} = 25.9$

Use 4 x 12 $\left(S = \frac{11.5 \times 3.5^2}{6} = 23.5\right)$ Consider OK

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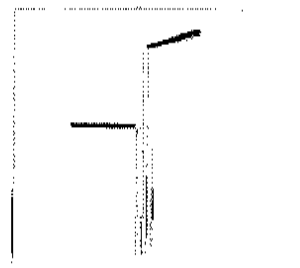
Soldier Pile Retaining Wall Design

Code: IBC 2006

Summary

Wall height (retained height), ft	10.00
Backfill slope	18.40
Soil Density, pcf	110.00
Soil Phi angle	34.00
Ka (horizontal)	0.31
Surcharge, psf	100
Allow. Passive	380
Apply S.F. to Passive	1.5
Pile Spacing, ft	8.0
Drilled Diameter, in.	24.00
Multiplier to Passive Wedge	2.5
Required Embedment, ft	15.14
Embedment Used, ft	16.00
Moment in Pile Max, ft-lbs	111,739
Max. Mom., Factored 1.6, LRFD	178,783
Soldier Beam Selection	W10x49
Lagging Depth, ft	0.00
Lagging Selection	4x12

Selected option to drill hole, insert soldier beam, and encase beam in lean concrete.



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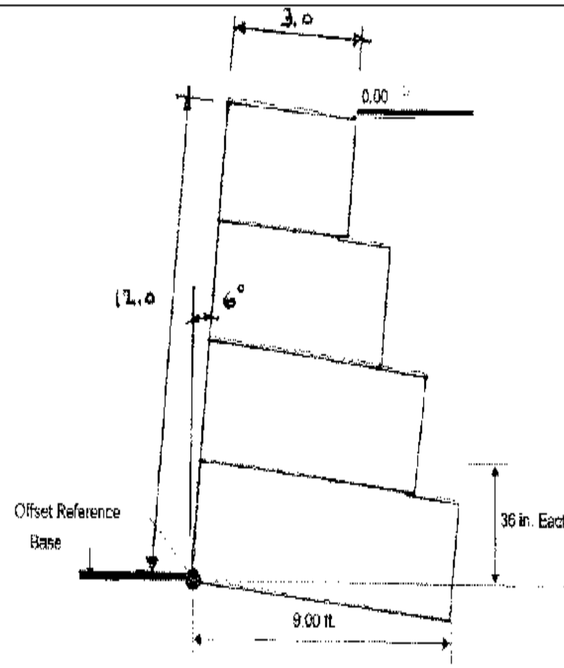

Gabion Wall

Criteria

Height of each course = 36"

Retained height = 12.0 ft.

Wall tilt from vert. = 6°

Surcharge = 0.0 psf

Density of cages (or blocks) = 120 pcf

Density of backfill = 110 pcf

Backfill slope: Level

Soil friction angle = 33°

Soil/block friction angle assumed zero (Conservative)

Allow. soil bearing = 2500 psf

Coef. of inter-block course friction = 0.70

Coef. of friction at base/soil interface = 0.45

Use Coulomb equation for K_a (horiz)

α angle for Coulomb = $90° - \left[tan^{-1}\left(\frac{4.5}{12.0}\right)\right] + 6.0 = 75.4°$

$K_a = 0.41$

K_a (horiz.) = 0.41 [cos (20.6 + 6.0)] = 0.37

Check Course #4 (top)

Resisting moment = $3.0 \times 4.5 \times 120 \times \frac{4.5}{2} \times \cos 6° = 3625$ ft.-lbs.

Lateral on course 4 = $0.37 \times 110\,(12.0\text{-}9.0)^2 \times 0.5 \times \cos 6° = 182$ lbs.

OTM for #4 = 182 x 3.0 x .33 = 182 ft. lbs.

Stability S.F. = $\dfrac{3625}{182} = 19.9$ OK

Sliding resistance = 0.70 x 3.0 x 4.5 x 120 = 1134 lbs.

S.F. for sliding = $\dfrac{1134}{182} = 6.2$ 0k

(Repeat this for courses 2 and 3)

Check Stability at Base Course

Distance from reference point to c.g. of courses:

$$\#4 = \left(9 \tan 6° + \frac{4.5}{2}\right) \cos 6° = 3.17 \text{ ft.}$$

$$\#3 = \left(6 \tan 6° + \frac{6}{2}\right) \cos 6° = 3.59 \text{ ft.}$$

$$\#2 = \left(3 \tan 6° + \frac{7.5}{2}\right) \cos 6° = 4.02 \text{ ft.}$$

Overall Resisting Moment

= 3.17 (3 x 4.5 x 120) + 3.59 (3 x 6 x 120)

+ 4.02 (3 x 7.5 x 120) x 3.0 x 9.0 x 120 = 38,324 ft-lbs.

