Wind Loading: (ASCE 7-16 Sec. 29) — Wind Loading Governs

Risk Category I — *ASCE 7-16 Table 1.5-1*

Exposure Category B — *ASCE 7-16 Sec. 26.7.3*

$V := 88 \frac{mi}{hr}$ — *ASCE 7-16 Fig. 26.5-1A*

$k_d := 0.85$ — *ASCE 7-16 Table 26.6-1*

$k_{zt} := 1$ — *ASCE 7-16 Sec. 26.8*

$k_e := 1$ — *ASCE 7-16 Sec. 26.9*

$G := 0.85$ — *ASCE 7-16 Sec. 26.11*

$k_z := 0.57$ — *ASCE 7-16 Table 26.10-1*

$q_z := 0.00256 \cdot k_z \cdot k_{zt} \cdot k_d \cdot k_e \cdot V^2 \cdot \left(\frac{lb \cdot hr^2}{ft^2 \cdot mi^2}\right) = 9.605 \frac{lb}{ft^2}$ — *ASCE 7-16 Eq. 26.10-1*

Full Height of Sign:

$B := 2\ ft + \left(\frac{8\ in}{12 \frac{in}{ft}}\right) = 2.667\ ft$ — Horizontal width of sign

$s := 9\ ft + \left(\frac{3\ in}{12 \frac{in}{ft}}\right) = 9.25\ ft$ — Max height of sign

$\frac{B}{s} = 0.288$ — Aspect Ratio

$C_f := \frac{(1.65 - 1.55) \cdot \left(\frac{B}{s} - 0.5\right)}{(0.2 - 0.5)} + 1.55 = 1.621$ — *ASCE 7-16 Fig. 29.3-1*

$A_s := 8\ ft^2$ — Area of sign normal to wind

$F_{fh} := q_z \cdot G \cdot C_f \cdot A_s = 105.846\ lb$ — *ASCE 7-16 Eq. 29.3-1*

Assuming Sign Clearance:

$B := 2\ ft + \left(\frac{8\ in}{12\ \frac{in}{ft}}\right) = 2.667\ ft$ — Horizontal width of sign

$s := 3\ ft$ — Max height of sign

$\frac{B}{s} = 0.889$ — Aspect Ratio

$h := 7.5\ ft$ — Average Height

$\frac{s}{h} = 0.4$ — Clearance Ratio

$C_f := 1.78$ — *ASCE 7-16 Fig. 29.3-1*

$A_s := 8\ ft^2$ — Area of sign normal to wind

$F_{sc} := q_z \cdot G \cdot C_f \cdot A_s = 116.259\ lb$ — *ASCE 7-16 Eq. 29.3-1*

$F := \max\left(F_{fh}, F_{sc}\right) = 116.259\ lb$

Footing Embedment: (CBC Sec. 1807.3)

$b := 1\ ft + \left(\frac{7\ in}{12\ \frac{in}{ft}}\right) = 1.583\ ft$ — Diameter of footing

$h := 5.59\ ft + (0.05 \cdot s) = 5.74\ ft$ — Height of wind load application
ASCE 7-16 Fig. 29.3-1 Note 3

$P := F = 116.259\ lb$ — Wind loading

$$S_1 := 100\ \frac{lb}{ft^2} \cdot 2 = 200\ \frac{lb}{ft^2}$$

Allowable lateral soil-bearing pressure
CBC Table 1806.2 and CBC 1806.3.4

S1 can be increase per foot of embedment of depth.

S1 value based on a depth of one third of embement.

