FY 2014 Investigation
Integrative Genomics Building (IGB)
Previous Plans and Calculations

September 30, 2014 Compilation
Lawrence Berkeley Laboratory
Seismic Slope Stabilization Project
Berkeley, California

PFFA Job No. 1509-025
September 30, 1992

Structural Calculations

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NOTES:

1. Minimum depth of embedment for design profile 1 is 35 feet. Refer to report text for details and Table 7 for required shear resistance.

2. Refer to Site Plan-Building 51, Figure 5 for location of design profile 1.

3. Refer to text of report for description of the pressure diagrams.

4. The depths noted above are referenced to the existing ground surface at the top of the SPSE system.

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**DESIGN PROFILE 1**

Kaldveer Associates
Geoscience Consultants
A California Corporation

**LAWRENCE BERKELEY LABORATORY**
SEISMIC SLOPE STABILIZATION PROJECT
Berkeley, California

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\[ P'_u = P_u - 60H \]

where \( H \) = Height of Retaining Wall

\( P_u \) = Distributed Design Force Previously Used in Computation for Design Profiles I to V

Resisting forces of the retaining wall will be developed in the SPCE design in accordance with our previous recommendations.
Design Profile 1 - with balanced moments

- $P_p = 5.537$ ksf
- $99.9$ ksf tieback
- $2000$ ksf tieback

Slide Plane Depth = 20.00'

- $F_d = 380$ k
- $V = 280$ k
- $-M = 1207$ k @ Slide Plane

Slide Plane

- $M(max) = 1883$ k @ 15.16'
- 7.75'

Stronger Mat'1

- $23.96'$
- $18.118$ k
- $18.85'$
- $10.82'$
- $24.387$ ksf

Bottom of Pier

- $M(max) = 1915$ k @ 40.60'
- 42.82'

Pier Depth $= 62.82$'

Pressure Diagram

Shear Diagram

Moment Diagram

Design Profile I

7/27/92
pier spacing = 5,000'
passive pressure width = 3,500'

99.9 k Tieback

# .5" Ø strands = 1
# .6" Ø strands = 1

Pu = 57,750 ksf = 16.5 ksf * pier width

M(max) = 1,883 k' @ 15.16'
7.75'

-v = 119 k

10,150 ksf uniform
0.333 kcf fluid

Pier Depth = 59.88'

M = 1,529 k'

Iterate Pier Depth to Match

M @ Stronger Material

19,194 ksf uniform
1,500 ksf fluid

M(max) = 2,015 k' @ 36.51'
12.68'

Bottom of Pier

38.214 ksf
38.285 ksf

= passive pressure width * (10 + .095*(pier depth - 50))
Design Profile I - with retaining wall

Ground Level

Slide Plane Depth = 20.00'

Stronger Mat'l

Bottom of Pier

Pressure Diagram

Shear Diagram

Moment Diagram

EFP = 0.120 kcf M = 51.2 'k
h = 8.00' V = 19.2 k
117.2 k Tieback

# .5" Ø struts = 0
# .6" Ø struts = 2

12.36'

Pu = 56070 ksf = 16.0 ksf * pier width

M(max) = 1825 'k @ 15.32'

7.64'

F = 380 k V = 262 k -M = 1212 'k @ Slide Plane

M = 1306 'k @ Stronger Material

Iterate Pier Depth to Match M @ Stronger Material

M(max) = 1605 'k @ 37.63'

12.23'

Pier Depth = 57.23'

36.808 ksf
37.404 ksf = passive pressure width * (10+0.95*(pier depth-50))
<table>
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<td>0.000 0.000 0.000 0.045</td>
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<tr>
<td>1.015</td>
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<tr>
<td>inch</td>
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</table>

Distance 0.000 L/6 L/4 L/3 L/2 2L/3 3L/4 5L/6 6L/6 65.230

-1.169
NOTES:

1. Minimum depth of embedment for design profile II is 30 feet. Refer to report test for details and Table 7 for required shear resistance.

2. Refer to Site Plan-Building 51, Figure 5 for location of design profile II.

3. Refer to text of report for description of the pressure diagrams.

4. The depths noted above are referenced to the existing ground surface at the top of the proposed SPSE system.

Required shearing resistance ($F_d$)

- $H_1 = 45$ Feet
- $H_2 =$ Varies

DESIGN PROFILE II

Kaldveer Associates
Geoscience Consultants
A California Corporation

LAWRENCE BERKELEY LABORATORY
SEISMIC SLOPE STABILIZATION PROJECT
Berkeley, California

PROJECT NO. DATE Figure 21
K1228-1-164 February 1992
Design Profile II · with balanced moments

- Pier spacing = 6.000'
- Passive pressure width = 3.500'
- Pp = 5.485 ksf

**Ground Level**

- Fd = 400 k
- V = 283 k

**Slide Plane Depth = 20.00’**

- M = 1443 k'
- M(max) = 2078 k'
- Pier Depth = 59.58'

**Stronger Material**

- M = 1443 k'
- M(max) = 1800 k'

**Bottom of Pier**

- M = 19194 k'st uniform
- M(max) = 2078 k'

---

**Design Profile II.a**

7/29/92

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![Pressure Diagram](image)

![Shear Diagram](image)

![Moment Diagram](image)
NOTES:

1. Minimum depth of embedment for design profile III is 25 feet. Refer to report text for details and Table 9 for required shear resistance.

