

Retaining walls

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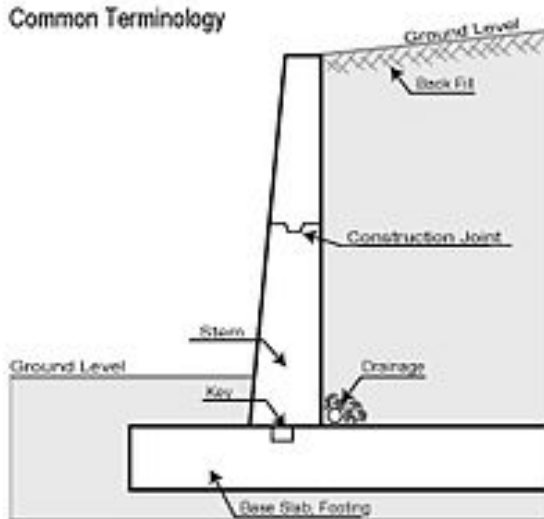
Retaining walls are structures designed to restrain soil to unnatural slopes. They are used to bound soils between two different elevations often in areas of terrain possessing undesirable slopes or in areas where the landscape needs to be shaped severely and engineered for more specific purposes like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

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Definition

A **retaining wall** is a structure designed and constructed to resist the lateral pressure of soil when there is a desired change in ground elevation that exceeds the angle of repose of the soil.

A basement wall is thus one kind of retaining wall. But the term usually refers to a cantilever retaining wall, which is a freestanding structure without lateral support at its top. These are cantilevered from a footing and rise above the grade on one side to retain a higher level grade on the opposite side. The walls must resist the lateral pressures generated by loose soils or, in some cases, water pressures



Every retaining wall supports a “wedge” of soil. The wedge is defined as the soil which extends beyond the failure plane of the soil type present at the wall site, and can be calculated once the soil friction angle is known. As the setback of the wall increases, the size of the sliding wedge is reduced. This reduction lowers the pressure on the retaining wall.

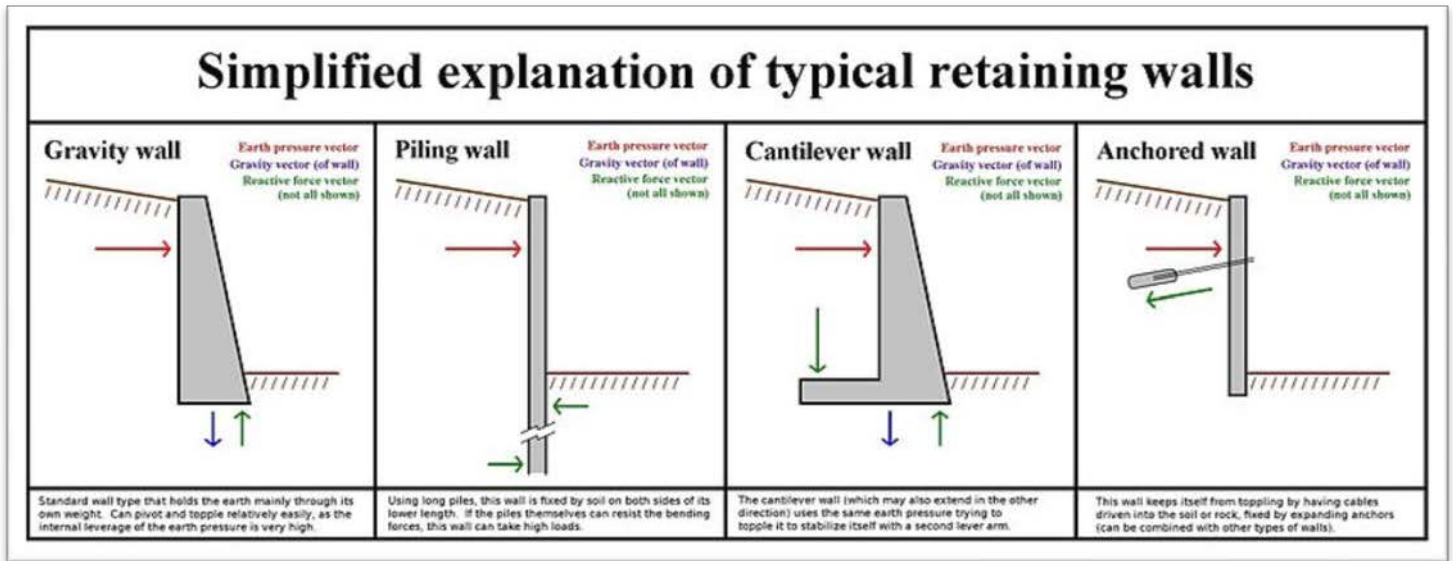
The most important consideration in proper design and installation of retaining walls is to recognize and counteract the tendency of the retained material to move downslope due to gravity. This creates lateral earth pressure behind the wall which depends on the angle of internal friction (ϕ) and the cohesive strength (c) of the retained material, as well as the direction and magnitude of movement the retaining structure undergoes.

