$kip := 1000 \cdot lbf$ $ksi := 1000 \cdot psi$ $ksf := \frac{kip}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $psf := \frac{lbf}{ft^2}$

PANEL ATTACHMENT DESIGN

References:

1. *ASCE 7-02, Minimum Design Loads for Buildings and Other Structures*
2. *HILTI Technical Literature*

Data:

$b := 6m$	Panel width
$h := 1.5m$	Panel height
$t := 200mm$	Panel thickness, including concrete base and stone veneer
$\gamma := 150pcf$	Panel unit weight
$z := 1$ $H := 1$	Assume conservatively ratio of component to structure height is equal to 1.
$a_p := 1.25$	Architectural component coefficient, Table 9.6.2.2.
$R_p := 1.0$	Architectural component coefficient, Table 9.6.2.2.
$I_p := 1.5$	Importance factor (for all components needed for continued operation of the facility), Section 9.6.1.5.
$S_s := 0.40$	Mapped maximum considered earthquake spectral response accelereration at short periods, Figure 9.4.1.1(a)
$F_a := 2.0$	Site coeffcient for soil class E (inerpolated), Table 9.4.1.2.4a.

Please note that the maximum mapped response spectral acceleration, S_s is applicable for Site Class B. The site under consideration is Class E. Thus the site coefficient, Fa is used to adjust for site effects as per Section 9.4.1.2.4. Such calculations are presented on Page 2.

<u>Calculations:</u>

Horizontal restrainers will be designed to withstand a seismic force calculated in accordnace with ASCE 7-02.

$W_p := b \cdot h \cdot t \cdot \gamma$ $\quad W_p = 9.535\,\text{kip}$ Panel weight

$S_{MS} := F_a \cdot S_s$ $\quad S_{MS} = 0.8$ Maximum considered earthquake spectral response for short periods, Eq. 9.4.1.2.4-1

$S_{DS} := \frac{2}{3} S_{MS}$ $\quad S_{DS} = 0.533$ Design spectral response acceleration at short periods. Eq. 9.4.1.2.5-1

<u>Horizontal Seismic Force, Section 9.6.1.3:</u>

$$F_p := \min\left[1.6 S_{DS} \cdot I_p \cdot W_p, \max\left[\frac{0.4 a_p \cdot S_{DS} \cdot W_p}{\frac{R_p}{I_p}}\left(1 + 2 \cdot \frac{z}{H}\right), 0.3 S_{DS} \cdot I_p \cdot W_p\right]\right] \qquad F_p = 11.442\,\text{kip}$$

Above logical statement includes requirements of Equations 9.6.1.3-1 through 9.6.1.3-3.

<u>Vertical Seismic Force, Section 9.5.2.7:</u>

$D := W_p$ Dead load

$$F_v := 0.2 S_{DS} \cdot D \qquad F_v = 1.017\,\text{kip}$$

<u>Anchor Bolt Design:</u>

Design is based on HILTI 5/8" Dia. HSE 2421 Epoxy Adhesive Anchor with 5 5/8" min embedment depth. Actual system used may vary. The mortar bond between panels and existing retaining wall will not be inlcuded. Full sesimic force will be equally distributed to top and bottom restaraints.

$n_h := 3$ number of horizontal restrainer per row

$n_v := 4$ number of vertical restrainers

$T_c := 6250\mathrm{lbf}$ Allowable tension based on bond or concrete capacity

$V_c := 5795\mathrm{lbf}$ Allowable shear based on bond or concrete capacity

$T_b := 12655\mathrm{lbf}$ Allowable tensile strength of 5/8" Dia. stainless steel bolt

$V_b := 6520\mathrm{lbf}$ Allowable shear strength of 5/8" Dia. stainless steel bolt

Vertical Support Design - Shelf Angle Design:

Use L5x5x1/2x2ft

$F_y := 36\text{ksi}$

$b_{a1} := 2\text{ft}$ Width of an angle

$t_a := 0.5\text{in}$ Thickness of an angle

$l_1 := 3\text{in}$ Moment arm

Shelf angle:

$I_1 := \dfrac{b_{a1} \cdot t_a^{\,3}}{12}$ $I_1 = 0.25\,\text{in}^4$ Moment of inertia

$M_1 := \dfrac{\left(W_p + 0.7F_v\right) \cdot l_1}{n_v}$ $M_1 = 0.64\,\text{kip} \cdot \text{ft}$ Bending moment

$f_{y1} := \dfrac{M_1 \cdot \dfrac{t_a}{2}}{I_1}$ $f_{y1} = 7.685\,\text{ksi}$ Bending stress