Assume depth: $D := 2.8\ ft$

$$S_1 := S_1 \cdot \frac{D}{3} = 186.667\ \frac{lb}{ft^2}$$

$$A := \frac{2.34 \cdot P}{S_1 \cdot b} = 0.92\ ft$$

$$d := 0.5 \cdot A \cdot \left(1 + \sqrt{1 + \left(\frac{4.36 \cdot h}{A}\right)}\right) = 2.904\ ft$$

Req'd depth of footing
CBC Eq. 18-1

<u>Use 1'-7" DIA x 3'-0" Deep</u>
<u>With 4 #5 Vert and #4 Ties at 6" OC</u>

Sample Bikeways
Concrete Seatwall
Channel Wall Impact Calculations

Concrete Seatwall:
15'-6" Channel Wall, 11'-9" Distance

$D := 11.75\ ft$ — Minimum distance between concrete seatwall & channel wall

$H_c := 15.5\ ft$ — Channel Wall Height

$h := H_c - D = 3.75\ ft$ — Load from seatwall wall applied height

$w_c := 150\ \dfrac{lb}{ft^3}$ — Unit weight of concrete

$\gamma_s := 120\ \dfrac{lb}{ft^3}$ — Unit weight of soil

$H_{cs} := 18\ in$ — Height of concrete seatwall

$W_{cs} := 1.5\ ft$ — Width of concrete seatwall

$H_{cs\_fnd} := 8\ in$ — Height of concrete seatwall foundation

$W_{cs\_fnd} := 2\ ft$ — Width of concrete seatwall foundation

$b := 1\ ft$ — Unit length

$V_{cs} := H_{cs} \cdot W_{cs} \cdot b = 2.25\ ft^3$ — Volume of concrete seatwall

$V_{cs\_fnd} := H_{cs\_fnd} \cdot W_{cs\_fnd} \cdot b = 1.333\ ft^3$ — Volume of concrete seatwall foundation

$W_c := V_{cs\_fnd} \cdot (w_c - \gamma_s) + V_{cs} \cdot w_c = 377.5\ lb$ — Additional weight from concrete per foot of length

Concrete seatwall foundation is replacing equal volume of soil

$P := \dfrac{W_c}{(W_{cs\_fnd} \cdot b)} = 188.75\ \dfrac{lb}{ft^2}$ — Additional bearing pressure from concrete seatwall

$R := 0.33 \cdot P = 62.288\ \dfrac{lb}{ft^2}$ — Lateral pressure on channel wall from additional weight of concrete seatwall

$$M_{max\_r} := \frac{R \cdot h^2}{2} \cdot b \cdot \left(\frac{1\ kip}{1000\ lb}\right) = 0.438\ kip \cdot ft$$

Max moment at base of channel wall from additional weight of concrete seatwall

Existing Demand on Channel Wall:

$$EFP_{active} := 37\ psf$$

Active earth pressure on channel wall

$$h_{surcharge} := 2\ ft$$

Surcharge height acting on channel wall

$$P_{active} := EFP_{active} \cdot H_c^{\ 2} = 8.889\ kip$$

Point load from active pressure

$$P_{surcharge} := EFP_{active} \cdot h_{surcharge} \cdot H_c = 1.147\ kip$$

Point load from surcharge

$$M_{max\_c} := \left(P_{active} \cdot \frac{H_c}{3}\right) + \left(P_{surcharge} \cdot \frac{H_c}{2}\right) = 54.817\ kip \cdot ft$$

Max moment at base of channel

$$\frac{M_{max\_r}}{M_{max\_c}} = 0.008$$

Added channel loading from new concrete seatwall is within 1% of original demand

The channel walls for Big Dalton Wash and San Dimas Wash would have been originally designed for a vehicular loading surcharge. With the addition of the concrete seatwall and other landscaping improvements, it can be assumed that there will no longer be vehicular loading at the concrete seatwall locations. Therefore, the concrete seatwall will not induce additional load to the existing Big Dalton Wash and San Dimas Wash channel walls.

All other concrete seatwall locations are at a greater distance away from channel wall and/or at locations with a shorter channel wall. The calculations above consider worst case for this project.

Existing grade will be excavated to fill with decomposed granite. The decomposed granite sections will be replacing existing soil in kind. The weight of decomposed granite is similar to existing soil. Therefore, the decomposed granit sections will not induce additional loading to the existing Big Dalton Wash and San Dimas Wash channel walls.