2. Refer to Site Plan-Building 77, Figure 6 for location of design profile III.

3. Refer to text of report for description of the pressure diagrams.

4. The depths noted above are referenced to the existing ground surface at the top of the proposed SPSE system.

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**DESIGN PROFILE III**

**LAWRENCE BERKELEY LABORATORY**

**SEISMIC SLOPE STABILIZATION PROJECT**

Berkeley, California

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Kaldveer Associates
Geoscience Consultants
A California Corporation
Design Profile III - with balanced moments

Ground Level

Slide Plane Depth = 13.00'

Slide Plane

Stronger Mat'l

Bottom of Pier

Pier spacing = 12,000'
passive pressure width = 7,000'

Pp = 9,584 ksf

M(max) = 1447 k @ 9.73'

M(max) = 1953 k @ 28.87'

123.9 k Tieback

# .5" Ø strands = 3
# .6" Ø strands = 0

Pu = 101,500 ksf = 14.5 ksf * pier width

V = 332 k

-Fd = 456 k

-F = 904 k

Strands

0.00'

18.69'

15.16'

8.73'

38.151 ksf

Pressure Diagram

Shear Diagram

Moment Diagram

Design Profile III

7/27/92

Page # C2
NOTES:

1. Minimum depth of embedment for design profile IV is 30 feet. Refer to report test for details and Table 9 for required shear resistance.

2. Refer to Site Plan—Building 77, Figure 6 for location of design profile IV.

3. Refer to text of report for description of the pressure diagrams.

4. The depths noted above are referenced to the existing ground surface at the top of the proposed SPSE system.
Design Profile IV - with balanced moments

Pp = 4,868 ksf
Pier spacing = 8.000'
passive pressure width = 3.500'
1.15Fd = 50.6 klf

# .5" Ø strands = 2
# .6" Ø strands = 1

Pu = 50.750 ksf = 14.5 ksf * pier width

M(max) = 1652 'k
8.23'

M(max) = 2037 'k
37.88'

9.153 ksf uniform
0.333 kcf fluid

9.153 ksf uniform
0.333 kcf fluid

M(max) = 2037 'k
37.88'

23,332 ksf

Pressure Diagram
Shear Diagram
Moment Diagram

Design Profile IV
7/27/92
Design Profile IV - with balanced moments

Slide Plane Depth = 17.00'

Pp = 4.868 ksf

141.2 k Tieback

# .5" Ø strands = 2
# .6" Ø strands = 1

8.23'

Pu = 50.750 ksf = 14.5 ksf * pier width

M(max) = 1652 k @ 11.81'

8.77'

Fd = 405 k

V = 264 k

-M = 967 k @ Slide Plane

Slide Plane

28.00'

-V = 123 k

M = 1609 k @ Stronger Material

Iterate Pier Depth to Match

M @ Stronger Material

18.463 ksf uniform

1.470 kcf fluid

18.563 ksf

13.00'

8.71'

Bottom of Pier

37.678 ksf

37.684 ksf = passive pressure width * (10-.095*(pier depth-50)

Pier Depth = 58.07'

9.153 ksf uniform

0.333 kcf fluid

M(max) = 2024 k @ 37.00'

13.07'

Stronger Mat'l

0.07'

Pressure Diagram

Shear Diagram

Moment Diagram

Design Profile IV.a

7/29/92

Page 1
NOTES:

1. Minimum depth of embedment for design profile V is 35 feet. Refer to report test for details and Table 9 for required shear resistance.

2. Refer to Site Plan—Building 77, Figure 6 for location of design profile.

3. Refer to text of report for description of the pressure diagrams.

4. The depths noted above are referenced to the existing ground surface at the top of the proposed SPSE system.
Design Profile V - with balanced moments

pier spacing = 12,000'
passive pressure width = 7,000'
1.15Fq = 32.2 klf

Ground Level

Slide Plane Depth = 24.00'

Fd = 386 k
V = 386 k

Pp = 12,433 ksf

Slide Plane

11.00'

V = 94 k
M = 1755 k

Stronger Matl

1.92'

71,275 ksf

11.21'

5.88'

Bottom of Pier

78,729 ksf

Pier Depth = 48.13'

M(max) = 1928 k' @ 18.98'

7.66'

M = 1755 k

Iterate Pier Depth to Match M @ Stronger Material

70,000 ksf uniform
0.665 kcf fluid

M(max) = 1817 k' @ 36.33'

13.13'

Pressure Diagram

Shear Diagram

Moment Diagram

Design Profile V.a

7/27/92
TIE BACK DESIGN - BONDED LENGTH

\[ F = \frac{2}{\sqrt{5}} \cdot \frac{5}{12} \pi \cdot 2500 = 2927 \text{ PLF} \]

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<th>LENGTH REQ'D</th>
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<td>I</td>
<td>17.2 k</td>
<td>40'</td>
</tr>
<tr>
<td>II</td>
<td>141.2 k</td>
<td>48'</td>
</tr>
<tr>
<td>III</td>
<td>123.9 k</td>
<td>42'</td>
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<tr>
<td>IV</td>
<td>141.2 k</td>
<td>48'</td>
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</table>
TIE BACK CONNECTION DESIGN

WELD GROUP

\[ \begin{align*}
S_1 &= 512 \text{ in}^2 \\
f_w &= 395 \text{ lb/in}^2 \\
&S_0 = 316 \text{ in}^2 \\
f_w &= 395 \text{ lb/in}^2 \\
\end{align*} \]

Combined Weld Stress:

\[\left(\frac{(\frac{158}{2})(.72)}{\frac{2(37)}{2}}\right)^{1/2} \left(\frac{1.25}{2}\right)^{1/2} = 1.25 \text{ ksf}\]

\[1.22 < 4.65 \text{ ksf} \]

Check 6" Tube, L = 51/2" P = 158k M = \frac{158k(5\frac{1}{2})}{4}

For 6" x 6" x 8"

\[\frac{S}{C} = \frac{6(1.875)}{0.765} = 765 \text{ in}^3\]

\[f_b = \frac{217}{4} \text{ ksf} \text{ too high, see below for revised calc}\]

\[\begin{align*}
14'' & M = 39.5 (1.75'') = 69 \text{ ksf} \\
1'' f_b & = 69 \text{ ksf} \\
2'' f_b & = 17 \text{ ksf} \\
1.5'' f_b & = 26 \text{ ksf} \\
\end{align*} \]