$\text{Check}_3 := \left| \begin{array}{ll} \text{"OK"} & \text{if } f_{y1} \le 0.55F_y \\ \text{"NG"} & \text{otherwise} \end{array} \right.$ $\text{Check}_3 = \text{"OK"}$

Vertical Support Design - Anchor Bolt Design:

Anchor Bolt Loads:

Anchor bolt tension is due to the eccentric loading on the shelf angle and direct tension due to Fp.

$$T_2 := \frac{\left(W_p + 0.7F_v\right)\cdot 3in}{3.5in\left(2n_v\right)} + \frac{\frac{F_p}{2}}{2\cdot n_v}$$

$T_2 = 1.813\,kip$

Tension in anchors

$$V_2 := \frac{W_p + 0.7F_v}{n_v}$$

$V_2 = 2.562\,kip$

Shear in anchors

Combined Shear and Tension Loading:

$f_{a2} := 1.0$

$f_{r2} := 1.0$

Anchor spacing and edge distance adjustment factors are based on HILTI Technical Manual, Section 4.2.5. The spacing and edge distance will be no less than 1.5 time the embedment depth.

$T_{recd} := \min\left[T_c\cdot\left(f_{a2}\cdot f_{r2}\right), T_b\right]$ $\quad T_{recd} = 6.25\,kip$ Tensile strength

$V_{recd} := \min\left[V_c\cdot\left(f_{a2}\cdot f_{r2}\right), V_b\right]$ $\quad V_{recd} = 5.795\,kip$ Shear strength

Combined effects of shear are in accordance with HILTI recommendations for Epoxy Adhesive Anchor System.

$$Check_4 := \begin{cases} \text{"OK"} & \text{if } \left(\frac{T_2}{T_{recd}}\right)^{\frac{5}{3}} + \left(\frac{V_2}{V_{recd}}\right)^{\frac{5}{3}} \le 1.0 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$Check_4 = \text{"OK"}$

Horizontal Restrainer - Anchor Bolt Design

$f_a := 1.0$

Anchor spacing adjustment factor per HILTI Manual, Section 4.2.5. Bolt spacing shall be no less than the embedment depth.

$f_{rn} := 0.65$

Edge distance adjustment factor per HILTI Manual, Section 4.2.5. Edge distance shall be no less than 50% of the embedment depth.

$$F_t := \frac{\frac{F_p}{2}}{n_h}$$

$F_t = 1.907\,\text{kip}$

Tension in anchors

$$T_{rec} := T_c \cdot (f_a \cdot f_{rn})$$

$T_{rec} = 4.063\,\text{kip}$

Tensile capacity

$$Check_1 := \begin{vmatrix} \text{"OK"} & \text{if } F_t \le T_{rec} \\ \text{"NG"} & \text{otherwise} \end{vmatrix}$$

$Check_1 = \text{"OK"}$

Horizontal Restraint - Connection Angle Design:

Use L4x3x1/2

$F_y := 36\text{ksi}$

$b_{a2} := 6\text{in}$ Width of an angle resisting bending

$t_{a2} := 0.5\text{in}$ Thickness of an angle

$l_2 := 3\text{in}$ Moment arm

$I_2 := \dfrac{b_{a2} \cdot t_a^{\,3}}{12}$ $I_2 = 0.063\,\text{in}^4$ Moment of inertia

$M_2 := \dfrac{\dfrac{F_p}{2} \cdot l_2}{2n_h}$ $M_2 = 0.238\,\text{kip} \cdot \text{ft}$ Bending moment

$f_{y2} := \dfrac{M_2 \cdot \dfrac{t_{a2}}{2}}{I_2}$ $f_{y2} = 11.442\,\text{ksi}$ Bending stress

$\text{Check}_5 := \left| \begin{array}{ll} \text{"OK"} & \text{if } f_{y2} \le 0.55F_y \\ \text{"NG"} & \text{otherwise} \end{array} \right.$ $\text{Check}_5 = \text{"OK"}$

Pull-out Cone:

$f'_c := 3800 \cdot psi$ Minimum concrete strength is based on cores

$l := 115mm$ Embedment depth

$A_p := \pi l^2$ Projected area of the pull out cone

$f_r := 7.5\sqrt{f'_c \cdot psi}$ Concrete modulus of rapture

$F_p := A_p \cdot (0.21 \cdot f_r)$ $F_p = 6.252\,kip$ Allowable Pull-out Force

$$Check_6 := \begin{cases} \text{"OK"} & \text{if } \max(T_2, F_t) \le F_p \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$Check_6 = \text{"OK"}$

Please note that the allowable shear bond capacity of concrete governs the design.