Overturning Moment

$$= \left(0.38 \times 110 \times 12^2 \times 0.5 \times \frac{12}{3}\right) \cos^2 6° = 11{,}918 \text{ ft-lbs.}$$

Stability S.F. = $\dfrac{38{,}324}{11{,}918} = 3.22$

Sliding Resistance

Total vertical load = 4.5 x 3 x 120 + 6.0 x 3 x 120 + 7.5 x 3 x 120

+ 9.0 x 3 x 120 = 9,720 lbs.

Resistance = 9,720 x 0.45 = 4374 lbs.

Lateral force = (0.38 x 110 x 12^2 x 0.5) cos 6° = 2980 lbs.Sliding S.F. = $\dfrac{4374}{2980}$ 1.47

Soil Bearing

Total vertical load = 9,720 lbs.

Dist. To c. g. vertical load

$$\frac{38{,}324 - 11{,}918}{9720} = 2.72 \text{ ft.}$$

$$e = \frac{9}{2} - 2.72 = 1.78$$

$$\text{Soil bearing} = \frac{9720}{9} + \frac{9720 \, x \, 1.78 \, x \, 6}{9^2} = 2362 \text{ psf.}$$

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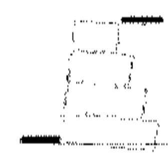
Gabion Retaining Wall Design

Code: IBC 2006

Summary

Input Values:

Course Height (Gabion/Block), in:	36.00
Retained Height, ft:	12.00
Wall Tilt from Vertical, degrees:	6
Surcharge, psf:	0
Density, Gabion Infill or Block, pcf:	120.00
Density of Backfill, pcf:	110.00
Backfill Slope, Degrees:	0.00
Soil Friction Angle, Phi:	34.00
Allow Soil Bearing, psf:	2,500
Coef. of Friction on Soil:	0.45
Coef. of Interblock Friction:	0.70



Thumbnail

Result Values

Ka (horiz):	0.34
Coulomb Alpha Angle:	69.44
Act. Soil Bearing Pressure, psf:	2,379
Lateral Force, Earth, lbs:	3,020
Lateral Force, Surcharge, lbs:	0
Total Lateral Force, lbs:	3,020
Resisting Moment, ft-lbs:	38,365
Overturning Moment, ft-lbs:	11,893
Overturning Ratio:	3.23
Sliding Ratio:	1.46

Course	Height	Offset	Length	Vertical	Dist	RM	Lateral	OTM	Stab. S.F.	Sliding S.F.
4	9.00	0.00	4.50	3625.0	3.18	3625	167.7	186	19.49	6.06
3	6.00	0.00	6.00	3703.0	3.61	10578	750.6	1487	7.11	3.42
2	3.00	0.00	7.50	6480.0	4.04	21832	1689.4	5017	4.35	2.69
1	0.00	0.00	9.00	8726.0	4.48	38365	3003.2	11893	3.23	1.40

Column Title Descriptions:

Course:	These are numbered in ascending order and cannot exceed 10
Height:	Measured from bottom of first (base) course.
Offset:	Measured from front edge bottom course.
Length:	Of cages or blocks in course.
Vertical:	Accumulated load from courses above.
Dist.:	From front edge bottom course to c.g. of course.
RM:	Resisting moment at level selected
Lateral:	Accumulated lateral from earth pressure and surcharge
OTM:	Accumulated overturning moment above Height
Stab. S.F.:	RM / OTM
Sliding S.F.:	Lateral / Vertical * (Coef. Interblock friction).

DESIGN EXAMPLE 14 — Report Printout

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Title : EX-14
Job # :
Description...

Dsgnr:

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Gabion Retaining Wall Design

Code: IBC 2006

Notes:

1. All courses are the same height and infill density.
2. If wall depth is uniform consider using segmental wall module, gravity, select "custom".
3. Concrete blocks may be used in lieu of Gabion cages.
4. Offset of successive layers limited to one-half course height. Earth side face flush.
5. Coulomb equation used for active pressure. Wall friction angle assumed zero.
6. If wall is tilted (battered), model effect by successive offsets (tan beta times course height).
7. This design not valid for reinforced soils (MSE). Use SRW module.
8. Vendor specifications may apply.