The 5" Tube Bearing \[\frac{158}{2(2.5)} = 15.8 \text{ ksf} \]

\[\frac{12}{12} \text{ ksf} M = 59.5 (1.25) = 49\]
Lagging Design

Piers spaced c. 6'-0 O.C., center to center of supports = 5.5'

80 H Static
120 H Pseudo-Static

\[
\frac{120}{1.33 \text{ Allow Stress Incl.}} = 90.2 \\
\text{Use 90.2} H \% \text{ Stress Incl.}
\]

For D.F. #2 \( F_0 = 1250 \) \( S = \frac{12+2}{6} = 2.4 \)

\[
F_0 = \frac{M}{S} = 1250 = (90.2H) \frac{5.5^2}{8} (12)
\]

\[
H = 0.6108 + z
\]

For \( z = 1.5 \) \( H = 1.374' \)

\( z = 2.5 \) \( H = 3.818' \)

\( z = 3.5 \) \( H = 7.482' \)

Use 3 x 12 Lagging up to 4'-0 deep

4 x 12 Lagging up to 8'-0 deep

For double W6 per Pier \( L = 3.88' \) \& \( L = 2' \)

For \( L = 4' \) \( H = \frac{1.26}{1.25} + z \) For \( L = 2' \) \( H = 4.62' + z \) For \( L = 2.83' \) \( H = 2.307 + z \)

\( z = 1.5 \) \( H = 2.8' \)

\( z = 2.5 \) \( H = 7.88' \)
CHECK LAGGING FOR SHEAR STRESS

For $L = 2'$, $H = 80'2+CM$ $V = (90.2 H)(\frac{1}{2}) = 722' \text{ ft}$

$f_v = \frac{1.5 V}{(1.5')(12'')} = 60 < 95 \text{ psi}$

For $L = 3'$, $H = 5'2+CM$ $V = 676$ $f_v = 50 < 95 \text{ psi}$

$L = 3'$, $H = 8'3+CM$ $V = 1082$ $f_v = 54 < 95 \text{ psi}$

For $L = 4'$, $H = 3'2+CM$ $V = 541$ $f_v = 45 < 95 \text{ psi}$

$L = 4'$, $H = 8'3+CM$ $V = 1443$ $f_v = 72 < 95 \text{ psi}$
SOLDIER BEAMS

For \( s = 6', h = 8' \)

\[
M = \frac{k \cdot f^3}{6} = \frac{0.090 \times 6 \times (8')^3}{6} = 46 \text{ kft}
\]

2-WC \times 20 \quad S = 2(13.4) = 26.8 \quad \sigma = 20.6 \text{ kips/ft}

\[
[90 \times 7.5 = 675 \text{ psf}] \times [2' \times 18'] = 1.35 \text{ k/ft}
\]

\[
M = 1.35 \text{ k}(2') = 2.7 \text{ k/ft}
\]

Check 4" flange, \( S = \frac{12(2.25)}{6} = 0.125 \)

\[
\frac{\sigma}{0.125} = 21.6 \text{ kpsi}
\]
PLANS FOR:
SLOPE AND SEISMIC STABILIZATION PROJECT - PHASE II
SUBCONTRACT No. 711
LAWRENCE BERKELEY LABORATORY
UNIVERSITY OF CALIFORNIA
BERKELEY, CALIFORNIA 94720
DECEMBER, 1994

TYPICAL BUILDING 77 BENCH

TYPICAL BUILDING 51 BENCH
STRUCTURAL CALCULATIONS

Calculations for LBNL Bevatron D & D
Retaining Walls East of Column Line Y
Berkeley, California

March 10, 2010

Cartwright Project #: 108096.2

Prepared For:
Lawrence Berkeley National Lab

Prepared By:

CARTWRIGHT
ENGINEERS

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RETAINING WALL DESIGN

A design based on pre-construction soil reports provided by LBWJ Active Backfill Material is unknown therefore existing conditions must be verified during construction.

1. ROAD SURCHARGE NEGLECTED DUE TO DISTANCE FROM WALL

GSE 7.5 H (AASHTO S 5.2)

2. DEEDED CONDITIONS ASSUMED PER WALL DETAILS FOR SURROUNDING RETAINING WALLS AND ORIGINAL WALL ONLY

3. 2 FEET OF SOIL USED TO COMPENSATE FOR SLIP ANGLE θ = 30° WITH 2 FT OF

SHEET A 6 = O ALLOWING THE USE OF LAG SCALES METHOD AS EXPLAINED IN THE CHAINING TRENCHING AND SHALING MANUAL SECTION G 0 10

q = 240 psf

LATERAL EARTH PRESSURE

\[ q_{50\%} = 0.65 \times K_a \times H (AASHTO \text{ FIG. } 5.7 A) \text{(SHEET 17)} \]

\[ K_a = 1.78 \text{ (SEE SHEET 18)} \]

\[ q_{50\%} = 9.12 \text{ PSF} \]

\[ q_{TOTAL} = 100 + 240 = 1140 \text{ PSF} \]

\[ \theta = \frac{q_{TOTAL} \times \text{HEIGHT} \times \text{SPACING}}{1140 \times \text{FT} \times \text{FT} \times \text{FT}} = 85 \text{ S KIPS} \]

\[ \text{Four Back Spacing} \]

2. PASSIVE PRESSURE AT TBD (NEGLIGENCE DUE TO SHAPE)

FOOTING + UNCERTAINTY AFFECTS OF TIED SURROUNDING WALL

THIS APPROACH IS SEEN AS CONSERVATIVE.
FS OVERTURN (SOUTH ROOM)

MOMENT OVERTURN

\( 85.5(9.83) = 840.72 \) k-ft

LAYOUT MOMENT (144 KIPS)

FS-TIM = 48 Kips (15.83 Ft) = 756 K-ft
FS-TOM = 48 Kips (0.85 Ft) = 40.4 K-ft
FS-TOM = 48 Kips (5.85 Ft) = 285.6 K-ft