Lateral earth pressures are zero at the top of the wall and - in homogenous ground - increase proportionally to a maximum value at the lowest depth. Earth pressures will push the wall forward or overturn it if not properly addressed. Also, any groundwater behind the wall that is not dissipated by a drainage system causes hydrostatic pressure on the wall. The total pressure or thrust may be assumed to act at one-third from the lowest depth for lengthwise stretches of uniform height.

Unless the wall is designed to retain water, It is important to have proper drainage behind the wall in order to limit the pressure to the wall's design value. Drainage materials will reduce or eliminate the hydrostatic pressure and improve the stability of the material behind the wall. Drystone retaining walls are normally self-draining.

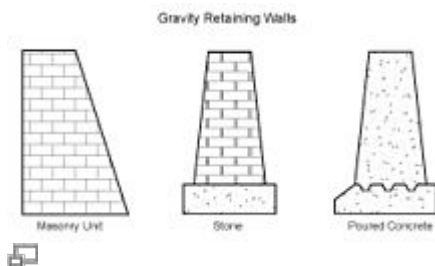
As an example, the International Building Code requires retaining walls to be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift; and that they be designed for a safety factor of 1.5 against lateral sliding and overturning.

Types of retaining wall



Various types of retaining walls

Gravity



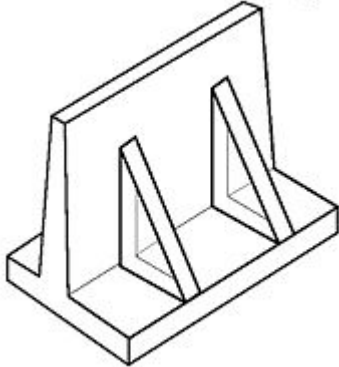
Construction types of gravity retaining walls

Gravity walls depend on their mass (stone, concrete or other heavy material) to resist pressure from behind and may have a 'batter' setback to improve stability by leaning back toward the retained soil. For short landscaping walls, they are often made from mortarless stone or segmental concrete units (masonry units). Dry-stacked gravity walls are somewhat flexible and do not require a rigid footing in frost areas. Home owners who build larger gravity walls that do require a rigid concrete footing can make use of the services of a professional excavator, which will make digging a trench for the base of the gravity wall much easier.

Earlier in the 20th century, taller retaining walls were often gravity walls made from large masses of concrete or stone. Today, taller retaining walls are increasingly built as composite gravity walls such as: geosynthetics such as Geoweb cellular confinement earth retention or with precast facing; gabions (stacked steel wire baskets filled with rocks); crib walls (cells built up log cabin style from precast concrete or timber and filled with soil); or soil-nailed walls (soil reinforced in place with steel and concrete rods).

