Chapter 4
System Loads

4-1. General

The loads governing the design of a sheet pile wall arise primarily from the soil and water surrounding the wall and from other influences such as surface surcharges and external loads applied directly to the piling. Current methodologies for evaluating these loads are discussed in the following paragraphs.

4-2. Earth Pressures

Earth pressures reflect the state of stress in the soil mass. The concept of an earth pressure coefficient, K, is often used to describe this state of stress. The earth pressure coefficient is defined as the ratio of horizontal stresses to the vertical stresses at any depth below the soil surface:

$$K = \frac{\sigma_h}{\sigma_v} \tag{4-1}$$

Earth pressures for any given soil-structure system may vary from an initial state of stress referred to as at-rest, K_o, to minimum limit state referred to as active, K_A, or to a maximum limit state referred to as passive, K_P. The magnitude of the earth pressure exerted on the wall depends, among other effects, on the physical and strength properties of the soil, the interaction at the soil-structure interface, the ground-water conditions, and the deformations of the soil-structure system. These limit states are determined by the shear strength of the soil:

$$\tau_f = c + \sigma_n \tan\phi \tag{4-2}$$

where

τ_f and σ_n = shear and normal stresses on a failure plane

c and ϕ = shear strength parameters of the soil, cohesion, and angle of internal friction, respectively (Figure 4-1)

a. At-rest pressures. At-rest pressure refers to a state of stress where there is no lateral movement or strain in the soil mass. In this case, the lateral earth pressures are the pressures that existed in the ground prior to installation of a wall. This state of stress is shown in Figure 4-2 as circle O on a Mohr diagram.

b. Active pressures. Active soil pressure is the minimum possible value of horizontal earth pressure at any depth. This pressure develops when the walls move or rotate away from the soil allowing the soil to expand horizontally in the direction of wall movement. The state of stress resulting in active pressures is shown in Figure 4-2 as circle A.

c. Passive pressures. Passive (soil) pressure is the maximum possible horizontal pressure that can be developed at any depth from a wall moving or rotating toward the soil and tending to compress the soil horizontally. The state of stress resulting in passive pressures is shown in Figure 4-2 as circle *P*.

d. Wall movements. The amount of movement required to develop minimum active or maximum passive earth pressures depends on the stiffness of the soil and the height of the wall. For stiff soils like dense sands or heavily overconsolidated clays, the required movement is relatively small. An example is shown in Figure 4-3 which indicates that a movement of a wall away from the fill by 0.3 percent of the wall height is sufficient to develop minimum pressure, while a movement of 2.0 percent of the wall height toward the fill is sufficient to develop the maximum pressure. For all sands of medium or higher density, it can be assumed that the movement required to reach the minimum active earth pressure is no more than about 0.4 percent of the wall height, or about 1 inch of movement of a 20-foot-high wall. The movement required to increase the earth pressure to its maximum passive value is about 10 times that required for the minimum, about 4.0 percent of the wall height or about 10 inches of movement for a 20-foot-high wall. For loose sands, the movement required to reach the minimum active or the maximum passive is somewhat larger. The classical design procedures described in this chapter assume that the sheet pile walls have sufficient flexibility to produce the limit state, active or passive earth pressures. A method to account for intermediate to extreme values of earth pressure by soil-structure interaction analysis is presented in Chapter 7.

e. Wall friction and adhesion. In addition to the horizontal motion, relative vertical motion along the wall soil interface may result in vertical shearing

