



This Design Guide has been developed to document the equations, assumptions, and references associated with a simplified procedure IDOT uses for estimating primary settlement of cohesive soil layers. This procedure has been programmed into an Excel spreadsheet titled "Cohesive Soil Settlement Estimate" available on the IDOT website. Note that elastic settlement of granular soils which occurs almost immediately upon loading and secondary settlement of organic layers which occurs over a much longer period of time is not included in this estimate.

The estimated settlement at the center of a given cohesive soil layer due to the placement of an earth surcharge is estimated using the equations below. Total estimated primary settlement is the summation of the settlement calculated for each individual cohesive soil layer below the surcharge.

$$\Delta_i = \frac{CF C_c H}{(1 + e_o)} \text{Log} \left(\frac{P_o' + \Delta P'}{P_o'} \right)$$

Where:

Δ_i = estimated primary settlement of a given cohesive soil layer

H = thickness of cohesive soil layer

e_o = initial void ratio

P_o' = existing effective overburden pressure at the center of the cohesive soil layer

$\Delta P'$ = increase in overburden pressure at the center of the cohesive soil layer due to new earth surcharge loading

C_c = compression index (dimensionless)

= 0.009 (LL - 10)

LL = liquid limit (%)

CF = unconfined compressive strength correction factor

The above equation for C_c is an approximation provided by Terzaghi and Peck (*Soil Mechanics in Engineering*) which says when the natural moisture content (w_n) of the soil is known, the LL may be assumed equal to the w_n .

Initial void ratio, e_o , may be estimated using the following relationship:

$$e_o = \frac{w_n G_s}{S}$$

Where:

w_n = moisture content (%)

G_s = specific gravity (assume 2.7 for IL soils)

S = degree of saturation (% - assume saturated conditions, i.e., 100%)

The simplified procedure for estimating settlement typically results in settlements that are believed to be reasonably accurate for very soft and soft clays, (i.e., $q_u \leq 0.5$ tsf). Past editions of the American Railway Engineering Association (AREA, now commonly referred to as American Railway Engineering and Maintenance-of-Way Association or AREMA) specifications indicated that for medium and stiff clays (i.e., $0.5 \text{ tsf} < q_u < 2.0$ tsf), actual settlements are likely to range between one-fourth and one-tenth of the computed values. This is consistent with recommendations made in the text book *Foundation Engineering* by Peck, Hanson and Thornburn which indicated that using this method with preloaded clays would result in over estimation of settlement by between one-fourth and one-tenth. As such, the following unconfined compressive strength correction factor, CF, was developed and is used by IDOT to reduce settlements estimated for soils with substantial unconfined compressive strength, indicating reduced initial void ratio due to preloading:

$$\begin{aligned} \text{CF} &= -1.5 q_u + 1.0 && \text{for } q_u \leq 0.3 \text{ tsf} \\ &= \frac{0.2}{(q_u)^{0.85}} && \text{for } 0.3 \text{ tsf} < q_u \leq 2.0 \text{ tsf} \\ &= 0.1 && \text{for } q_u > 2.0 \text{ tsf} \end{aligned}$$

The existing overburden pressure at the center of a compressible layer, P_o' , and the increase in pressure caused by the placement of new embankment loading, $\Delta P'$, must both be determined to accurately estimate settlement. To determine the existing overburden pressure, the product of the effective unit weight and layer thickness of each layer from the existing ground surface or bottom of existing embankment to the center of the layer in question, must be totaled. When an existing embankment exists, the existing vertical pressure from it must be computed using Boussinesq elastic-isotropic theories and added to that determined above (below the existing embankment). New embankment and existing embankment widening or grade raises result in an increase in load, $\Delta P'$ which is equal to the new embankment minus the existing embankment, if any, using Boussinesq equations. These equations are provided below which were obtained from the text book *Elastic Solutions for Soil and Rock Mechanics* by H.G. Poulos and E.H. Davis (1974).

We typically see two types of embankment surcharges. The first is a continuous embankment, having a cross section defined by its width at the top, side slope angle, unit weight and height above the assumed level ground. The second is a 2:1 bridge cone which requires the same input but results in less vertical pressure due to its being continuous in only one direction. In very rare occasions, a rectangular surcharge could exist above a level ground surface and would require the length of the rectangle to also be provided. Box culverts are not considered rectangular surcharges since the adjacent ground applies pressures larger than the box itself. The following equations are used to determine the vertical pressure from the embankment portion of vertical stress above the level ground surface.