Cantilevered

Counterfort or Buttress Retaining Wall



Counterfort/Buttress on Cantilevered Wall

Cantilevered retaining walls are made from an internal stem of steel-reinforced, cast-in-place concrete or mortared masonry (often in the shape of an inverted T). These walls cantilever loads (like a beam) to a large, structural footing, converting horizontal pressures from behind the wall to vertical pressures on the ground below. Sometimes cantilevered walls are buttressed on the front, or include a counterfort on the back, to improve their strength resisting high loads. Buttresses are short wing walls at right angles to the main trend of the wall. These walls require rigid concrete footings below seasonal frost depth. This type of wall uses much less material than a traditional gravity wall.

Sheet piling

Sheet pile retaining walls are usually used in soft soils and tight spaces. Sheet pile walls are made out of steel, vinyl or wood planks which are driven into the ground. For a quick estimate the material is usually driven 1/3 above ground, 2/3 below ground, but this may be altered depending on the environment. Taller sheet pile walls will need a tie-back anchor, or "dead-man" placed in the soil a distance behind the face of the wall, that is tied to the wall, usually by a cable or a rod. Anchors are then placed behind the potential failure plane in the soil.

Anchored

An anchored retaining wall can be constructed in any of the aforementioned styles but also includes additional strength using cables or other stays anchored in the rock or soil behind it. Usually driven into the material with boring, anchors are then expanded at the end of the cable, either by mechanical means or often by injecting pressurized concrete, which expands to form a bulb in the soil. Technically complex, this method is very useful where high loads are expected, or where the wall itself has to be slender and would otherwise be too weak.

Alternative retaining techniques

Cellular Confinement

Cellular confinement systems, first marketed by Presto Geosystems as Geoweb®, have become increasingly popular for earth retention applications. They can be constructed as a gravity wall or a "geogrid" wall which

consists of vertical layers of geocells with geogrid reinforcement installed behind the face of the wall every few layers of the geocell depending on design.

Soil nailing

Soil nailing is a technique in which soil slopes, excavations or retaining walls are reinforced by the insertion of relatively slender elements - normally steel reinforcing bars. The bars are usually installed into a pre-drilled hole and then grouted into place or drilled and grouted simultaneously. They are usually installed untensioned at a slight downward inclination. A rigid or flexible facing (often sprayed concrete) or isolated soil nail heads may be used at the surface.

Soil-strengthened

A number of systems exist that do not consist of just the wall, but reduce the earth pressure acting directly on the wall. These are usually used in combination with one of the other wall types, though some may only use it as facing, *i.e.*, for visual purposes.

Gabion meshes

This type of soil strengthening, often also used without an outside wall, consists of wire mesh 'boxes' into which roughly cut stone or other material is filled. The mesh cages reduce some internal movement/forces, and also reduce erosive forces.

Mechanical stabilization

Mechanically stabilized earth

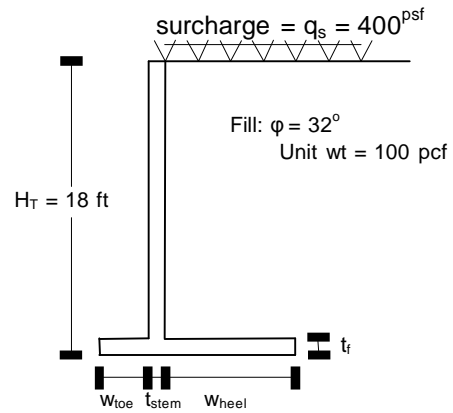
Mechanically stabilized earth, also called MSE, is soil constructed with artificial reinforcing via layered horizontal mats (geosynthetics) fixed at their ends. These mats provide added internal shear resistance beyond that of simple gravity wall structures. Other options include steel straps, also layered. This type of soil strengthening usually needs outer facing walls (S.R.W.'s - Segmental Retaining Walls) to affix the layers to and vice versa.