Sample Bikeways
Concrete Header
Channel Impact Calculations

<u>Concrete Header:</u>
15'-6" Channel Wall, 0'-0" Distance

$D := 0\ ft$	Minimum distance between concrete header & channel wall
$H_c := 15.5\ ft$	Channel Wall Height
$w_c := 150\ \frac{lb}{ft^3}$	Unit weight of concrete
$\gamma_s := 120\ \frac{lb}{ft^3}$	Unit weight of soil
$H_{ch} := 12\ in$	Height of concrete header
$W_{ch} := 6\ in$	Width of concrete header
$b := 1\ ft$	Unit length
$V_{ch} := H_{ch} \cdot W_{ch} \cdot b = 0.5\ ft^3$	Volume of concrete header
$W_c := V_{ch} \cdot (w_c - \gamma_s) = 15\ lb$ *Concrete header is replacing equal volume of soil*	Additional weight fom concrete per foot of length
$P := \frac{W_c}{(W_{ch} \cdot b)} = 30\ \frac{lb}{ft^2}$	Additional bearing pressure from concrete header
$R := 0.33 \cdot P = 9.9\ \frac{lb}{ft^2}$	Lateral pressure on channel wall from additional weight of concrete header
$M_{max\_r} := \frac{R \cdot {H_c}^2}{2} \cdot b \cdot \left(\frac{1\ kip}{1000\ lb}\right) = 1.189\ kip \cdot ft$	Max moment at base of channel wall from additional weight of concrete header

Existing Demand on Channel Wall:

$EFP_{active} := 37\ psf$ Active earth pressure on channel wall

$h_{surcharge} := 2\ ft$ Surcharge height acting on channel wall

$P_{active} := EFP_{active} \cdot H_c^{\ 2} = 8.889\ kip$ Point load from active pressure

$P_{surcharge} := EFP_{active} \cdot h_{surcharge} \cdot H_c = 1.147\ kip$ Point load from surcharge

$M_{max\_c} := \left(P_{active} \cdot \frac{H_c}{3}\right) + \left(P_{surcharge} \cdot \frac{H_c}{2}\right) = 54.817\ kip \cdot ft$ Max moment at base of channel

$\frac{M_{max\_r}}{M_{max\_c}} = 0.022$ Added channel loading from new concrete header is within 5% of original demand

The channel walls for Big Dalton Wash and San Dimas Wash would have been originally designed for a vehicular loading surcharge. With the addition of the concrete header and other landscaping improvements, it can be assumed that there will no longer be vehicular loading at the concrete header locations. Therefore, the concrete header will not induce additional load to the existing Big Dalton Wash and San Dimas Wash channel walls.

All other concrete header locations are at a greater distance away from channel wall and/or at locations with a shorter channel wall. The calculations above consider worst case for this project.

Sample Bikeways
Bollard
Channel Impact Calculations

Bollard:

15'-6" Channel Wall, 1'-6" Distance

$D := 1.667\ ft$ — Minimum distance between bollard & channel wall

$H_c := 15.5\ ft$ — Channel Wall Height

$w_c := 150\ \frac{lbf}{ft^3}$ — Unit weight of concrete

$\gamma_s := 120\ \frac{lbf}{ft^3}$ — Unit weight of soil

$w_p := 18.99\ \frac{lbf}{ft}$ — Unit weight of steel pipe

$d_b := 6\ in$ — Diameter of bollard

$H_b := 3\ ft$ — Height of bollard above grave

$D_b := 2.5\ ft$ — Depth of bollard embedment

$D_{b\_fnd} := 3\ ft$ — Depth of bollard foundation

$d_{b\_fnd} := 1.5\ ft$ — Diameter of bollard foundation

$W_{b\_above} := w_p \cdot H_b + w_c \cdot \left(\frac{\pi \cdot d_b^{\ 2}}{4}\right) \cdot H_b = 145.327\ lbf$ — Weight of bollard above grade