\( \Sigma = 1488 \)

\[
\text{FS = Momentturn} \rightarrow \frac{1488 \text{ K-ft}}{840.72 \text{ K-ft}} = 1.77 > 1.5 \text{ OK}
\]

FS = SLIDE

Driving force = 85.5 Kips

Resisting force = 415 (3) = 1245 K
3 Pins

\[
\text{FS = 1245 K} = 1.68 > 1.5 \text{ OK}
\]
F.S. OVERTURN (NORTH ROOM)

MOMENT OVERTURN

\[ 86.5 \times 12.17^2 = 1041.54 \text{ k-ft} \]

\[ 48k \times 12.17 = 580.32 \text{ k-ft} \]

RIGHTING MOMENT

\[ 48k \times 12.17 \times 12 = 730.44 \text{ k-ft} \]

For T1 = 48 kips (17.47) = 848 k-ft

For T2 = 48 kips (12.67) = 608 k-ft

For T3 = 48 kips (7.67) = 368 k-ft

\[ \varepsilon = 1824 \text{ k-ft} \]

F.S. = MAXIMUM OVERTURN = \[ \frac{1824 \text{ k-ft}}{1041.54 \text{ k-ft}} = 1.75 > 1.5 \Rightarrow OK \]

F.S. SLIDE

F.S. = RESISTING FORCE

\[ \frac{T1}{T2} = \frac{48k}{48k} = 1 \]

\[ T2 = 48k \text{ kips} \]

\[ T3 = 48k \text{ kips} \]

DRIVING FORCE

\[ = 85 \text{ kips} \]

\[ F.S. = \frac{144k}{85k} = 1.69 > 1.5 \Rightarrow OK \]
WALL CHECK SOUTH FORM

SHEAR CHECK
FROM F I S A  (SHEETS 18-29)

\[ \text{SHEAR } x = 10.53 \text{ K} \ (\text{PLATE 975A}) \ (\text{IDENTIFIED BY HOTSPOTS ON ENVELOPE PLT TYP}) \]
\[ \text{SHEAR } y = 10.14 \text{ K} \ (\text{PLATE 56B}) \]

WALL SHEAR CAPACITY = 13.66 K (SEE SHEET 5.)

DIC RATIO = 10.53, 10.14 K = 0.77 < 1.0 OR OK

BENDING CHECK

VALUES TAKEN FROM F I S A  - LOCALIZED HOTSPOTS IDENTIFIED ON THE CONTACT PLATE (SHEET 18-29) VALUES FOR PLATES SURROUNDING HOTSPOTS (TYPICALLY ALONG TIEBACKS) WERE THEN AVERAGED TO GIVE VALUES TAKEN FROM THE PLATE FORCES. REPORT TO OBTAIN THE MAX POSITIVE AND NEGATIVE MOMENT. (SHEETS 23 - 60)

\[ \text{MAX. POSITIVE MOMENT: (X) PLATES } 1570, 1571, 1540, 1541 \]
\[ X + XY = 5.84 \text{ K-FT/FT} \]

\[ \text{MAX. POSITIVE MOMENT: (Y) PLATES } 1240, 1190, 1220, 1250 \]
\[ Y + XY = 4.99 \text{ K-FT} \]

\[ \text{MAX NEGATIVE MOMENT: (X) PLATES } 1666, 1695, 1665, 1695 \]
\[ X + XY = -4.17 \text{ K-FT/FT} \]

\[ \text{MAX NEGATIVE MOMENT: (Y) PLATES } 1605, 1655, 1605, 1695, 1725, 1755 \]
\[ Y + XY = -5.83 \text{ K-FT/FT} \]

\[ \text{MAX MOMENT CAPACITY} = 6.1 \text{ K-FT (SHEET 6)} \]

DIC RATIO = 5.84/6.1 = 0.96 < 1.0 OR OK
Shear Capacity Calculator

Location ID: South Retaining Wall (Worst Case)

\[ b = 12 \text{ in} \]
\[ d = 12 \text{ in} \]
\[ f_c = 4000 \text{ psi} \]
\[ \phi = 0.75 \]
\[ V_c \text{ (one way)} = 13,661.04 \text{ lbs} \]

Shear Calculator: Two Way

\[ b = 80 \text{ in} \]
\[ d = 12 \text{ in} \]
\[ f_c = 4000 \text{ psi} \]
\[ \phi = 0.75 \]
\[ \text{Ratio of column} = 1 \]
\[ \text{Column Location} = 30 \]
\[ V_c \text{ C} = 182,147.19 \text{ lbs} \]
\[ V_c \text{ A} = 273,220.79 \text{ lbs} \]
\[ V_c \text{ B} = 296,989.19 \text{ lbs} \]
\[ V_c \text{ (two way)} = 182,147.19 \text{ lbs} \]

Punching Shear Demand (Worst Case)

\[ T_{1.3} = 51100 \text{ lbs} \]

(See Sheet 10)

Punching Shear Demand Capacity Ratio

0.28 OK
Moment Calculator

Location ID: North Room Retaining Wall Located at Col. Line 7 From Y to Z (Worst Case)

\[ M_u = 6.06 \text{ k ft} \quad \text{(See Sheet 7)} \]

\[ A_s = 0.31 \text{ in}^2 \]

\[ \# \text{ of bars} = 0.55 \quad \text{Equiv. Number of bars per foot (#5 bar at 16" oc.)*} \]

\[ d = 12 \text{ in} \]

\[ b = 12 \text{ in}^* \]

\[ h = 14 \text{ in} \]

\[ f_c = 4 \text{ ksi} \]

\[ f_y = 40 \text{ ksi} \]

\[ E_s = 29000 \text{ ksi} \]

\[ \phi = 0.9 \]

\[ a = 0.167 \]

\[ a = \frac{A_s f_y}{0.85 f_c c * b} \]

Check c/d

\[ c = 0.197 \]

\[ c/d = 0.016 \quad \phi = 0.9 \]

\[ c = \frac{a}{\beta_1} \]