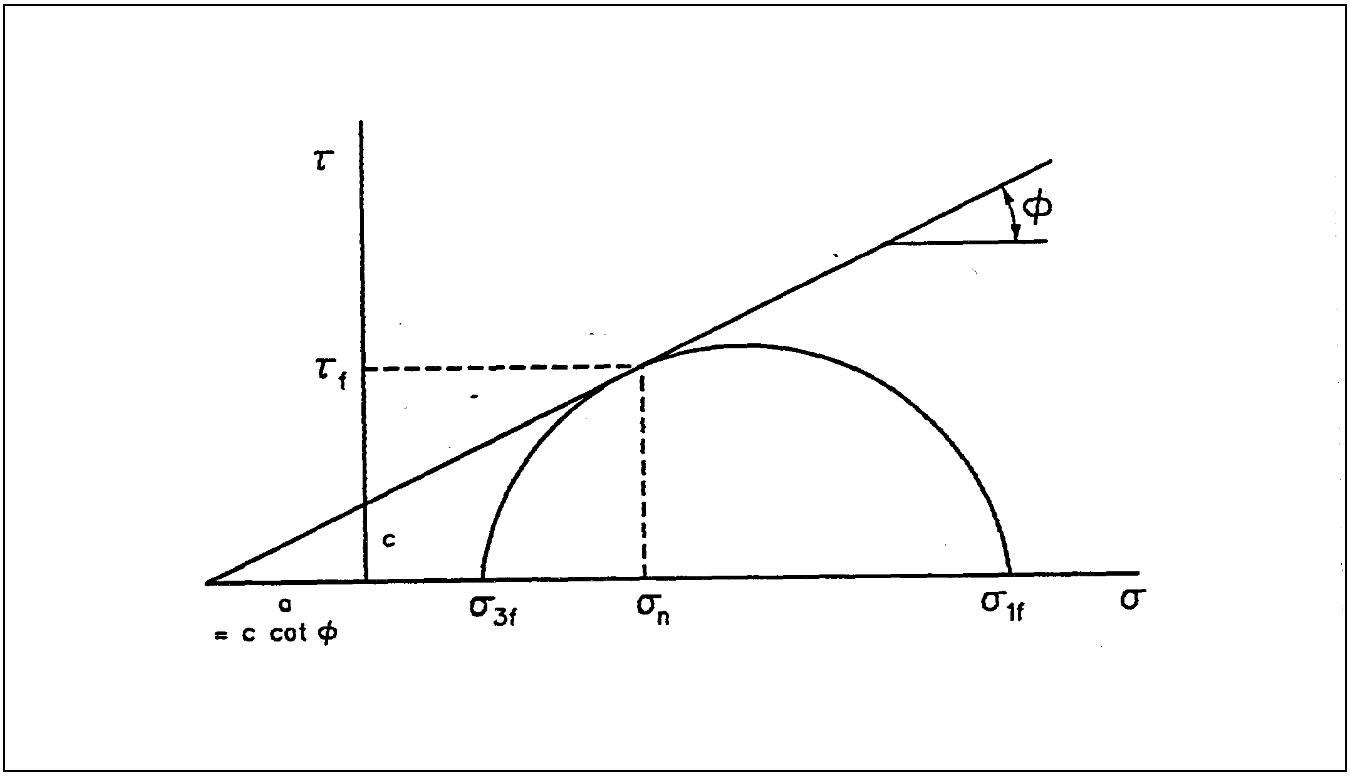


Figure 4-1. Shear strength parameters

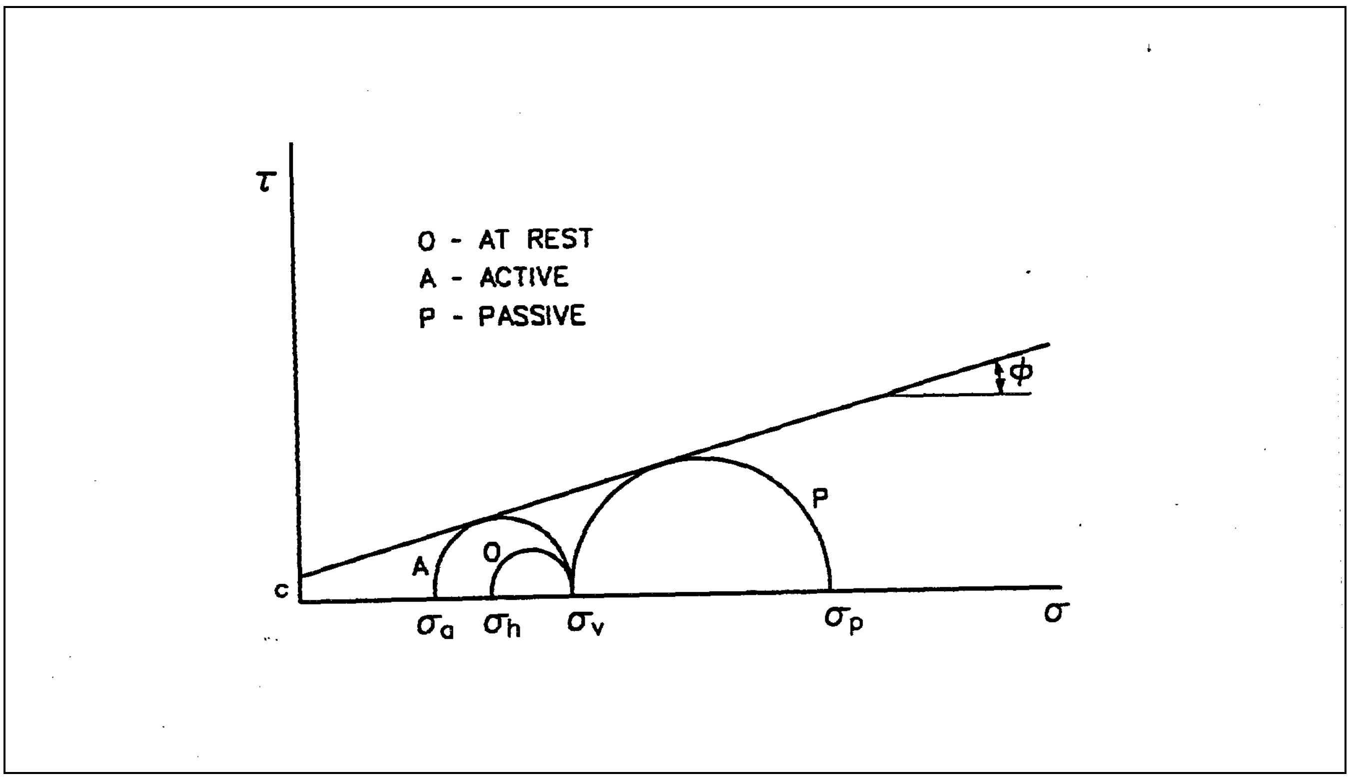


Figure 4-2. Definition of active and passive earth pressures

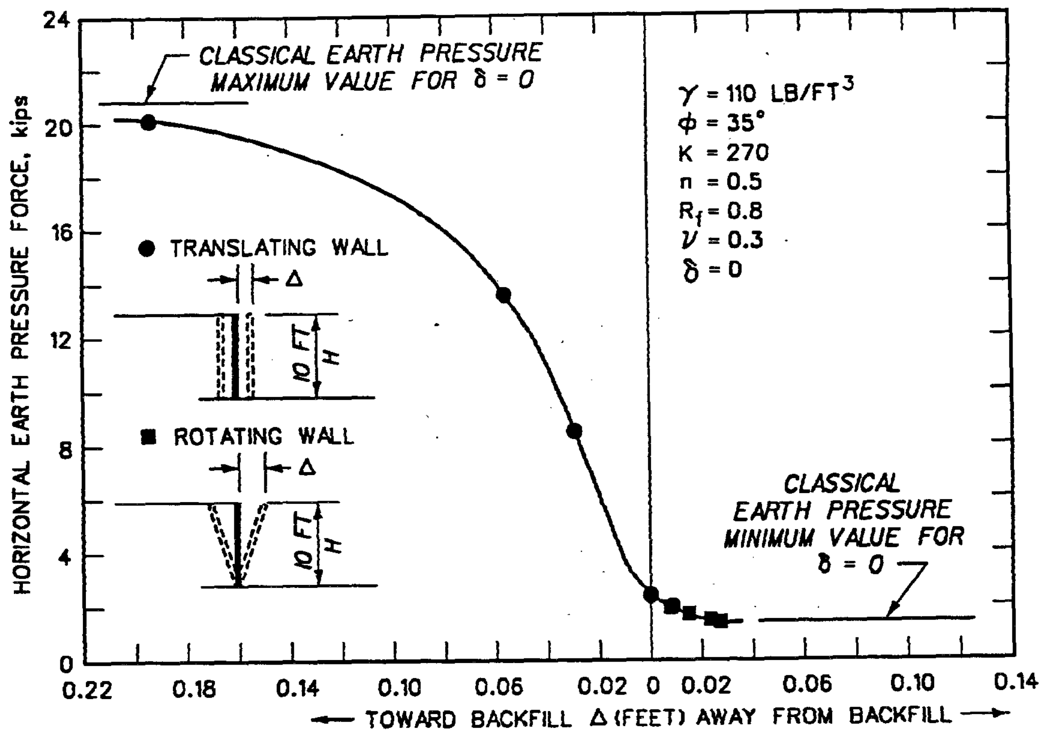


Figure 4-3. Variations of earth pressure force with wall movement calculated by finite element analyses (after Clough and Duncan 1971)

stresses due to wall/soil friction in the case of granular soils or in wall/soil adhesion for cohesive soils. This will have an effect on the magnitude of the minimum and maximum horizontal earth pressures. For the minimum or active limit state, wall friction or adhesion will slightly decrease the horizontal earth pressure. For the maximum or passive limit state, wall friction or adhesion may significantly increase the horizontal earth pressure depending on its magnitude.