APPENDIX

Appendix A

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR SUBDIVISIONS			GROUP SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.
			GP	POORLY GRADED GRAVELS, OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.
	MORE THAN 50% OF COARSE FRACTION RETAINED ON A NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES.
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES.
	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES.
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	MORE THAN 50% OF COARSE FRACTION PASSING A NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND-SILT MIXTURES.
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES.
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50.	ML	INORGANIC SILTS, SANDY SILTS, AND CLAYEY SILTS OF LOW PLASTICITY.
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY; GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS.
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY.
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50.	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS. ELASTIC SILTS.
			CH	INORGANIC CLAYS OF HIGH PLASTICITY FAT CLAYS.
			OH	ORGANIC CLAYS AND SILTY CLAYS OF MEDIUM TO HIGH PLASTICITY.
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE CLASSIFICATIONS

SOIL CLASSIFICATION CHART

Appendix B - Summary of Design Formulas

CONCRETE (SD)

ϕ = .90 for flexure

= .75 for shear

= .55 for plain concr flexure/shear

$\rho_{bal} = .85 \frac{f'_c}{f_y} \beta \left(\frac{87{,}000}{87{,}000 + f_y}\right) [\beta = 0.85]$

$\rho_{max} = .75_{bal}$

$\rho_{min} = {}^{200}/_{f_y}$

E_s = 29,000,000 psi

$E_c = 57{,}000\sqrt{f'_c}$

$n = {}^{E_s}/_{E_c}$

$a = \frac{A_s f_y}{.85 f'_c b}$

The general solution for A_s (per CRSI) =

$$\frac{1.7 f'_c bd}{2 f_y} - \frac{1}{2}\sqrt{\frac{2.89 (f'_c bd)^2}{f_y^2} - \frac{6.8 f'_c b M_u}{\phi f_y^2}}$$

For b = 12", f'_y = 60 ksi, this reduces to:

A_s =

$$0.17\, f'_c d - \sqrt{.029 (f'_c d)^2 - .0063\, f'_c M_u}$$

$M_n = A_s f_y \left(d - {}^{a}/_{2}\right)$

$M_u \leq \phi M_n$

$$\ell_{hb} = \frac{0.02 f_y d_b \times 0.7}{\sqrt{f'_c}} \left(\frac{A_s \text{ req'd}}{A_s \text{ provided}}\right)$$

or .8d_b or 6"

*ℓ_{db} (#6 and smaller)

$$= \frac{0.024\, d_b f_y}{\sqrt{f'_c}} \left(\frac{A_s \text{ req'd}}{A_s \text{ provided}}\right)$$

*ℓ_{db} (#7 and larger)

$$= \frac{.03\, d_b f_y}{\sqrt{f'_c}} \left(\frac{A_s \text{ req'd}}{A_s \text{ provided}}\right)$$

* From ACI 12.2.3

Lap length Class B splice = 1.3 ℓ_{db}

$v_c = 2\sqrt{f'_c}\, bd$

Plain concr tension = $5\phi\sqrt{f'_c}$

Plain concr shear = $2\phi\sqrt{f'_c}$

MASONRY (WSD)

F_s = 0.5 f_y (24,000 psi max.)

E_s = 29,000,000 psi

E_m = 750 f'_m

$n = {}^{E_s}/_{E_m}$

F_b = .33 f'_m

$v_v = 1.0\sqrt{f'_m}$ (50 psi max)

$\rho = {}^{A_s}/_{bd}$

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

$j = 1 - {}^{k}/_{3}$

$M_s = F_s A_s jd$

$M_m = F_b bd^2 \left({}^{kj}/_{2}\right) = Kdb^2$

$v_a = {}^{V}/_{bd}$

l_d = .002$d_b F_s$ (but not less than 12")

MASONRY (LRFD)

$A = {}^{A_s f_y}/_{0.80 f'_m b} = 0.80$

$M_n = A_s f_y \left(d - {}^{a}/_{2}\right)$

ϕ = 0.90

Appendix C - Masonry Design Data

Rebar Position Depth for Masonry, Default Values.

Thickness	Rebar Depth (in) Center	Rebar Depth (in) Edge
6"	2.75"	2.75"
8"	3.75"	5.25"
10"	4.75"	7.25"
12"	5.75"	9.0"
14"	6.75"	11.0"
16"	7.75"	13.0"

Masonry Equivalent Solid Thickness (inches)

	Grout Spacing					
Thickness (inches)	8"	16"	24"	32"	40"	48"
6	5.6	4.5	4.1	3.9	3.8	3.7
8	7.6	5.8	5.2	4.9	4.7	4.6
10	9.6	7.2	6.3	5.9	5.7	5.5
12	11.6	8.5	7.5	7.0	6.7	6.5
14	13.6	9.9	8.7	8.1	7.6	7.4
16	15.6	11.6	10.1	9.5	8.6	8.3

Wall Thickness		Concrete Masonry Units											
		Lightweight 103 pcf				Medium Weight 115 pcf				Normal Weight 135 pcf			
		6"	8"	10"	12"	6"	8"	10"	12"	6"	8"	10"	12"
Solid Grouted Wall		52	75	93	118	58	78	98	124	63	84	104	133
Vertical Cored Grouted at:	16" o.c.	41	60	69	88	47	63	80	94	52	66	86	103
	24" o.c.	37	55	61	79	43	58	72	85	46	61	78	94
	32"o.c.	36	52	57	74	42	55	68	80	47	58	74	89
	40" o.c.	35	50	55	71	41	53	66	77	46	56	72	86
	48" o.c.	34	49	53	69	40	45	64	75	45	55	70	83

Appendix D - Development and Lap Lengths

Lap Splice Lengths(1) and Hooked Bar Embedments (inches)

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

(1) Min. lap for spliced bars, in., assumes f_y = 60 ksi, per ACI 316-05, Equation (12-1).