$V_{b\_fnd} := \frac{\pi \cdot d_{b\_fnd}^{\ 2}}{4} \cdot D_{b\_fnd} = 5.301\ ft^3$ — Volume of bollard foundation

$W_{b\_fnd} := V_{b\_fnd} \cdot w_c + D_b \cdot w_p = 842.691\ lbf$ — Weight of bollard foundation

$W_c := W_{b\_above} + W_{b\_fnd} - V_{b\_fnd} \cdot \gamma_s = 351.845\ lbf$ — Additional weight from bollard

Bollard foundation is replacing equal volume of soil

$P := \frac{W_c}{\frac{\pi \cdot d_{b\_fnd}^{\ 2}}{4}} = 199.104\ psf$ — Additional bearing pressure from bollard

Using Boussinesq method forstrip loading to calculate the load applied to Existing Channel wall

CT TRENCHING AND SHORING MANUAL

4.8.3 Boussinesq Loads

Typically, there are three (3) types of Boussinesq Loads. They are as follows:

4.8.3.1 Strip Load

Strip loads are loads such as highways and railroads that are generally parallel to the wall.

The general equation for determining the pressure at distance h below the ground line is (See Figure 4-48):

$$\sigma_h = \frac{2Q}{\pi}[\beta_R - \sin\beta\cos(2\alpha)] \qquad \text{Eq. 4-67}$$

Where β_R is in radians.

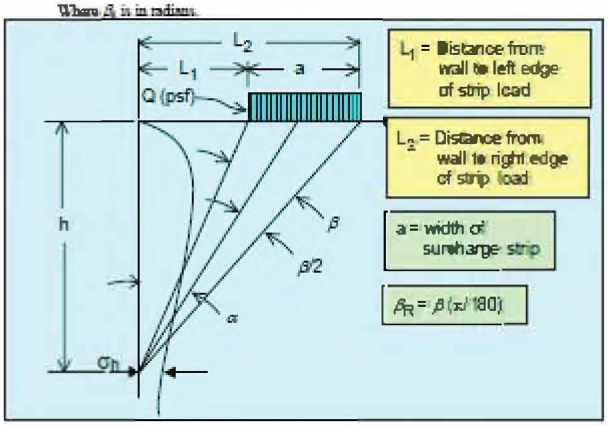


Figure 4-48. Boussinesq Type Strip Load

@ 5' from top of wall:

$\beta := 17.9\ deg$

$\alpha := 50.4\ deg$

$\beta_R := \beta = 0.312\ rad$

$Q := P = 199.104\ psf$

$\sigma_{h1} := \left(\frac{2 \cdot Q}{\pi}\right) \cdot \left(\beta_R - \sin(\beta) \cdot \cos(2 \cdot \alpha)\right) = 46.9\ psf$

@ 7' from top of wall:

$\beta := 15.8\ deg$

$\alpha := 31.1\ deg$

$\beta_R := \beta = 0.276\ rad$

$Q := P = 199.104\ psf$

$\sigma_{h2} := \left(\frac{2 \cdot Q}{\pi}\right) \cdot \left(\beta_R - \sin(\beta) \cdot \cos(2 \cdot \alpha)\right) = 18.858\ psf$

9' from top of wall:

$\beta := 12.3\ deg$

$\alpha := 21.9\ deg$

$\beta_R := \beta = 0.215\ rad$

$Q := P = 199.104\ psf$

$\sigma_{h3} := \left(\frac{2 \cdot Q}{\pi}\right) \cdot \left(\beta_R - \sin(\beta) \cdot \cos(2 \cdot \alpha)\right) = 7.722\ psf$

@ 11' from top of wall:

$\beta := 10.3\ deg$

$\alpha := 16.8\ deg$

$\beta_R := \beta = 0.18\ rad$

$Q := P = 199.104\ psf$

$\sigma_{h4} := \left(\frac{2 \cdot Q}{\pi}\right) \cdot \left(\beta_R - \sin(\beta) \cdot \cos(2 \cdot \alpha)\right) = 3.909\ psf$

@ 13' from top of wall:

$\beta := 8.1\ deg$

$\alpha := 13.6\ deg$

$\beta_R := \beta = 0.141\ rad$

$Q := P = 199.104\ psf$

$$\sigma_{h5} := \left(\frac{2 \cdot Q}{\pi}\right) \cdot \left(\beta_R - \sin(\beta) \cdot \cos(2 \cdot \alpha)\right) = 2.035 \ psf$$

@ 15.5' from top of wall:

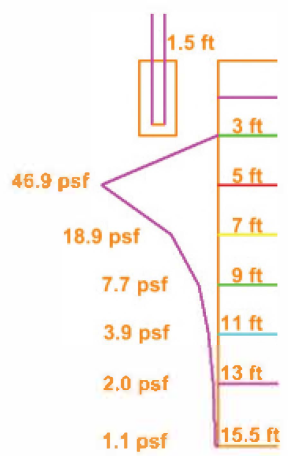
$\beta := 6.6 \ deg$

$\alpha := 10.9 \ deg$

$\beta_R := \beta = 0.115 \ rad$

$Q := P = 199.104 \ psf$

$$\sigma_{h6} := \left(\frac{2 \cdot Q}{\pi}\right) \cdot \left(\beta_R - \sin(\beta) \cdot \cos(2 \cdot \alpha)\right) = 1.074 \ psf$$



Average active pressure at every 2 ft to convert to retangular diagram and calculate the moment at the base of wall from the c.g. area of the rectangular active pressure

$w_1 := \sigma_{h1} \cdot 2 \ ft = 93.799 \ plf$ $\qquad h_1 := \frac{2 \ ft}{3} + 10.5 \ ft = 11.167 \ ft$

$w_2 := \frac{(\sigma_{h1} + \sigma_{h2})}{2} \cdot 2 \ ft = 65.757 \ plf$ $\qquad h_2 := 9.5 \ ft$

$w_3 := \frac{(\sigma_{h2} + \sigma_{h3})}{2} \cdot 2 \ ft = 26.579 \ plf$ $\qquad h_3 := 7.5 \ ft$

$w_4 := \frac{(\sigma_{h3} + \sigma_{h4})}{2} \cdot 2 \ ft = 11.631 \ plf$ $\qquad h_4 := 5.5 \ ft$

$$w_5 := \frac{(\sigma_{h4} + \sigma_{h5})}{2} \cdot 2\ ft = 5.944\ plf$$

$h_5 := 3.5\ ft$

$$w_6 := \frac{(\sigma_{h5} + \sigma_{h6})}{2} \cdot 2\ ft = 3.109\ plf$$

$h_6 := 1.25\ ft$

$b := 1\ ft$

$$M_o := (w_1 \cdot h_1 + w_2 \cdot h_2 + w_3 \cdot h_3 + w_4 \cdot h_4 + w_5 \cdot h_5 + w_6 \cdot h_6) \cdot b = 1.96\ kip \cdot ft$$

<u>Existing Demand on Channel Wall:</u>

$H_C := 15.5\ ft$ — Channel Wall Height

$EFP_{active} := 37\ psf$ — Active earth pressure on channel wall

$h_{surcharge} := 2\ ft$ — Surcharge height acting on channel wall

$P_{surcharge} := EFP_{active} \cdot h_{surcharge} \cdot H_C = 1.147\ kip$ — Point load from surcharge

$$M_{max\_c} := P_{surcharge} \cdot \frac{H_C}{2} = 8.889\ kip \cdot ft$$

Max moment at base of channel

$$\frac{M_o}{M_{max\_c}} = 0.221$$

ratio shows that the bollard load is less than 2 feet surcharge