Check pmax

\[ \beta_1 = 0.85 \]

\[ \varepsilon_s = 0.005 \]

\[ p_{\text{max}} = 0.027 \]

\[ P = 0.001 \quad \text{ok} \]

\[ \rho_{\text{max}} = 0.85 \beta_1 \frac{f_c}{f_y} \left( \frac{e_{\text{cu}}}{e_{\text{eu}} + 0.004} \right) \]

\[ \rho = \frac{A_s}{b * d} \]

Deflection

Length = 5 ft

\[ W_{DL} = 0 \] pfl

\[ W_{LL} = 1140 \] pfl

Pinned

\[ M_{u, \text{fixed}} = 28.5 \text{ lb in} \]

\[ M_{u, \text{pinned}} = 42.75 \text{ lb in} \]

\[ M_u = 42.75 \text{ lb in} \]

\[ E_c = 3,604,996.53 \text{ psi} \]

\[ n = 8.0444 \]

\[ p = 0.0012 \]

\[ k = 0.1288 \]

\[ f_c = 474.34 \text{ psi} \]

\[ M_{\text{cr}} = 185,941.93 \text{ lb in} \]

\[ I_g = 2,744.00 \text{ in}^4 \]

\[ I_{cr} = 164.67 \text{ in}^4 \]

\[ I_o = 2,744.00 \text{ in}^4 \]

*Based on a 5 foot section of wall with 1 bar cut during tieback installation.
WALL CHECK - NORTH BACK

SHEAR CHECK

From K.A.C. (Sheets 0, 17)

\[
\begin{align*}
\text{SHEAR } x &= -10.00 \text{ kips (Plate 454)} \quad \text{(OK)} \\
\text{SHEAR } y &= -9.73 \text{ kips (Plate 515)} \\
\text{WALL SHEAR CAPACITY} &= 13.66 \text{ kips (Sheet 5)}
\end{align*}
\]

\[
\frac{V}{C \text{ kips}} = 10.00 \div 13.66 = 0.73 < 1 \cdot \text{OK}
\]

BENDING CHECK

VALUES TAKEN FROM K.A.C. (SEE NOTE ON SHEET 4) (SHEETS 68-69, 66-90)

MAX POSITIVE MOMENT (X) PLATES 110.55, 116.5, 125, 129.5

\[
\begin{align*}
\text{MAX POSITIVE MOMENT (X)} &= 3.02 \text{ kips ft/ft}
\end{align*}
\]

MAX POSITIVE MOMENT (Y) PLATES 110.55, 116.5, 118.5, 125, 129.5, 127.5

\[
\begin{align*}
\text{MAX POSITIVE MOMENT (Y)} &= 6.06 \text{ kips ft/ft}
\end{align*}
\]

MAX NEGATIVE MOMENT (X) PLATES 248, 249, 312, 313

\[
\begin{align*}
\text{MAX NEGATIVE MOMENT (X)} &= 5.73 \text{ kips ft/ft}
\end{align*}
\]

MAX NEGATIVE MOMENT (Y) PLATES 248, 249, 312, 313

\[
\begin{align*}
\text{MAX NEGATIVE MOMENT (Y)} &= 5.96 \text{ kips ft/ft}
\end{align*}
\]

MAX MOMENT CAPACITY = 0.9 kips ft (Sheet 67)

\[
\frac{V}{C \text{ kips}} = 60.00 \div 60.10 = 0.99 < 1 \cdot \text{OK}
\]
CHECK WALL FOOTING BEARING CAPACITY / DEMAND (SOUTH EOM)

WALL AXIAL LOAD (WORST CASE)  

WALL DEAD LOAD  
= 150 PSF (14.17 FT) (16 FT) = 2800 PLF  

FOOTING DEAD LOAD  
= 150 PSF (1.5 FT) (1.35 FT) = 300 VPLF  

GRADE BEAM DEAD LOAD  
= 150 PSF (15) = 150 PLF  

FLOOR TRIB DEAD LOAD  
= 150 PSF (5.5 FT) = 825 PLF  

WDL= 2800 + 300 + 825 + 150 = 3551 PLF  

VERTICAL COMPONENT OF THE BACK (WORST CASE)  

TV = TV1 + TV2 + TV3  

TV1 = TV(60") = 17.47 kips  

TV2 = TV(3") = 52.41 / 5 FT = 10.48 kips  

TV3 = WDL = 2800 + 300 + 3551 = 14.1 kips  

14.1 / 15 = 9.36 ksf  

SOIL BEARING CAPACITY = 10 ksf (ASSUME MIN FOOTING DEPTH)  

D/R RATIO = 9.36 / 10 = 0.94 < 1.0 % OK
CHECK WALL FOOTING BEARING CAPACITY / DEMAND (NORTH BOOM)

WALL AXIAL LOAD (WORST CASE)

WALL DEAD LOAD

\[ P = 150 \text{pcf} \times (1.5)(11.5 \text{ ft}) = 3,258 \text{ pcf} \]

FOOTING DEAD LOAD

\[ P = 150 \text{pcf} \times (1.5)(11.5 \text{ ft}) = 3,258 \text{ pcf} \]

GRADE BEAM DEAD LOAD

\[ P = 150 \text{pcf} \times (1.5)(10) = 1,875 \text{ pcf} \]

FLOOR TIE BAR DEAD LOAD

\[ P = 150 \text{pcf} \times (1.5)(1.5 \text{ ft}) = 113 \text{ pcf} \]

Vertical Component of Tie Bar (Worst Case)

\[ T_v = T_{v1} + T_{v2} + T_{v3} \]

\[ T_{v3} = \tan 23^\circ \times (14 \text{ ft}) = 17.47 \text{ kips} \]

\[ T_v = 17.47 \right + 10.98 \right = 52.41 \text{ kips / sq ft} \]