(2) 40 bar diameters for f_y = 40 ksi and 48 diameters for f_y = 60 ksi IBC '06-2107.5

(3) Min. 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 that IBC '06, 2107.5, modifies ACI 530-05, Section 2.1.10.7.1.1 which has the effect of deleting the following onerous development length equation (2-9) in ACI-530-05:

$$\ell_d = \frac{0.13\, d_b^2\, f_y\, \gamma}{K\sqrt{f_m^1}}$$

γ – 1.0 for #3,4,5 bars, 1.4 for #6, 7, and 1.5 for #8

K = Masonry cover but not less than 5 d_b

This requirement resulted in much longer lap lengths and has met with considerable objection.

Appendix E - Sample Construction Notes

Brief specifications, or notes, should accompany any retaining wall design. A checklist for items to include:

Reference to foundation investigation report recommendations (if applicable)
Excavating / grading requirements
Concrete strength
Masonry
Mortar
Grout
Reinforcing, including placement requirements
Soil bearing value and special requirements
Inspections
Drainage

And here are a few additional notes that will help solve problems and keep you out of trouble:

1. Should a discrepancy arise between the drawings and field conditions, or where a detail is doubtful of interpretation or an unanticipated field condition be encountered, the structural engineer shall be called right away for procedure to be followed which shall be confirmed in writing by the structural engineer with copies to all parties.

2. Wherever there is a conflict between details and specifications, or between details, or where doubtful of interpretation, the most restrictive shall govern, as determined by the structural engineer.

3. The contactor and each subcontractor shall visit the site and consider field conditions affecting the work depicted on the plans, and his submission of a bid indicates his acceptance of such conditions.

4. The contractor shall assure that each subcontractor has copies of latest plan revisions and is kept current with any change orders or directives affecting the subcontractors work.

And your experience will add more!

Appendix F. - Conversion Factors

English – S.I. – Metric Conversions		
Multiply	**by**	**to get**
inches	2.54	cm (centimeters)
feet	0.305	m (meters)
centimeters	0.394	inches
centimeters	10	mm (millimeters)
meters	3.28	feet
psf	47.9	kPa (pascals)
psi	6.89	kPa (kilopascals)
pcf	16.0	kg/m^3 (kilograms per cubic meter)
psf/ft	0.157	kPa/m (kilopascals per meter)
in-lbs	0.113	Nm (newton meters)
ft-lbs	1.36	Nm (newton meters)
pounds	4.45	N (newtons)
kip	4.45	kN (Kilo Newtons)
lbs per lin ft	1.49	kg/m (kilograms per meter)
inches	25.4	mm (millimeters)
milimeters	.039	inches
Pascals	1.0	Nm^2 (newtons per square meter)
Newtons/m^2	1.0	Pa (pascals)

Common Equivalents	
English	**S.I.**
1,500 psi	= 10.34 mPa
2,000 psi	= 13.8 mPa
2,500,psi	· 17.24 mPa
3,000 psi	20.7 mPa
24,000 psi	= 165 mPa
60,000 psi	= 414 mPa
100 psf	= 4.788 mPa
1,000 psf	= 47.9 mPa

Appendix G - Reinforcing Bar US/SI Conversions

Reinforcing Bar Conversions ("soft" metric)			
U.S. Bar No.	Metric Bar No.	Diameter in/mm	Area in^2/mm^2
3	10	0.379/9.5	0.11/71
4	13	0.50/12.7	0.20/129
5	16	0.625/15.9	0.31/199
6	19	0.44/19.1	0.44/284
7	22	0.875/22.2	0.60/387
8	25	1.00/25.4	0.79/510
9	29	1.125/28.7	1.00/645
10	32	1.25/32.3	1.27/819
11	36	1.375/38.8	1.56/1006

Appendix H. - Reference Bibliography

1. International Building Code, 2009, International Code Council, Inc., Falls Church, VA.
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Appendix I: Notations & Symbols

A = depth of equivalent rectangular stress block for strength design.

AASHTO = American Association of State Highway & Transportation Officials

A = ground acceleration. (symbol varies)

A_s = effective cross-sectional area of reinforcement in a column or flexural member, square inches.

ACI = American Concrete Institute.

AISC = American Concrete Masonary Association.

ASCE = American Society of Civil Engineers.