The wall face is often of precast concrete units that can tolerate some differential movement. The reinforced soil's mass, along with the facing, then acts as an improved gravity wall. The reinforced mass must be built large enough to retain the pressures from the soil behind it. Gravity walls usually must be a minimum of 50 to 60 percent as deep or thick as the height of the wall, and may have to be larger if there is a slope or surcharge on the wall.

Retaining Wall Design Example

Design a reinforced concrete retaining wall for the following conditions.

$$f_y = 60 \text{ ksi}$$



Natural Soil: $\phi = 32^\circ$
allowable bearing pressure = 5000psf

Development of Structural Design Equations. In this example, the structural design of the three retaining wall components is performed by hand. Two equations are developed in this section for determining the thickness & reinforcement required to resist the bending moment in the retaining wall components (stem, toe and heel).

Equation to calculate effective depth, d: Three basic equations will be used to develop an equation for d.

$$M_u = \phi M_n$$

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$M_u = \phi A_s f_y \left(d - \frac{a}{2} \right) \quad [Eqn 1]$$

$$C = T, \quad 0.85 f'_c a b = A_s f_y$$

$$A_s = 0.85 \frac{f'_c}{f_y} a b \quad [Eqn 2]$$

$$\text{strain compatibility: } \frac{0.003}{a/\beta_1} = \frac{\epsilon_s + 0.003}{d}, \quad \frac{a}{d} = \frac{0.003}{\epsilon_s + 0.003} \beta_1$$

Assuming $\beta_1 = 0.85$,

ϵ_s	a/d
0.005	0.319
0.00785	0.235
0.010	0.196

and choosing a value for ϵ_s in about the middle of the practical design range,

$$\frac{a}{d} = 0.235, \quad a = 0.235 d \quad [Eqn 3]$$

Substituting Eqn. 2 into Eqn. 1:

$$M_u = \phi \left(0.85 \frac{f'_c}{f_y} ab \right) f_y \left(d - \frac{a}{2} \right)$$

And substituting Eqn. 3 into the above:

$$M_u = \phi 0.85 \frac{f'_c}{f_y} 0.235d b f_y \underbrace{\left(d - \frac{0.235d}{2} \right)}_{0.883d}$$

Inserting the material properties: $f'_c = 3 \text{ ksi}$ and $f_y = 60 \text{ ksi}$, and $b = 12 \text{ in}$ (1-foot-wide strip of wall, in the direction out of the paper).

$$M_u = 0.90(0.85) 3^{ksi} (12 \text{ in}) (0.235)(0.883)d^2$$

$$M_u = 5.71 \frac{k}{in} d^2$$

Equation for area of reinforcement, A_s . The area of reinforcement required is calculated from Eqn. 1:

$$M_u = \phi A_s f_y 0.883d = 0.90 A_s 60^{ksi} 0.883d$$

$$M_u = 47.7^{ksi} A_s d$$

Design Procedure

1. Determine H_T . Usually, the top-of-wall elevation is determined by the client. The bottom-of-wall elevation is determined by foundation conditions.

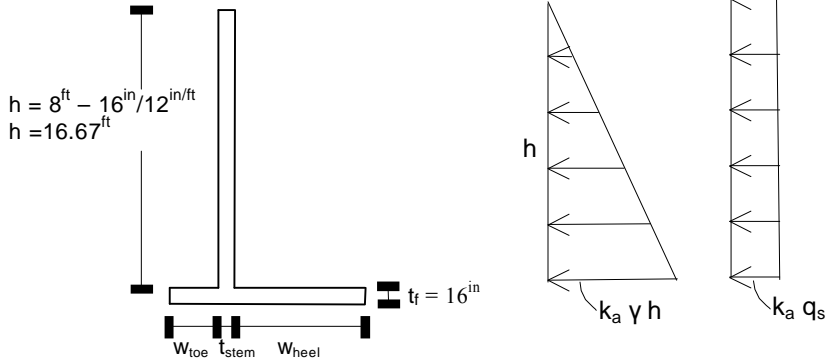
$$\underline{H_T = 18 \text{ feet.}}$$

2. Estimate thickness of base. $t_f \approx 7\%$ to 10% H_T (12" minimum)

$$T_f = 0.07 (18' \times 12"/') = 15.1"$$

$$\text{use } \underline{t_f = 16"}$$

3. Design stem (t_{stem} , $A_{S_{stem}}$). The stem is a vertical cantilever beam, acted on by the horizontal earth pressure.