4-3. Earth Pressure Calculations

Several earth pressures theories are available for estimating the minimum (active) and maximum (passive) lateral earth pressures that can develop in a soil mass surrounding a wall. A detailed discussion of various theories is presented by Mosher and Oner (1989). The Coulomb theory for lateral earth pressure will be used for the design of sheet pile walls.

a. Coulomb Theory. The evaluation of the earth pressures is based on the assumption that a failure plane develops in the soil mass, and along that failure the shear and normal forces are related by the shear strength expression (Equation 4-2). This makes the problem statically determinate. Free-body diagrams of a wedge of homogeneous soil bounded by the soil surface, the sheet pile wall, and a failure plane are shown in Figure 4-4. Equilibrium analysis of the forces shown in Figure 4-4 allows the active force, P_a, or passive force, P_p, to be expressed in terms of the geometry and shear strength:

γ = unit weight of the homogeneous soil

ϕ = angle of internal soil friction

c = cohesive strength of the soil

δ = angle of wall friction

θ = angle between the wall and the failure plane

z = depth below the ground surface

β = slope of the soil surface

For the limit state (minimum and maximum), active or passive, the angle i, critical angle at failure, is obtained from $dP/d\theta = 0$. Finally, the soil pressure at depth z is obtained from $p = dP/dz$. These operations result in

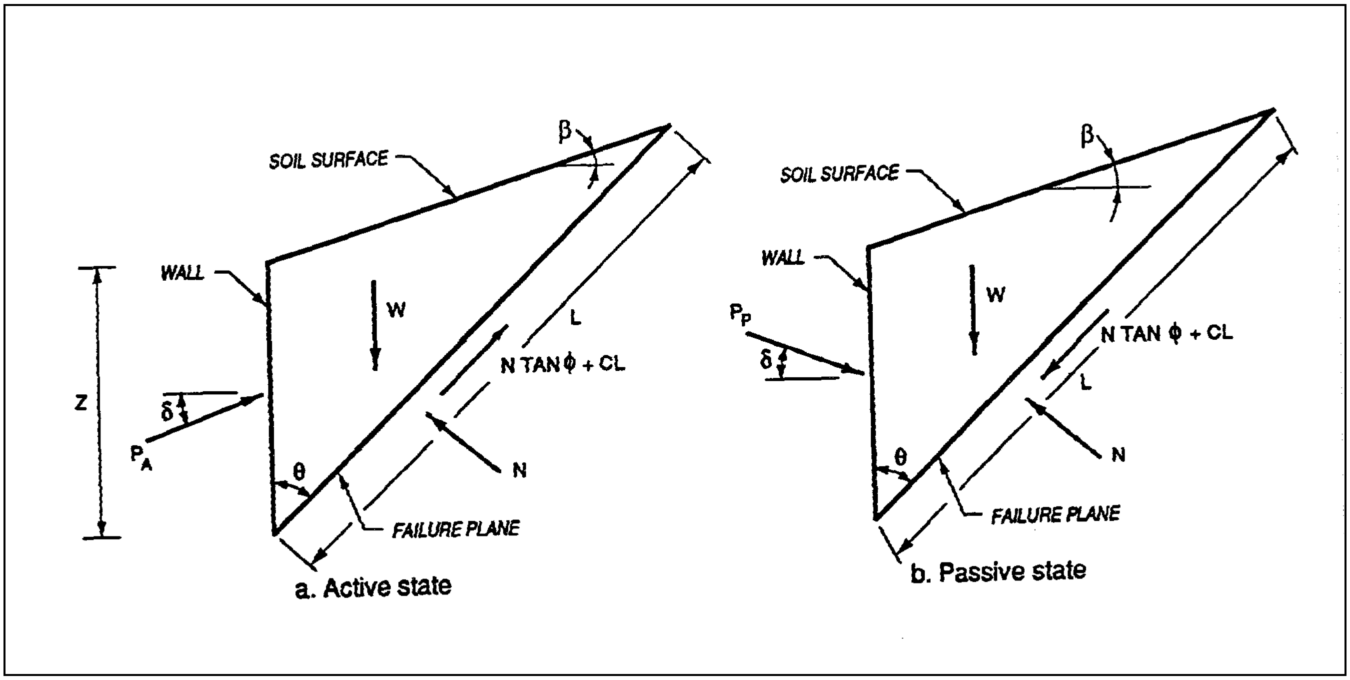


Figure 4-4. Soil wedges for Coulomb earth pressure theory

values of active pressure given by

$$p_a = \gamma z \; K_A \; - \; 2c \; \sqrt{K_A} \tag{4-3}$$

and passive pressure given by

$$p_p = \gamma z \; K_P \; + \; 2c \; \sqrt{K_P} \tag{4-4}$$

where K_A and K_P are coefficients of active and passive earth pressures given by