Total Vertical Load:

\[ 52.41 \text{ kips / sq ft} \times 10.5 \text{ ft} = 551.46 \text{ kips} \]

\[ 14.48 \text{ kips / sq ft} = 9.65 \text{ ksf} \]

\[ 9.65 \text{ ksf} / 100 \text{ ksf} = 0.97 < 1.0 \text{ OK} \]
TIEBACK DESIGN

DESIGN LOAD

Total Load = 48 kips

DL = 11.5 kips

(LDL) = 61.1 kips

LOCKOFF LOAD = DL * 8 = 91 k (CALTRANS 9.14)

PULLOUT CAPACITY

ULTIMATE BOND STRESS = 4 k/ft (AASHTO TSL 5.7.6.78)

FOR HARD CLAY, MUST BE CONFIRMED BY TESTING DURING INSTALLATION

FOR TIEBACK LENGTH = 30'

MIN CLAY - BONDED LENGTH IS (AASHTO TROJE 5.7.1A)

MIN BONDED LENGTH = 15'

BOND CAPACITY = 4 k/ft x 15 ft = 60 kips

Fs = Bond Capacity / Bond Demand = 0.66 kips/ft Ok

USE AASHTO MIN BOND LENGTH

BAR DESIGN

USE WILLIAMS 1" 156 ksi ALL THREAD BAR (SEE SHEET 91)

ALLOWABLE TENSILE STRESSES

At design load F_t < 0.6 F_u

At proof load F_t < 0.8 F_u

CHECK 1

0.6 * 156 = 93.6 > 511.1; OK

CHECK 2

0.8 (156) = 124.9 > 63; OK

1.53 (511)

OK USE WILLIAMS 1" 156 KSI ALL THREAD BAR OR APPROVED EQUAL
EARTH PRESSURES (LEVEL SECTIONS)

\[ \gamma = 120 \text{ VEP} \quad \gamma_1 = 15' \]
\[ \gamma = 38' \quad \gamma_2 = 9.2' \]

(1) TRANSFER SURCHARGE LOAD ADDED FOR COUPLING
LOAD (CALCULATED \( G = 1 \))

PASSIVE PRESSURE RESISTED DUE TO:

POSITIVE: SEISMIC EFFECTIVE

LATERAL EARTH PRESSURES

\[ p = 0.65 \times \gamma_1 \times D \times (AASHTO E16-67, 7A) \]

\[ h_1 = 0.8 \times \text{SCE} = 0.8 \times 13' \]

ASH = 3 1/2" VEP

\[ q_{\text{TOTAL}} = 378 + 72 = 450 \text{ VEP} \]

\[ y_{S\text{AV}} = q_{\text{TOTAL}} \times \text{HEIGHT} \times \text{SPACING} \]

400 (15") (5") = 9.02 kips

LATERAL SPACING

MIN SPACING:

\[ T_{\text{LS}} = 400 \times 5" \times 7.5' \times (1.5) = 67.5 \times 500" \]

\[ 6' \times 6' \times 7.5' \]

\[ 7,500" \]
F.S. OVERTURN LEVEL SECTIONS

NORTH ROOM

MOMENT OVERTURN

\[ M = 30K \times 12 = 360 \text{ k-ft} \]

RIGHTING MOMENT

\[ M = 30K \times 12 = 360 \text{ k-ft} \]

FOR T.L. = 725K \times 14.67 = 13300 \text{ k-ft}

FOR I.S. = 725K \times 0.467 = 218 \text{ k-ft}

\[ \phi = \frac{540}{360} = 1.5 \text{ k-ft} \]

\[ F_s = \frac{M_{OVERTURN}}{F_i} = \frac{360}{218} = 1.65 \text{ k-ft} \]

SOUTH ROOM

SEE NORTH ROOM CALC

F.S. = 1.54

F.S. SLIDE LEVEL SECTIONS (NORTH + SOUTH)

\[ F_s = \frac{F_s}{F_i} = \frac{72.5}{30} = 2.43 \]

\[ 72.5 \quad 1 \quad 90K \quad \text{SOUTH ROOM} \]

\[ 22.5 \quad 1 \quad 8.83 \]

\[ 5 \quad 1 \quad 1 \]

RESISTING FORCE

\[ = 72.5K + 72.5K = 145K \]

\[ 14 \text{ k-ft} \]

DRIVING FORCE

\[ = 30 \text{ k-ft} \]

\[ F_s = \frac{145K}{30K} = 4.83 \text{ k-ft} \]
CHECK: WALL FOOTING BEARING CAPACITY / DEMAND (LEVEL 3)

WALL AXIAL LOAD (WORST CASE)

WALL DEAD LOAD

\[ = 150 \text{ VFE} \times (14.1 \text{ ft}) \times (9 \text{ ft}) = 19426 \text{ VFE} \]

FOOTING DEAD LOAD

\[ = 150 \text{ VFE} \times (2 \text{ ft}) \times (3.5) = 900 \text{ VFE} \]

FLOOR TRIB DEAD LOAD

\[ = 150 \text{ VFE} \times (1.5 \text{ ft}) \times (10.6 \text{ ft}) = 2305 \text{ VFE} \]

\[ \text{BEAMS:} \]

\[ 4 \times (18 \text{ in.}) (7) \times (150 \text{ VFE}) \times (10 \text{ ft.}) = 3725 \text{ #} \]

\[ B = 14.95 \text{ in.} \times 30 \text{ in.} = 500 \text{ VPF} \]

\[ L = \text{ WALL LENGTH} \]

\[ \varepsilon = 5025 \text{ VPF} \]

VERTICAL COMPONENT OF TIEBACK (WORST CASE)

\[ T_V = T_{TV} + T_{BV} \]

\[ T_{4.5V} = \tan 26^\circ (22.5 \text{ k}) = 8.2 \text{ KIPS} \]

\[ T_V = 8.2 \text{ (2)} = 16.4 \text{ KIPS ft} = 3.3 \text{ KLF} \]

\[ T_V \times \text{ WPL} = 8 + 3.3 = 8.3 \]

\[ 8.3 \div 2 = 4.15 \text{ KSF} \]