calc. d:

$$P_{fill} = \frac{1}{2} (k_a \gamma h) h \quad (1\text{ ft out of page})$$

$$k_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1 - \sin(32^\circ)}{1 + \sin(32^\circ)} = 0.31$$

$$P_{fill} = \frac{1}{2} (0.31)(100\text{pcf})(16.67\text{ft})^2 \quad (1\text{ft}) = 4310\text{lb}$$

$$P_{sur} = k_a q_{sur} h \quad (1\text{ft}) = 0.31(400\text{psf})(16.67\text{ft})(1\text{ft}) = 2070\text{lb}$$

$$M_u = (\text{Earth Pressure Load Factor})(P_{fill})\left(\frac{h}{3}\right) + (\text{Live Load Factor})(P_{sur})\left(\frac{h}{2}\right)$$

$$M_u = (1.6)(4310\text{lb})\left(\frac{16.67\text{ft}}{3}\right) + (1.6)(2070\text{lb})\left(\frac{16.67\text{ft}}{2}\right) = 65.9\text{k-ft}$$

$$M_u = 5.71\frac{\text{k}}{\text{in}} d^2$$

$$65.9\text{k-ft} \left(12\frac{\text{in}}{\text{ft}}\right) = 5.71\frac{\text{k}}{\text{in}} d^2, \quad d = 11.8\text{in}$$

$$t_{stem} = 11.8\text{in} + 2\text{in cover} + \frac{1}{2}(1.0\text{in}) = 14.3\text{in}, \quad (\text{assume \#8 bars})$$

$$d = 15\text{in} - 2\text{in} - 0.5\text{in} = 12.5\text{in}$$

use $t_{stem} = 15\text{in}$

calc. A_s :

$$M_u = 47.7^{ksi} A_s d$$

$$65.9^{k-ft} \left(12 \frac{in}{ft}\right) = 47.7^{ksi} A_s (12.5^{in}), \quad A_s = 1.33 in^2$$

$$A_s \text{ of one \#8 bar} = 0.79 in^2$$

$$\frac{0.79 \frac{in^2}{bar}}{1.33 \frac{in^2}{ft \text{ of wall}}} 12 \frac{in}{ft} = 7.13 \frac{in}{bar}$$

use #8 @ 6in

4. Choose Heel Width, w_{heel} Select w_{heel} to prevent sliding. Use a key to force sliding failure to occur in the soil (soil-to-soil has higher friction angle than soil-to-concrete).

Neglect soil resistance in front of the wall.

$$\text{set } \frac{F_{resist}}{FS} = F_{sliding}$$

FS = Factor of Safety = 1.5 for sliding

$F_{resist} = (\text{Vertical Force})(\text{coefficient of friction})$

$$F_{resist} = W_T (\tan \phi_{natural soil})$$

$$\tan \phi_{natural soil} = \tan(32^\circ) = 0.62$$

$$W_T = W_{fill} + W_{stem} + W_{found}$$

$$W_{fill} = (100 pcf)(16.67^{ft})(w_{heel})(1^{ft}) = 1670 \frac{lb}{ft} w_{heel}$$

$$W_{stem} = (150 pcf)(16.67^{ft}) \left(\frac{12^{in} + 15^{in}}{2} \frac{1^{ft}}{12^{in}} \right) (1^{ft}) = 2810^{lb}$$