Continuous Embankments: For this configuration, it is assumed that the embankment is symmetric about its centerline and the maximum settlement will occur at its center. In the equation below, “p” is the maximum pressure at the base of the embankment which is equal to the embankment height multiplied by its unit weight. β and α below are expressed in radians not degrees.

$$\sigma_z = \frac{2p}{\pi} \left[\beta + \frac{b \alpha}{2a} \right]$$

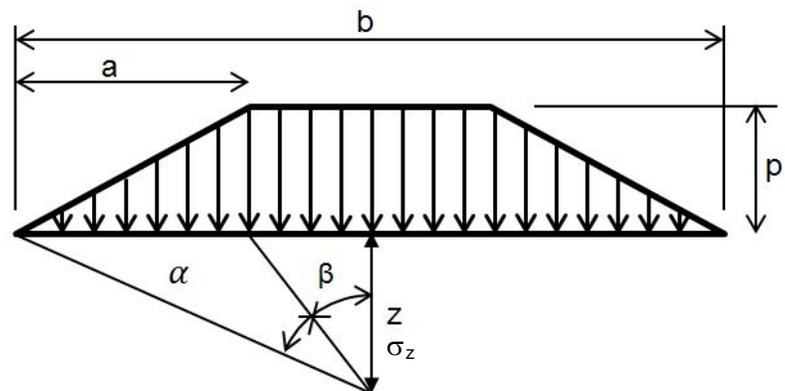


Figure 2. Loading Diagram for Continuous Embankments.

2:1 Bridge Cones: Bridge cones almost always have a 2:1 end slope and transition to side slopes which are often flatter than 2:1. The pressure increase at depth due to this surcharge is calculated at the center of the embankment, directly below the point where the 2:1 end slope meets the continuous embankment. To model this surcharge, the continuous embankment portion of this surcharge is obtained as shown above for the continuous embankment except it is divided by two since it is not continuous in both directions. In addition to this, the end slope triangular surcharge loading portion must be included and obtained using the following equation:

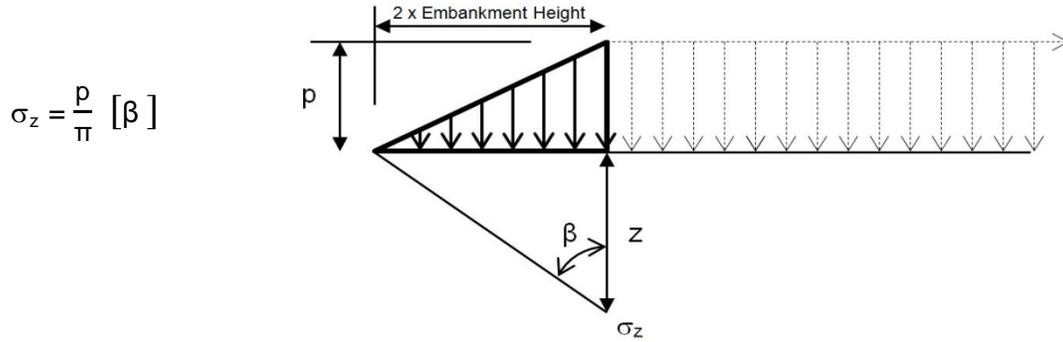


Figure 4. Loading Diagram for 2:1 Slope Portion of Bridge Cone Surcharge.

Rectangular Surcharge: The pressure at any given depth z below the center of a rectangular surcharge can be calculated by dividing the rectangular area into 4 equal quadrants as shown. The following equation will provide the pressure below the interior corner of the four quadrants.

$$\sigma_z = \frac{2p}{\pi} \left[\tan^{-1} \frac{lb}{zR_3} + \frac{lbz}{R_3} \left(\frac{1}{R_1^2} + \frac{1}{R_2^2} \right) \right]$$

$$R_1 = \sqrt{(l^2 + z^2)}$$

$$R_2 = \sqrt{(b^2 + z^2)}$$

$$R_3 = \sqrt{(l^2 + b^2 + z^2)}$$

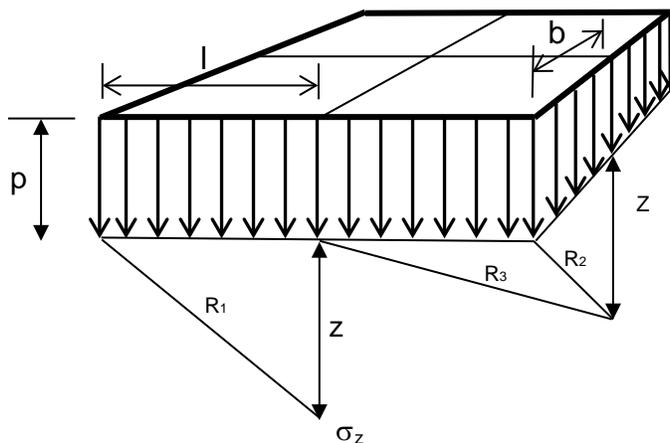
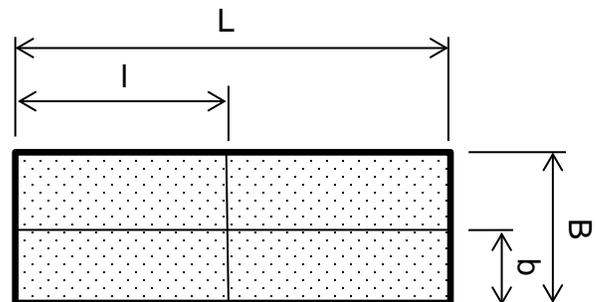


Figure 3. Loading Diagram for Rectangular Surcharge.