ASD = Allowable Stress Design

ASTM = American Society for Testing and Materials.

b = width of rectangular member.

c = coefficient that determines the distance to the neutral axis in a beam in strength design.

CBC = California Building Code

cm = centimeter

CMU = Concrete Masonry Unit.

d = depth of reinforcing from compression edge.

DL = dead load.

e = eccentricity measured from the vertical axis of a section to the resultant force.

E_m = modulus of elasticity of masonry in compression, psi.

E_s = modulus of elasticity of steel = 29,000,000 psi.

f_a = actual axial compressive stress due to axial load psi.

f_b = actual flexural stress in the extreme fiber due to bending moment, psi.

f_c = compressive stress in concrete in flexure, psi.

f'_c = specified compressive strength of concrete, psi.

f'_m = allowable compressive strength of masonry, psi.

f_r = modulus of rupture, psi.

f_s = computed stress in reinforcement due to design loads, psi.

f'_s = stress in compressive reinforcement in flexural members, psi.

f_y = yield strength of reinforcement, psi.

f_t = flexural tensile stress in masonry, psi.

f_v = actual shear stress, psi.

F_a = allowable axial compressive stress, psi.

F_b = allowable flexural compressive stress, psi.

IBC = International Building Code.

ICC = International Code Council

I_g, = gross section moment of inertia, in^4.

I_{cr} = moment of inertia of cracked section, in^4.

K_a = active earth pressure cueficient, static

K_{ae} = active earth pressure, static and seismic.

K_{ah} = coefficients for seismic lateral earth pressure of backfill against cantilever retaining wall.

Kip = 1000 pounds.

kN = kilonewtons.

K_o = At-rest earth pressure cuficient.

kPa = kilopascals

ksi = Kips per square inch

ksf = Kips per square foot

Kg = kilogram.

l_d = required development length of the reinforcement.

L_{dh} = basic development length, inches.

L_{hd} = hooked bar development length, inches

LL = live load.

LRFD = Load Resistance Factor Desgin.

MIA = Masonery Institute of America

MSJC = Masonary Standards Joint Committee

M_m = the moment of the compressive force in the masonry about the centroid of the tensile force in the reinforcement.

M_n = nominal moment strength of a masonry section.

MPa = megapascals.

M_R = resisting moment.

M_s = the moment of the tensile force in the reinforcement about the centroid of the compressive force in masonry.

NCMA = National Concrete Masonery Association.

OTM = Overturning Moment.

Pa = Pascals.

pcf = pounds per square foot per foot or pounds per cubic foot.

plf = pounds per linear foot.

psi = pounds per square inch.

psf = pounds per square foot.

RM = Resisting Moment

r = radius of gyration.

SF = Safety Factor

SI = International Systems of Measurements as adopted by the General Conference of Weights and Measures.

SRW = Segmental Retaining Wall

v = shear stress, psi

Vu = factored shear stress, psi.

Vm = allowable shear stress for masonry, psi.

V = total design shear force, lbs.

V_u = factored shear force at section.

w = uniformly distributed load.

W_p = weight of wall or component.

α = In Coulomb equation, clockwise angle from horiztonal to back face of wall (90^0 if wall is vertical).

β = angle of the backfill slope from a horizontal level plane.

γ = unit weight of soil, pcf.

δ = angle of the wall friction to a horizontal level plane.

$\triangle$ = deflection of element.

μ = coefficient of sliding friction.

ϕ = angle of internal friction of soil degrees.

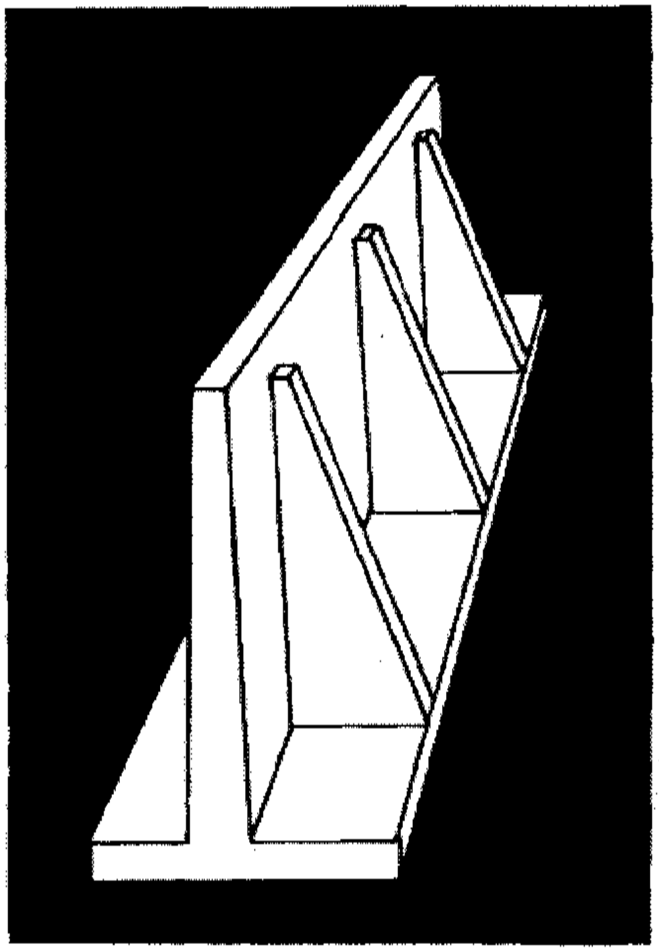
NOTE: SOME SYMBOLS MAY HAVE DIFFERENT MEANINGS IN DIFFERENT CONTEXTS.