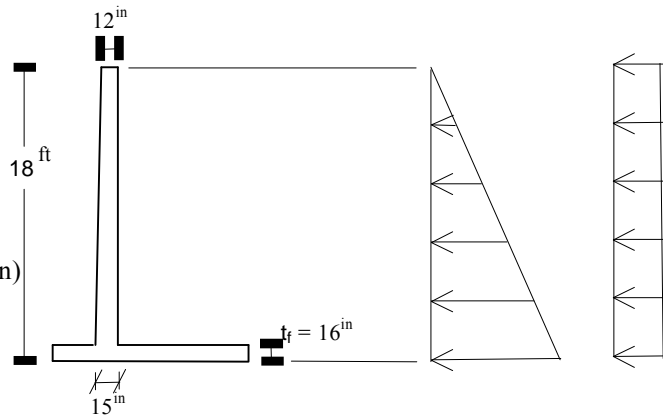
$$W_{found} = (150 pcf) \left(\frac{16}{12} ft \right) (w_{heel} + \frac{15}{12} ft + 3^{ft}) (1^{ft}) = 200 plf w_{heel} + 850$$

$$F_{sliding} = P_{fill} + P_{sur}$$

$$P_{fill} = \frac{1}{2} (0.31 \times 100 pcf) (18^{ft})^2 (1^{ft}) = 5020^{lb}$$

$$P_{sur} = (0.31 \times 400 pcf) (18^{ft}) (1^{ft}) = 2230^{lb}$$

$$F_{sliding} = 5020^{lb} + 2230^{lb} = 7250^{lb}$$



$$7250^{lb} = \frac{\left[1670 \frac{lb}{ft} w_{heel} + 2810^{lb} + 200 \frac{lb}{ft} w_{heel} + 850^{lb} \right]}{1.5} \quad (0.62)$$

$$7250^{lb} \frac{1.5}{0.62} = 3660^{lb} + 1870 \frac{lb}{ft} w_{heel}, \quad w_{heel} = 7.42^{ft},$$

use $w_{heel} = 7.5^{ft}$

5. Check Overturning.

$$M_{over} = P_{fill} \left(\frac{18^{ft}}{3} \right) + P_{sur} \left(\frac{18^{ft}}{2} \right)$$

$$M_{over} = 5.02^k (6^{ft}) + 2.23^k (9^{ft}) = 50.2^{k-ft}$$

$$M_{resist} = W_{fill} \left(\frac{7.5^{ft}}{2} + \frac{15}{12} ft + 3^{ft} \right), \text{ assume } w_{toe} = 3^{ft}$$

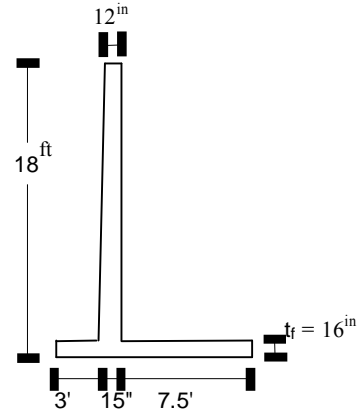
$$+ W_{stem} \left(3^{ft} + \frac{1.25^{ft}}{2} \right)$$

$$+ W_{found} \left(\frac{11.75^{ft}}{2} \right)$$

$$M_{resist} = \underbrace{(1.67^{klf} \times 7.5^{ft})(8^{ft})}_{12.53^k} + (2.81^k)(3.625^{ft}) + \underbrace{(0.20^{klf} \times 7.5^{ft} + 0.85^k)(5.875^{ft})}_{2.35^k}$$

$$M_{resist} = 124.2^{k-ft}$$

$$\frac{M_{resist}}{M_{over}} = \frac{124.2^{k-ft}}{50.2^{k-ft}} = 2.47 > 2.0 = FS_{over}, \text{ OK}$$