SOIL BEARING CAPACITY \[ = 150 \text{ KSF} \] (ASSUME MINIMUM DEPTH)

\[ 4.15 \div 10 \text{ KSF} = 0.41 \text{ KSF} \]

\[ 0.41 > 1.0 \text{ KSF} \]
Tieback Design

(TL) TIE-S HORIZ = 22.5 k

TIE-S VERT = 8.7 k

TIE S = 24 k

Lockoff Load = 1.8 0% = 19.2 k (CALTRANS 9-14)

Pullout Capacity

Ultimate Bond Stress = 4 k/ft (AASHTO TBL 5.7.6.28)

For Hard Clay (Must be Field Verified)

For Tieback Length = 30 ft

Min Unbonded Length = 15 (AASHTO Fig. 5.7.1A)

Min Bonded Length = 15

Bond Capacity = 4 k/ft (15') = 60 kips

\[
\frac{Fe}{Bond\ Demand} = \frac{60\ kips}{24\ kips} = 2.5 < 1\ OK
\]

Use AASHTO Min Bond Length

Bar Design

Use Williams 1" 15 ksi all tie end bar (See Sheet 9.1)

T < 24 K < 50 k < OK (See Sheet 10)

Use Williams and 1" 50 ksi all thread bar or approved equal.
**EXHIBIT IN**

- TY = 30 FT
- TS = 30 FT
- TG = 45 FT
- TT = 30 FT

**TIEBACK LENGTHS**

**EXHIBIT 2**

- T4 = 22.5 K
- TS = 22.5 K
- TG = 41 K (FROM RISA NODE 1807)
- TT = 31 K (FROM RISA NODE 1953)

**CONCLUSIONS**

- TG - TT < 48 K ⇒ OK (SEE SHEET 10 FOR TIE-BACK CAPACITIES)

**LOCK OFFLOAD FOR T4 TIEBACK**

\[ \frac{41 K}{\cos 20^\circ} \cdot 8 = 35 K \]

**LOCK OFFLOAD FOR TT TIEBACK**

\[ \frac{31 K}{\cos 20^\circ} \cdot 8 = 26 K \]
NORTH ELEVATION

CONJUNCTIONS FOR TIEBACKS SURROUNDING FLOOR TUNNEL

TIEBACK LENGTHS
- T4 = 30 ft
- T5 = 30 ft
- T8 = 35 ft
- T9 = 50 ft
- T10 = 45 ft

CONJUNCTIONS
- T4 = 22.5 k (SHEET 14)
- T5 = 22.5 k
- T8 = 41 k (FROM RIGA NODE 1299)
- T9 = 31 k (FROM RIGA NODE 1445)

CONJUNCTIONS AT T8 - T9 < 48 k Max OK SEE SHEET 10 FOR TIEBACK CAPACITIES

LOCK-OFF LOAD FOR T8 TIEBACK
- 35 k (SEE SHEET 10A)

LOCK-OFF LOAD FOR T9 TIEBACK
- 26 k (SEE SHEET 10A)

T6, T8, T10 TIEBACKS EXTENDED TO 35 - 45 FEET RESPECTIVELY

1/4 1/4 1/4 1/4

To ensure at least 15 ft of bonded length when floor tunnel is removed.

Because the min. bond length will be 15 ft or greater, a separate capacity has not been calculated. This is conservative.
Move tieback down 2 feet to audio hitting exterior manhole. The manhole is approx. 5 ft. deep and located 9 ft. from the proposed retaining wall.

Wall check (from right plates 277, 278, 341, 442):

Max moment X (mx + mx')

\[ = -3.35 \text{ k-ft} \]

Max moment Y (my + my')

\[ = -5.48 \text{ k-ft} \]

Mallow = 0.1 k-ft (Sheet 6)

\[ \frac{M}{C} = \frac{5.48}{0.1} = 54.8 < 71 \text{ is OK} \]

Max shear X (plate 342)

\[ = 8.84 \text{ k-ft} \]

Max shear Y (plate 278)

\[ = 9.5 \text{ k-ft} \]

Shear allow (one way)

\[ = 13.7 \text{ k-ft} > 9.5 \text{ is OK} \]

Max punching shear

\[ 1140 \times 1.5 \times 5.5 \times 5 \text{ ft} = 47 \text{ k-cu ft} : \text{ use 4B (Sheet 37)} \]

Punching shear OK; see sheet 5.

It is OK to move (1) tieback down (2) feet to audio exterior manhole.
MOVE (2) SOUTHERNMOST "T1" TIEBACKS DOWN 2 FEET TO AVOID 2.5" DEEP BEAM.

WALL CHECK (FROM G4A PLATES 249, 313, 377, 441)

\[
\text{MAX MOMENT } x (Mx + Mx'y') = -3.16 \text{ k-ft/ft}
\]

\[
\text{MAX MOMENT } y (My + Mx'y') = -5.412 \text{ k-ft/ft}
\]

\[
M\text{ALLOW} = 6.1 \text{ k-ft (SHEET 4)}
\]

\[
D/C = 5.412 / 6.1 = 0.89 < 1 \therefore OK
\]

\[
\text{MAX SHEAR } x \text{ (PLATE 315)} = 10.38 \text{ k/ft}
\]

\[
\text{MAX SHEAR } y \text{ (PLATE 315)} = 11.21 \text{ k/ft}
\]

\[
\text{SHEAR ALLOW (ONE WAY)} = 15.7 \text{ k} > 11.21 \therefore \text{OK}
\]

\[
\text{MAX PUNCHING SHEAR}
\]

\[
= 1140 \text{ psi} \times (5.5 \text{ ft}) \times (5 \text{ ft}) = 97 \times 4.48 \text{ k} \therefore \text{USE 4B (SHEET 3)}
\]

\[
\therefore \text{OK TO MOVE (2) TIEBACKS DOWN (2) FEET TO AVOID 2.5'}\text{ BEAM}
\]
LOG - SPIRAL FAILURE SURFACE

NOTE: R is not a safety factor.