Appendix J: Moments and Reactions for Rectangular Plates

These four pages are adopted from a U.S. Department of the Interior, Bureau of Reclamation, Water Resources publication: Engineering Monograpoh No 27, prepared by W.T. Moody. Find it in a library near you at http://www.worldcat.org/oclc/56307609&referer=brief_results.

A WATER RESOURCES TECHNICAL PUBLICATION

ENGINEERING MONOGRAPH NO. 27



Moments and Reactions for Rectangular Plates

UNITED STATES DEPARTMENT
OF THE INTERIOR
BUREAU OF RECLAMATION

A WATER RESOURCES TECHNICAL PUBLICATION

Engineering Monograph No. 27

Moments and Reactions for Rectangular Plates

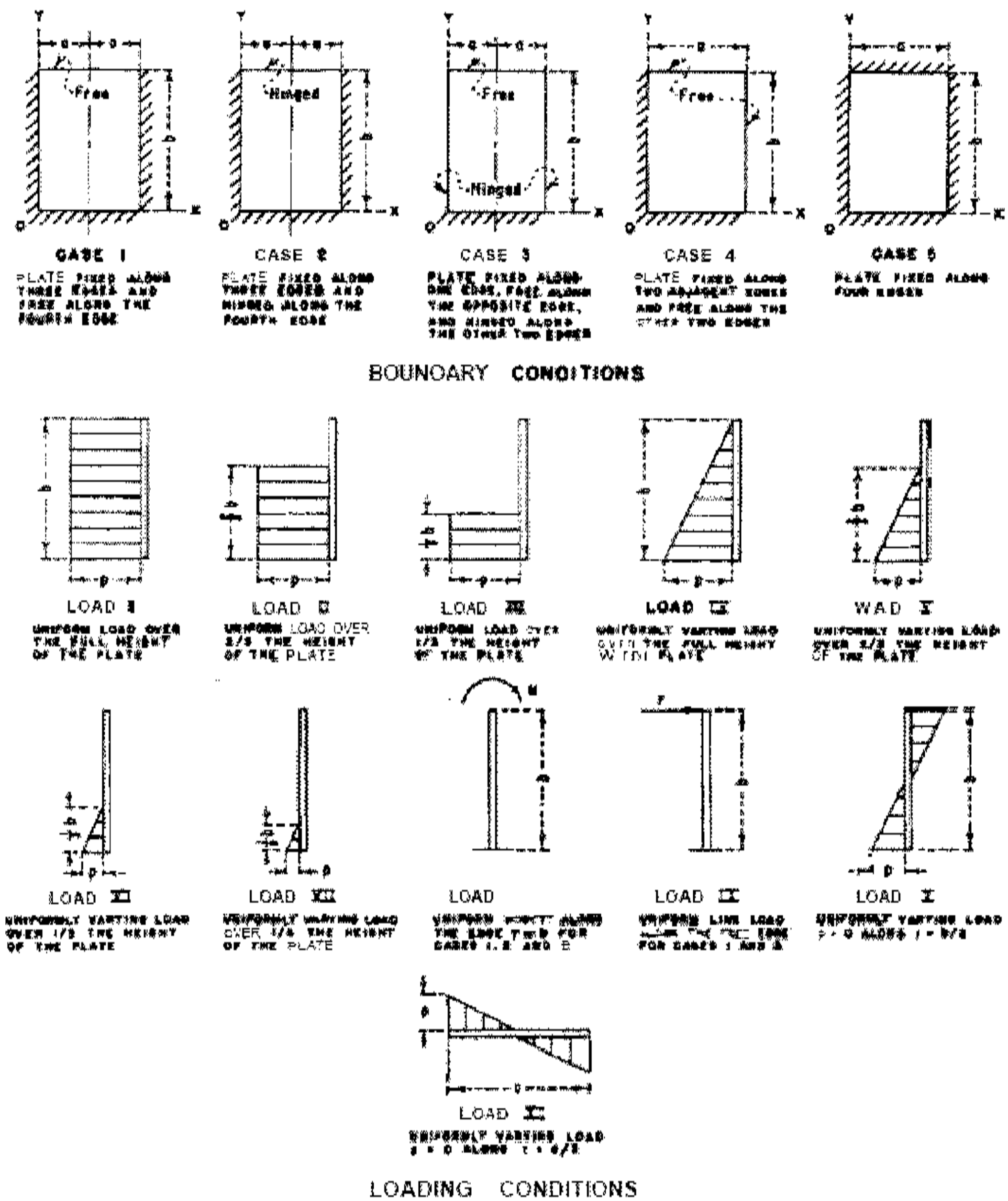
By W. T. MOODY

Division of Design

Denver, Colorado

United States Department of the Interior

BUREAU OF RECLAMATION



NOTES

The various cases are analyzed for the indicated ratios of a/b.

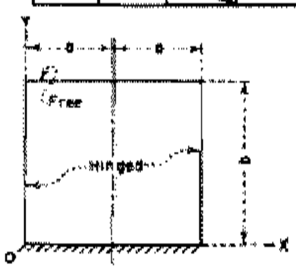
Cases 1, 2, and 3: 1/8, 1/4, 3/8, 1/2, 3/4, 1, and 3/2
Case 4: 1/8, 1/4, 3/8, 1/2, 3/4, and 1.
Case 5: [illegible], and 1.

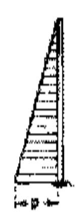
All results are based on a Poisson's ratio of 0.2.