$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2\theta\cos(\theta + \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\cos(\delta + \phi)\cos(\beta - \phi)}}\right]^2} \tag{4-5}$$

and

$$K_P = \frac{\cos^2(\phi - \theta)}{\cos^2\theta\cos(\theta + \delta)\left[1 - \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi + \beta)}{\cos(\delta - \phi)\cos(\beta - \phi)}}\right]^2} \tag{4-6}$$

b. Coefficient method for soil pressures. The Coulomb theory outlined in paragraph 4.3a, although originally developed for homogeneous soils, is assumed to apply to layered soil systems composed of horizontal, homogeneous layers. The product γz in Equations 4-3 and 4-4 is the geostatic soil pressure at depth z in the homogeneous system. In a layered system this term is replaced by the effective vertical soil pressure p_v at depth z including the effects of submergence and seepage on the soil unit weight. The active and passive earth pressures at any point are obtained from

$$p_a = p_v \; K_A \; - \; 2c\sqrt{K_A} \tag{4-7}$$

and

$$p_p = p_v \; K_A \; + \; 2c\sqrt{K_A} \tag{4-8}$$

where K_A and K_P are the coefficients of active and passive earth pressure from equations 4-5 and 4-6 with ϕ and c being the "effective" (see subsequent discussion of soil factor of safety) strength properties and δ is the angle of wall friction at the point of interest. This procedure can result in large discontinuities in calculated pressure distributions at soil layer boundaries.

c. Wedge methods for soil pressures. The coefficient method does not account for the effects of sloping ground surface, sloping soil layer boundaries, or the presence of wall/soil adhesion. When any these effects are present, the soil pressures are calculated by a numerical procedure, a wedge method, based on the fundamental assumptions of the Coulomb theory. Practical evaluation of soil pressures by the wedge method requires a computer program. (CWALSHT User's Guide (USAEWES 1990) or CWALSSI User's Guide (Dawkins 1992.)

4-4. Surcharge Loads

Loads due to stockpiled material, machinery, roadways, and other influences resting on the soil surface in the vicinity of the wall increase the lateral pressures on the wall. When a wedge method is used for calculating the earth pressures, the resultant of the surcharge acting on the top surface of the failure wedge is included in the equilibrium of the wedge. If the soil system admits to application of the coefficient method, the effects of surcharges, other than a uniform surcharge, are evaluated from the theory of elasticity solutions presented in the following paragraphs.

a. Uniform surcharge. A uniform surcharge is assumed to be applied at all points on the soil surface. The effect of the uniform surcharge is to increase the effective vertical soil pressure, p_v in Equations 4-7 and 4-8, by an amount equal to the magnitude of the surcharge.

b. Strips loads. A strip load is continuous parallel to the longitudinal axis of the wall but is of finite extent perpendicular to the wall as illustrated in Figure 4-5. The additional pressure on the wall is given by the equations in Figure 4-5. Any negative pressures calculated for strips loads are to be ignored.

c. Line loads. A continuous load parallel to the wall but of narrow dimension perpendicular to the wall may be treated as a line load as shown in Figure 4-6. The lateral pressure on the wall is given by the equation in Figure 4-6.

d. Ramp load. A ramp load, Figure 4-7, increases linearly from zero to a maximum which subsequently remains uniform away from the wall. The ramp load is assumed to be continuous parallel to the wall. The equation for lateral pressure is given by the equation in Figure 4-4.