6. Check Bearing.

$$\sigma_v \text{ at end of toe} = \frac{W_T}{bL} + \frac{M}{\frac{bL^2}{6}}, \text{ equation is valid only if } e < \frac{L}{6}$$

$$W_T = W_{fill} + W_{stem} + W_{found}$$

$$W_T = 12.45^k + 2.81^k + 2.35^k = 17.69^k$$

$$M = M_{over} - W_{fill} \left(5.875^{ft} - \frac{7.5^{ft}}{2} \right) + W_{stem} \left(7.5^{ft} + \frac{1.25^{ft}}{2} - 5.875^{ft} \right) + W_{found} (0)$$

$$M = 50.2^{k-ft} - 12.53^k (2.125^{ft}) + 2.81^k (2.25^{ft}) = 29.9^{k-ft}$$

Check that $e < L/6$:

$$e = \frac{m}{W_T} = \frac{29.9^{k-ft}}{17.69^k} = 1.68^{ft}, \quad \frac{L}{6} = \frac{11.75^{ft}}{6} = 1.96^{ft}, \quad \therefore e < \frac{L}{6}, \text{ OK}$$

$$\sigma_v = \frac{17.69^k}{(1^{ft})(11.75^{ft})} + \frac{29.9^{k-ft}}{\frac{1}{6}(1^{ft})(11.75^{ft})^2} = 2.80^{ksf} < 5.0^{ksf} = \text{allowable bearing capacity, OK}$$

7. Heel Design.

Max. load on heel is due to the weight of heel + fill + surcharge as the wall tries to tip over.

Flexure:

$$W = W_{heel} + W_{fill} + W_{sur}$$

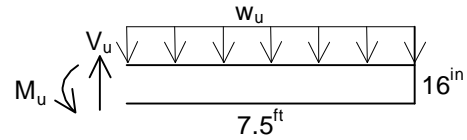
$$W = 1.2(150 pcf) \left(\frac{16}{12} ft \right) (1^{ft}) + 1.2(100 pcf) (16.67^{ft}) (1^{ft}) + 1.6(400 plf)$$

$$W = 2.88^{klf}$$

$$M_u = \frac{w_u L^2}{2} = \frac{2.88^{klf} (7.5^{ft})^2}{2} = 81.0^{k-ft}$$

$$M_u = 5.71 \frac{k}{in} d^2$$

$$81.0^{k-ft} \left(12 \frac{in}{ft} \right) = 5.71 \frac{k}{in} d^2, \quad d = 13.0^{in} \text{ for flexure}$$



Shear:

$$V_u = w_u (7.5^{ft}) = 2.88^{klf} (7.5^{ft}) = 21.6^k$$

$$\phi V_c = (0.75) 2 \sqrt{f'_c} b_w d = (0.75) 2 \sqrt{3000 \text{ psi}} (12^{in}) d$$

$$\text{set } V_u = \phi V_c, \quad 21,600^{lb} = (0.75) 2 \sqrt{3000 \text{ psi}} (12^{in}) d, \quad d = 21.9^{in} \text{ for shear, controls}$$

Shear controls the thickness of the heel.

$$t_{heel} = 21.9^{in} + 2^{in} \text{ cover} + \frac{1}{2} in = 24.4^{in} \text{ (assume \#8 bar),}$$

$$\underline{\underline{\text{use } t_{heel} = 21.5^{in}}}$$

Reinforcement in heel:

$$M_u = 47.7^{ksi} A_s d$$

$$81.0^{k-ft} \left(12 \frac{in}{ft} \right) = 47.7^{ksi} A_s (21.9^{in}), \quad A_s = 1.07 in^2$$

$$\frac{0.79 \frac{in^2}{bar}}{1.07 \frac{in^2}{ft}} \left(12 \frac{in}{ft} \right) = 8.83^{in},$$

$$\underline{\underline{\text{use \#8 @ 8"}}$$

8. Toe Design.

Earth Pressure at Tip of Toe:

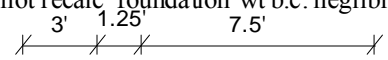
$$\sigma_v = \frac{W_u}{bL} \pm \frac{M_u}{bL^2}$$

$$W_u = 1.2(W_{fill} + W_{stem} + W_{found}) + 1.6(W_{sur})$$

$$W_u = 1.2(12.53^k + 2.81^k + 2.35^k) + 1.6(0.4^{ksf})(18^{ft})(1^{ft}) = 32.7^k, \text{ (did not recalc foundation wt b.c. negligible change)}$$

$$M_u = 1.6M_{over} - 1.2(W_{soil} \times 2.125^{ft} + W_{stem} \times 1.0^{ft})$$

$$M_u = 1.6(50.2^{k-ft}) - 1.2[12.53^k(2.125^{ft}) + 2.81^k(1^{ft})] = 45.0^{k-ft}$$

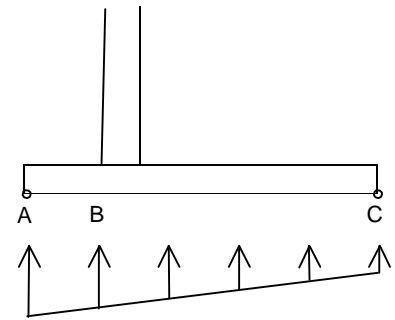


$$\sigma_v = \frac{32.7^k}{(1^{ft})(11.75^{ft})} + \frac{45.0^{k-ft}}{6(1^{ft})(11.75^{ft})^2}$$

$$\sigma_{vA} = 2.78^{ksf} + 1.96^{ksf} = 4.74^{ksf}$$

$$\sigma_{vC} = 2.78^{ksf} - 1.96^{ksf} = 0.82^{ksf}$$

$$\sigma_{vB} = 0.82^{ksf} + \frac{4.74^{ksf} - 0.82^{ksf}}{11.75^{ft}}(8.75^{ft}) = 3.74^{ksf}$$



d for flexure:

$$M_u = (3.74^{ksf})(3^{ft})(1^{ft})(\frac{3^{ft}}{2}) + \frac{1}{2}(1.00^{ksf})(3^{ft})(1^{ft})(\frac{2}{3}3^{ft}) = 19.8^{k-ft}$$

$$M_u = 5.71 \frac{k}{in} d^2$$

$$19.8^{k-ft} (12 \frac{in}{ft}) = 5.71 \frac{k}{in} d^2, \quad d = 6.5^{in} \text{ for flexure}$$

d for shear:

Assume $t_{heel} = t_{toe} = 21.5^{in}$

Critical section for shear occurs at "d" from face of stem, $d = 21.5 - 3 \text{cover} - 1/2 = 18"$

$$\sigma_{v_{critical\ section}} = 0.82^{ksf} + \frac{4.74^{ksf} - 0.82^{ksf}}{11.75^{ft}} (8.75^{ft} + \frac{18}{12}^{ft}) = 4.24^{ksf}$$

$$V_u = \frac{1}{2}(4.74^{ksf} + 4.24^{ksf})(3^{ft} - \frac{18}{12}^{ft})(1^{ft}) = 6.74^k$$

$$\phi V_c = (.75)2\sqrt{3000\text{psi}}(12^{in})(18^{in}) = 17,750^{lb} > V_u, \text{ OK, } d \text{ for flexure controls}$$

Reinforcement in toe:

$$M_u = 47.7^{ksi} A_s d$$

$$19.8^{k-ft} \left(12 \frac{in}{ft}\right) = 47.7^{ksi} A_s (18^{in}), \quad A_s = 0.28 in^2$$

$$\frac{0.79 \frac{in^2}{bar}}{0.28 \frac{in^2}{ft}} \left(12 \frac{in}{ft}\right) = 33^{in}, \text{ try smaller bars, say \#4}$$

$$\frac{0.20 \frac{in^2}{bar}}{0.28 \frac{in^2}{ft}} \left(12 \frac{in}{ft}\right) = 8.6^{in} \quad \text{use \#4@8"}$$