INDEX OF BOUNDARY AND LOADING CONDITIONS

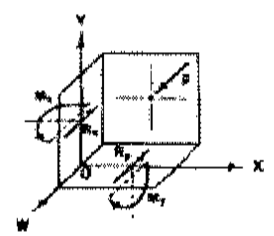
—FRONTISPIECE

	y/b	R_x \ x/a	M_x 0	M_x 0.2	M_x 0.4	M_x 0.6	M_x 0.8	M_x 1.0	M_y 0	M_y 0.2	M_y 0.4	M_y 0.6	M_y 0.8	M_y 1.0
$a/b = 1/8$	1.0	+.0094	0	-.0003	-.0006	-.0008	-.0010	-.0010	0	0	0	0	0	0
	0.8	+.0253	0	-.0006	-.0011	-.0015	-.0017	-.0017	0	-.0001	-.0002	-.0002	-.0002	-.0003
	0.6	+.0500	0	-.0011	-.0020	-.0026	-.0030	-.0031	0	-.0002	-.0004	-.0005	-.0006	-.0006
	0.4	+.0758	0	-.0016	-.0028	-.0037	-.0042	-.0044	0	-.0004	-.0007	-.0009	-.0011	-.0011
	0.2	+.0833	0	-.0015	-.0028	-.0036	-.0041	-.0042	0	-.0006	-.0010	-.0013	-.0015	-.0016
	0	+.0043	0	+.0004	+.0008	+.0010	+.0012	+.0012	0	+.0021	+.0039	+.0051	+.0058	+.0061
		R_x \ R_y	+.0043	+.0775	+.1160	+.1417	+.1551	+.1591						
$a/b = 1/4$	1.0	+.0249	0	-.0022	-.0040	-.0054	-.0062	-.0065	0	0	0	0	0	0
	0.8	+.0548	0	-.0028	-.0051	-.0067	-.0077	-.0081	0	-.0004	-.0006	-.0008	-.0010	-.0010
	0.6	+.1014	0	-.0040	-.0070	-.0091	-.0104	-.0108	0	-.0012	-.0021	-.0028	-.0032	-.0033
	0.4	+.1306	0	-.0048	-.0078	-.0100	-.0113	-.0117	0	-.0019	-.0034	-.0046	-.0052	-.0055
	0.2	+.1003	0	-.0033	-.0053	-.0065	-.0071	-.0072	0	-.0018	-.0031	-.0041	-.0046	-.0048
	0	-.0513	0	+.0014	+.0025	+.0033	+.0038	+.0040	0	+.0071	+.0127	+.0167	+.0191	+.0195
		R_x \ R_y	-.0513	+.1404	+.2060	+.2476	+.2668	+.2734						
$a/b = 3/8$	1.0	+.0631	0	-.0054	-.0099	-.0133	-.0154	-.0160	0	0	0	0	0	0
	0.8	+.0852	0	-.0080	-.0108	-.0142	-.0163	-.0170	0	-.0011	-.0020	-.0026	-.0030	-.0031
	0.6	+.1388	0	-.0071	-.0122	-.0156	-.0176	-.0183	0	-.0027	-.0050	-.0066	-.0076	-.0079
	0.4	+.1701	0	-.0070	-.0116	-.0143	+.0158	-.0162	0	-.0038	-.0069	-.0091	-.0104	-.0108
	0.2	+.0890	0	-.0044	-.0064	-.0071	-.0074	-.0074	0	-.0021	-.0035	-.0040	-.0043	-.0044
	0	-.0960	0	+.0025	+.0045	+.0059	+.0068	+.0071	0	+.0124	+.0225	+.0298	+.0339	+.0353
		R_x \ R_y	-.0960	+.1884	+.2717	+.3203	+.3456	+.3538						
$a/b = 1/2$	1.0	+.1322	0	-.0087	-.0159	-.0210	-.0241	-.0252	0	0	0	0	0	0
	0.8	+.1049	0	-.0090	-.0159	-.0207	-.0235	-.0244	0	-.0021	-.0038	-.0050	-.0058	-.0061
	0.6	+.1584	0	-.0096	-.0161	-.0201	-.0224	-.0231	0	-.0045	-.0080	-.0106	-.0121	-.0127
	0.4	+.1757	0	-.0087	-.0136	-.0161	-.0173	-.0176	0	-.0053	-.0092	-.0119	-.0134	-.0139