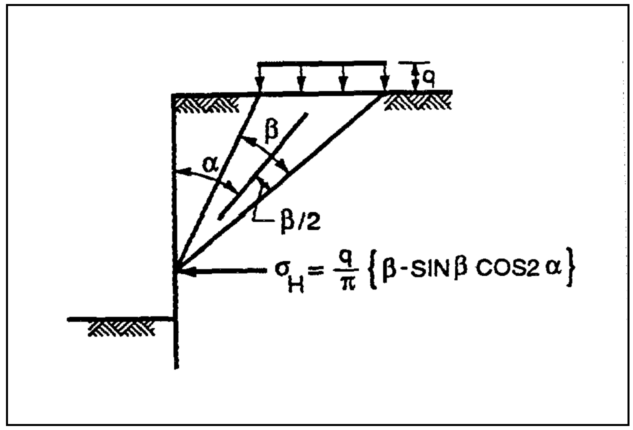


Figure 4-5. Strip load

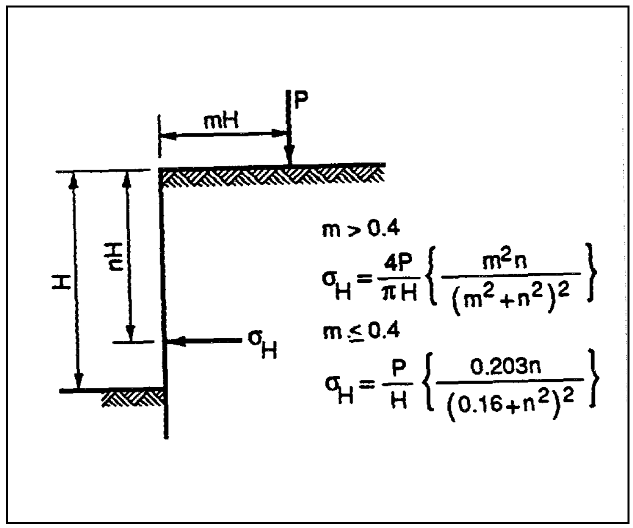


Figure 4-6. Line load

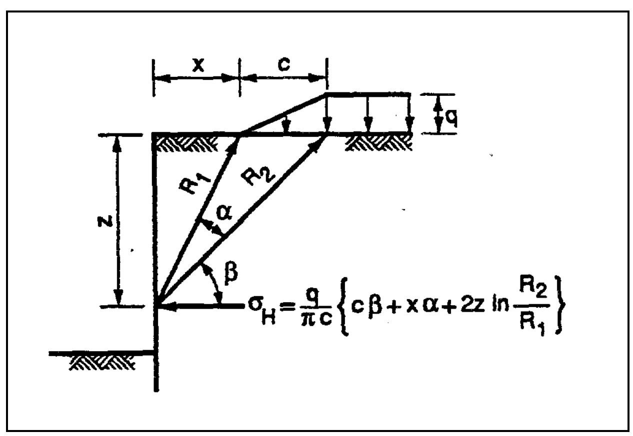


Figure 4-7. Ramp load

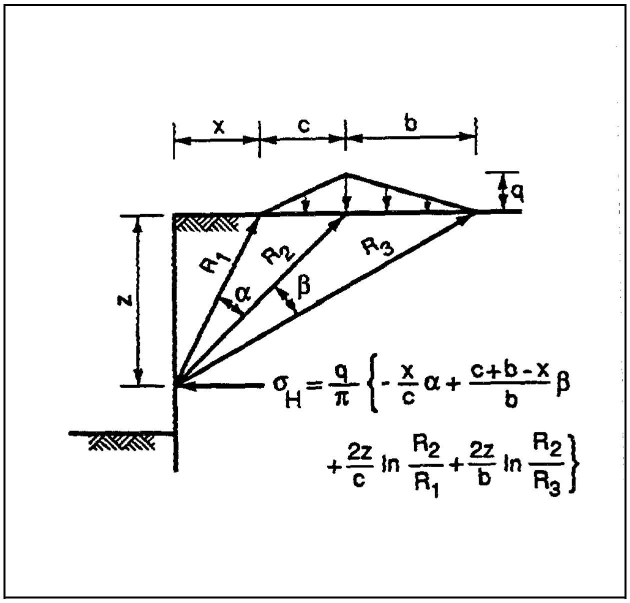


Figure 4-8. Triangular load

e. Triangular loads. A triangular load varies perpendicular to the wall as shown in Figure 4-8 and is assumed to be continuous parallel to the wall. The equation for lateral pressure is given in Figure 4-8.

f. Area loads. A surcharge distributed over a limited area, both parallel and perpendicular to the wall, should be treated as an area load. The lateral pressures induced by area loads may be calculated using Newmark's Influence Charts (Newmark 1942). The lateral pressures due to area loads vary with depth below the ground surface and with horizontal distance parallel to the wall. Because the design procedures discussed subsequently are based on a typical unit slice of the wall/soil system, it may be necessary to consider several slices in the vicinity of the area load.

g. Point loads. A surcharge load distributed over a small area may be treated as a point load. the equations for evaluating lateral pressures are given in Figure 4-9. Because the pressures vary horizontally parallel to the wall; it may be necessary to consider several unit slices of the wall/soil system for design.