REDUCTION FACTOR (R) OF $k_p$
FOR VARIOUS RATIOS OF $R_i / \phi$:

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Active and passive coefficients with wall friction (sloping backfill)

FROM USS STEEL SHEET PILING DESIGN MANUAL

FIGURE 8

4-10
The graph shown above was taken from the original Bevatron Foundation investigation performed in 1948.
5.7.4 Seismic Pressure

Refer to Section 6 of Division I-A—Seismic Design for guidance regarding the design of anchored retaining walls subjected to dynamic and seismic loads. In general, the pseudo-static approach developed by Mononobe and Okabe may be used to estimate the equivalent static forces provided the maximum lateral earth pressure be computed using a seismic coefficient $k_s = 1.5A$. Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure.

5.7.5 Structure Dimensions and External Stability

The design of anchored walls includes determination of the following:

- Size, spacing, and depth of embedment of vertical wall elements and facing;
- Type, capacity, spacing, depth, inclination and corrosion protection of anchors; and
- Structural capacity and stability of the wall, wall foundation, and surrounding soil mass for all intermediate and final stages of construction.
*Cartwright Engineers, 2010b,* "Bevatron Field Order #2, Retaining Wall, Berkeley, California," structural plan sheets S101, S201, S202 and S301 dated April 23, 2010, Cartwright Project # 108096.2
# BEVATRON CAISSON LOCATIONS

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<tr>
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<td>283</td>
<td>701.44</td>
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<td>1383.64</td>
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**REVIEWS**

- Date: 2016-01-01
- Description: Review

**ENGINEERS**

- Lawrence Berkeley National Laboratory
- CALIFORNIA

**PROJECT #** 11109

**SCALE:** NOTED

**DATE:** FEBRUARY 2012

**APPROVALS**

- DRAWN BY: [Signature]
- CHECKED BY: [Signature]

**CO01**

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1. This drawing shows the location of the caissons that were excavated by excavation during construction of the building. Many of these caissons are shown on sheet 56, which is the footing and foundation plan for building 51 and was presented to be used by UCB.

2. There were many caissons that were found and located that were not shown on any of the as-built drawings that were presented to be used by UCB. Therefore we assume that there may be additional caissons present that were not found and located.

3. During the process of the building plan and survey research, various utility excavation general caissons that are shown on this drawing may have been removed or cut down to a lower elevation.

4. Representation of new caissons constructed further locates the locations of the caissons, these marks were then surveyed by foot and correcting the drawing is the process of this survey.
BASIS OF BEARING 49°12' 98°49'50"E

5017 BRASS CAP MONUMENT N: 998.17' E: 1805.02' Z: 708.54

5011 BRASS CAP MONUMENT N: 947.61' E: 1403.07' Z: 673.51

1. THIS DRAWING SHOWS THE LOCATION OF THE CASINGS THAT WERE UNCOVERED BY EXCAVATION DURING DEMOSSION OF BUILDING S. MANY OF THESE CASINGS ARE SHOWN ON DETAIL 31, WHICH IS THE FOUNDATION PLAN FOR BUILDING S AND WAS PROVIDED TO US BY L&L.

2. THERE WERE MANY CASINGS THAT WERE FOUND AND LOCATED THAT WERE NOT SHOWN ON ANY OF THE AS BUILT DRAWINGS THAT WERE PROVIDED TO US. IT IS POSSIBLE THAT THERE ARE ADDITIONAL CASINGS PRESENT THAT WERE NOT FOUND AND LOCATED.

3. DURING THE PROCESS OF THE BUILDING SLAB AND TUNNEL REMOVAL AND UTILITY EXCAVATION SEVERAL CASINGS THAT ARE SHOWN ON THIS DRAWING MAY HAVE BEEN REMOVED OR CUT DOWN TO A LOWER LEVEL.

4. REPRESENTATIVES FROM CLASSIC CONSTRUCTION MARKED THE LOCATIONS OF THE CASINGS. THERE WERE MORE THEN SURVEYED BY FORMENT SURVEYING. THIS DRAWING IS THE PRODUCT OF THAT SURVEY.
1. This drawing shows the location of the casings that were uncovered by excavation during demolition of building C1. Many of these casings were shown on sheet E2, which is the footing and foundation plan for building E2 and were positioned to US by US.

2. There were many casings that were found and located that were not shown on any of the as-built drawings that were provided to us by US. Therefore, we assume that there may be additional casings present that were not found and located.

3. During the process of the building slab and trench excavation and utility excavation several casings that are shown on this drawing may have been removed or cut down to a lower elevation.

4. Representatives from Niles Construction marked the locations of the casings. These marks were then surveyed by footprint engineering. This drawing is the product of that survey.
1. This drawing shows the location of the caissons that were uncovered by excavation during demolition of Building 51. Many of these are in the area west of Building 51 and are shown in the drainage and foundation plan for Building 51 and were provided to us by CAL.

2. There were many caissons that were found and located that were not shown on any of the existing drawings that were provided to us by CAL. Furthermore, we assume that there may be additional caissons present that were not found and located.

3. During the process of the building plan and elevation revision, mechanical and utility location sketches, caissons that are shown on this drawing may have been removed or cut down to a lower elevation.

4. Representatives from Cal/Proration marked the locations of the caissons. These marks were then surveyed by foreговор [sic], this drawing is the product of that survey.
1. This drawing shows the location of the caissons that were discovered during excavation of building #1. Many of these caissons are shown on sheet #2, which is the footing and foundation plan for building #1 and was provided to us by law.

2. There were many caissons that were found and located that were not shown on any of the as-built drawings that were provided to us by law. Therefore, we assume that there may be additional caissons present that were not found and located.

3. During the process of the building slab and trench design and utility excavation several caissons that are shown on this drawing may have been removed or cut down to a lower elevation.

4. Representatives from Blaisdell Construction marked the locations of the caissons. These marks were then surveyed by independent surveyor. This drawing is the product of that survey.