	0.2	+.0595	0	-.0047	-.0060	-.0060	-.0058	-.0057	0	-.0011	-.0010	-.0004	+.0002	+.0005
	0	-.1169	0	+.0040	+.0070	+.0091	+.0103	+.0107	0	+.0198	+.0351	+.0455	+.0515	+.0535
		R_x \ R_y	-.1169	+.2296	+.3241	+.3786	+.4050	+.4130						
$a/b = 3/4$	1.0	+.3017	0	-.0132	-.0230	-.0292	-.0326	-.0337	0	0	0	0	0	0
	0.8	+.1125	0	-.0129	-.0214	-.0265	-.0291	-.0299	0	-.0036	-.0065	-.0086	-.0098	-.0102
	0.6	+.1595	0	-.0126	-.0194	-.0227	-.0242	-.0246	0	-.0066	-.0124	-.0145	-.0162	-.0168
	0.4	+.1576	0	-.0104	-.0142	-.0151	-.0152	-.0152	0	-.0080	-.0093	-.0109	-.0114	-.0115
	0.2	+.0115	0	-.0043	-.0036	-.0021	-.0011	-.0007	0	+.0030	+.0080	+.0128	+.0161	+.0172
	0	-.1259	0	+.0070	+.0119	+.0151	+.0168	+.0174	0	+.0348	+.0594	+.0753	+.0842	+.0871
		R_x \ R_y	-.1259	+.2975	+.3991	+.4504	+.4744	+.4816						
$a/b = 1$	1.0	+.4312	0	-.0134	-.0247	-.0293	-.0312	-.0317	0	0	0	0	0	0
	0.8	+.1046	0	-.0147	-.0224	-.0256	-.0268	-.0270	0	-.0043	-.0075	-.0093	-.0103	-.0106
	0.6	+.1468	0	-.0139	-.0191	-.0205	-.0205	-.0204	0	-.0073	-.0115	-.0136	-.0143	-.0145
	0.4	+.1380	0	-.0109	-.0125	-.0117	-.0107	-.0103	0	-.0050	-.0059	-.0048	-.0035	-.0029
	0.2	-.0108	0	-.0032	-.0006	+.0020	+.0036	+.0041	0	+.0082	+.0186	+.0275	+.0331	+.0350
	0	-.1123	0	+.0086	+.0159	+.0198	+.0213	+.0224	0	+.0442	+.0797	+.0990	+.1094	+.1129
		R_x \ R_y	-.1123	+.3359	+.4399	+.4841	+.5035	+.5090						
$a/b = 3/2$	1.0	+.3418	0	-.0175	-.0229	-.0223	-.0205	-.0197	0	0	0	0	0	0
	0.8	+.0931	0	-.0164	-.0201	-.0188	-.0168	-.0160	0	-.0045	-.0069	-.0073	-.0070	-.0068
	0.6	+.1335	0	-.0149	-.0160	-.0135	-.0113	-.0105	0	-.0073	-.0086	-.0073	-.0056	-.0050
	0.4	+.1257	0	-.0104	-.0083	-.0050	-.0027	-.0019	0	-.0023	+.0026	+.0086	+.0129	+.0144
	0.2	-.0072	0	-.0008	+.0043	+.0080	+.0101	+.0108	0	+.0178	+.0369	+.0501	+.0579	+.0806
	0	-.0724	0	+.0139	+.0218	+.0258	+.0279	+.0285	0	+.0594	+.1079	+.1288	+.1594	+.1426
		R_y	-.0724	+.3914	+.4785	+.5060	+.5149	+.5169						





Moment = (Coefficient) (pb^2)
Reaction = (Coefficient) (pb)



POSITIVE SIGN CONVENTION

FIGURE 21.—*Plate fixed along one edge—Hinged along two opposite edges, moment and reaction coefficients, Load IV, uniformly varying load.*