4-5. Water Loads

a. Hydrostatic pressure. A difference in water level on either side of the wall creates an unbalanced hydrostatic pressure. Water pressures are calculated by multiplying the water depth by its specific weight. If a nonflow hydrostatic condition is assumed, i.e. seepage effects neglected, the unbalanced hydrostatic pressure is assumed to act along the entire depth of embedment. Water pressure must be added to the effective soil pressures to obtain total pressures.

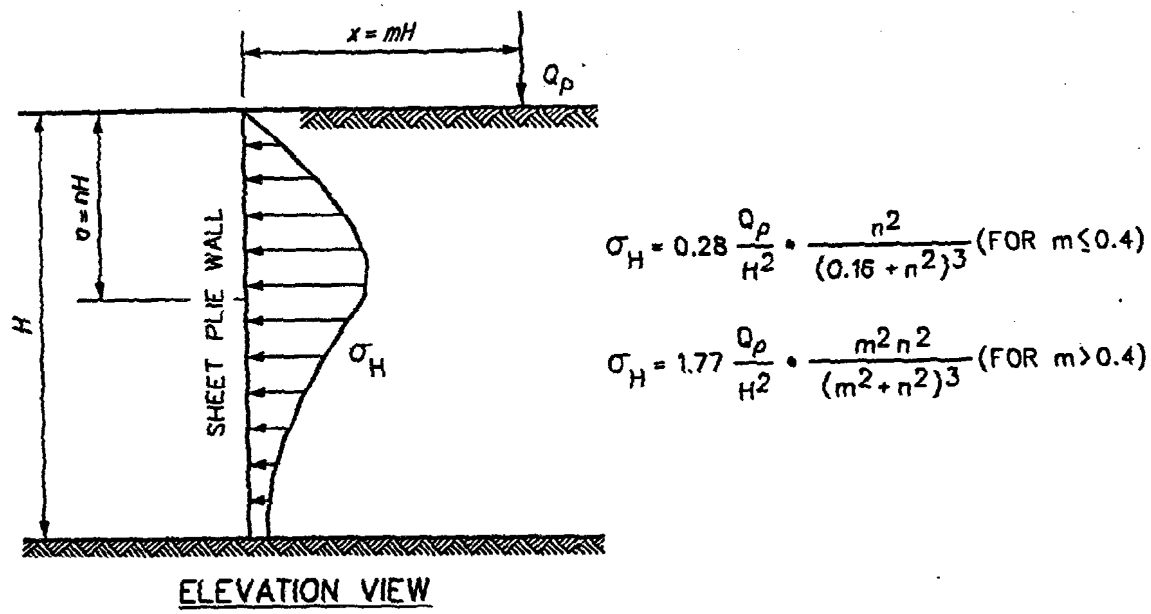
b. Seepage effects. Where seepage occurs, the differential water pressure is dissipated by vertical flow beneath the sheet pile wall. This distribution of the unbalanced water pressure can be obtained from a seepage analysis. The analysis should consider the permeability of the surrounding soils as well as the effectiveness of any drains if present. Techniques of seepage analysis applicable to sheet pile wall design include flow nets, line of creep method, and method of fragments. These simplified techniques may or may not yield conservative results. Therefore, it is the designer's responsibility to decide whether the final design should be based on a more rigorous analysis, such as the finite element method. Upward seepage in front of the sheet pile wall tends to reduce the effective weight of the soil, thus reducing its ability to offer lateral support. In previous material the effects of upward seepage can cause piping of material away from the wall or, in extreme cases, cause the soil to liquefy. Lengthening the sheet pile, thus increasing the seepage path, is one effective method of accommodating seepage. For sheet pile walls that retain backfill, a drainage collector system is recommended. Some methods of seepage analysis are discussed in EM 1110-2-1901.

c. Wave action. The lateral forces produced by wave action are dependent on many factors, such as length, height, breaking point, frequency and depth at structure. Wave forces for a range of possible water levels should be determined in accordance with the U.S. Army Coastal Engineering Research Center Shore Protection Manual (USAEWES 1984).

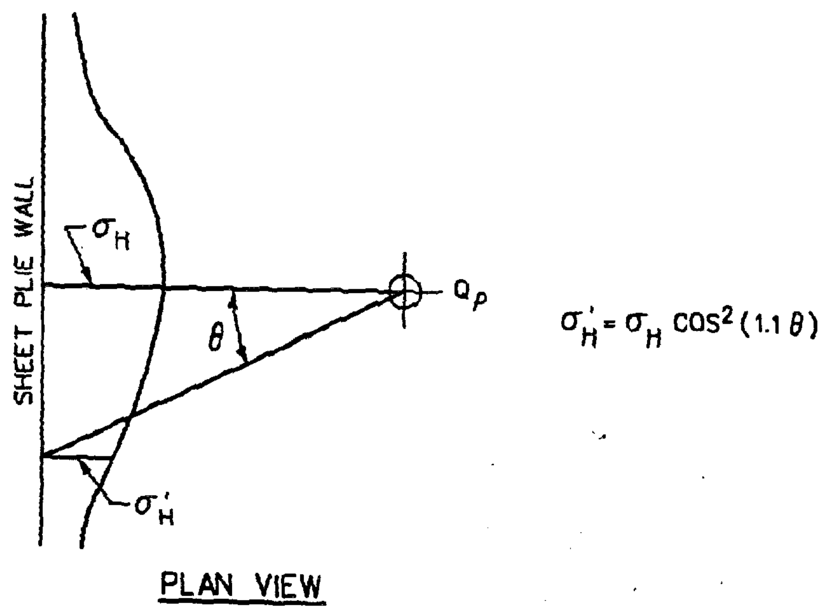
4-6. Additional Applied Loads

Sheet Pile walls are widely used in many applications and can be subjected to a number of additional loads, other than lateral pressure exerted by soil and water.

a. Boat impact. Although it becomes impractical to design a sheet pile wall for impact by large vessels, waterfront structures can be struck by loose barges or smaller vessels propelled by winds or currents. Construction of a submerged berm that would ground a vessel will greatly reduce this possibility of impact. When the sheet pile structure is subject to docking impact, a fender system should be provided to absorb



a. Vertical variation of pressures



b. Horizontal variation of pressures

Figure 4-9. Point load (after Terzaghi 1954) and spread the reaction. The designer should weigh the risk of impact and resulting damage as it applies to his situation. If conditions require the inclusion of either of these boat impact forces in the design, they should be evaluated based on the energy to be absorbed by the wall. The magnitude and location of the force transmitted to the wall will depend on the vessel's mass, approach velocity, and approach angle. Military Handbook 1025/1 (Department of the Navy 1987) provides excellent guidance in this area.

b. Mooring pulls. Lateral loads applied by a moored ship are dependent on the shape and orientation of the vessel, the wind pressure, and currents applied. Due to the use of strong synthetic lines, large forces can be developed. Therefore, it is recommended that mooring devices be designed independent of the sheet pile wall.

c. Ice forces. Ice can affect marine-type structures in many ways. Typically, lateral pressures are caused by impact of large floating ice masses or by expansion upon freezing. Expansive lateral pressures induced by water freezing in the backfill can be avoided by backfilling with a clean free-draining sand or gravel or installation of a drainage collector system. EM 1110-2-1612 should be references when the design is to include ice forces.

d. Wind forces. When sheet pile walls are constructed in exposed areas, wind forces should be considered during construction and throughout the life of the structure. For sheet pile walls with up to 20 feet of exposure and subjected to hurricanes or cyclones with basic winds speeds of up to 100 mph, a 50-pound per square foot (psf) design load is adequate. Under normal circumstances, for the same height of wall exposure, a 30-psf design load should be sufficient. For more sever conditions, wind load should be computed in accordance with American National Standards Institute (ANSI) A58.1 (ANSI 1982).

e. Earthquake forces. Earthquake forces should be considered in zones of seismic activity. The earth pressures should be determined in accordance with procedures outlined in EM 1110-2-2502 and presented in detail in the Ebeling and Morrison report on seismic design of waterfront retaining structures (Ebeling and Morrison 1992). In the worst case, the supporting soil may liquify allowing the unsupported wall to fail. This possibility should be evaluated and addressed in the design documentation. If accepting the risk and consequences of a liquefaction failure is unacceptable, consideration should be given to replacing or improving the liquefiable material or better yet, relocating the wall.