APPENDIX I
Analysis Results
This Appendix contains a summary of the selected analyses results.

- Sensitivity Analysis
- FRP (Fiber Reinforce Polymer) Methodology Hand Calculation & Excel verification
- Arch Rib Analysis
- Vertical Hanger Analysis
- Tie Girder Analysis
- Portal Bracing Analysis
- Substructure Analysis miscellaneous
- Expected Existing Material Properties
- Elevation Shift at Rumsey (NGVD29 to NAVD88)
\[ F_c = 0.85 \times f_c' \times a \times b \quad \text{[kips]} \]

\[ a = 0.85 \quad \text{if} \quad f_c' < 4200 \]

\[ 0.85 \times f_c' \]

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

\[ F_s = A_s \times f_y \]

\[ f_c = E_c \times \varepsilon_c \]

\[ f_{fc} = E_c \times \varepsilon_c \]

\[ f_{fs} = E_s \times \varepsilon_{sf} \]

\[ \Delta = E_c \varepsilon_c \]

\[ \frac{E_c}{c} = \frac{E_s}{h - c} = \frac{E_{fr}}{h - c} \]

1st Assume steel yield,

\[ \varepsilon_c = 0.003 \]

2nd Find \( c \)

\[ \varepsilon_{fr} = \frac{(h - c)(\varepsilon_c)}{c} \]
**QUINCY ENGINEERING, INC.**

**Project**

**Job No.**

**By**

**Date**

**Description** FRP

**Arch Rib  Strong Axis**

**FRP**

- **Fy** = 40 ksi
- **E_{tens}** = 1.9 x 10^6 ksi
- **E_{break}** = 0.85%
- **f_{cu}** = 12 ksi

**Main bars**

- #9, T = 6
- **A_s** = 1 in^2 × 0.01 = 6 in^2
- **d_s** = 1.128 in

**Shear bars**

- #3
- **Ar** = 0.11 in^2
- **dv** = 0.075 in

**Clr** = 1.5 in assumed

**Step 1**: Assume **E_s** ≥ **Fy** in steel yield

\[
F_s = A_s \times f_y = (6 \text{ in}^2) \times (40 \text{ ksi}) = 240 \text{ kips}
\]

**Step 2**: **C_e** = **T_s** + **T_f**

Compression of conc = tension of steel + FRP

\[
C_e = T_s + T_f = 0.85 f_y e = A_s \times f_y + A_{frp} \times \frac{E_f}{E_t} f_y
\]

\[
(0.85)(2,500 \text{ ksi})(0.85)(27''') = (6\text{ in}^2)(40 \text{ ksi}) + (0.04 \times 2\times 2\text{ in}^2) \times \frac{E_t}{E_f} \times \frac{n_c}{n_c} (E_c) \times E_t
\]

but \( a = \beta \cdot C \)

\[
a = 0.85 \times C
\]

\[
(0.85)(2,500 \text{ ksi})(0.85)(27''') = (6\text{ in}^2)(40 \text{ ksi}) + \]

\[
+ (0.04 \times 2\times 2\text{ in}^2)(36'' - c)(0.005\times 19^2 \text{ ksi})
\]

\[
\rightarrow c = 9.4'' \quad \text{or} \quad 6''
\]

\[
\rightarrow a = (0.85)(9.4'') = 8''
\]
**Step 3:** Check if $E_s \geq E_Y$, if steel yields

$$\frac{E_s}{d-c} = \frac{E_Y}{C}$$

$$d = h - \frac{h}{2} - \frac{C}{2}$$

$$= 36'' - 12'' - \frac{9.4''}{2} (11.86'')$$

$$= 33.56''$$

$$\frac{E_s}{33.56'' - 9.4''} = \frac{0.003}{9.4''}$$

$$\Rightarrow E_s = 0.011$$

$$f_y = E_s \cdot E_Y$$

$$40 ksi = 29,000 ksi \times E_Y$$

$$\Rightarrow E_Y = 0.0014$$

$$E_s > E_Y \Rightarrow$$ steel does yield

**Step 4:** Calculate $E_{fe}$ and check limits of FRP

$$\frac{E_{fe}}{h-c} = \frac{E_s}{C}$$

$$\frac{E_{fe}}{36'' - 9.4''} = \frac{0.003}{9.4''}$$

$$\Rightarrow E_{fe} = 0.0085 \leq 0.85\%$$

**Design Elongation:** Off break

$$f_{frp} = E_{frp} \cdot E_t$$

$$= (0.0085)(1.19 \times 10^4 ksi) = 101 ksi < 12 ksi$$

OK
Step 5: Calculate moment strength

\[ M_n = \frac{[0.85 \times f'_c \times a \times b]}{457 \text{ kips}} \left[ C - \frac{a}{b} \right] \]

\[ + \left[ A_s \times f_y \right] \left[ d - c \right] \]

\[ + \left[ A_{fep} \times E_{fep} \times E_f \right] \left[ h - c \right] \]

\[ M_n = \frac{(0.85)(2.5 \text{ ksi})(8")(27")}{210 \text{ kips}} \left(9.4" - \frac{8"}{2}\right) \]

\[ + \frac{(6\text{ in}^2)(40 \text{ ksi})}{2.18 \text{ kips}} \left(35.6" - 9.4"\right) \]

\[ + \frac{(0.04\text{ in})(2.6\text{ kips})(2.7\text{ in})}{101 \text{ ksi}} \text{ (0.0085)(119 \times 10^4 \text{ ksi})(36" - 9.4")} \]

\[ M_n = 2478 \text{ kips} \]

\[ + 5808 \text{ kips} \]

\[ + 5811 \text{ kips} \]

\[ 14,097 \text{ kips} \]

\[ = 1175 \text{ kft} \]

Step 6: Calculate moment factor

\[ \phi M_n \]

\[ (0.9)(1175 \text{ kft}) \]

\[ = 1057 \text{ kft} \]
### Concrete Section Input:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall, h [in]</td>
<td>36.00</td>
</tr>
<tr>
<td>Overall, d [in]</td>
<td>33.56</td>
</tr>
<tr>
<td>Conc, f_c [ksi]</td>
<td>0.003</td>
</tr>
<tr>
<td>PS (yes, no)</td>
<td>no</td>
</tr>
<tr>
<td>phi_FS</td>
<td>0.90</td>
</tr>
<tr>
<td>phi_non-ps</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>9.40</td>
</tr>
</tbody>
</table>

### Steel Reinforcement Input:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>fy [ksi]</td>
<td>40</td>
</tr>
<tr>
<td>Main Bars</td>
<td># 9</td>
</tr>
<tr>
<td>d_s [in]</td>
<td>1.128</td>
</tr>
<tr>
<td>As [in^2]</td>
<td>6</td>
</tr>
<tr>
<td>A_s [in^2]</td>
<td>1</td>
</tr>
<tr>
<td>Photos shows 7 bars</td>
<td></td>
</tr>
<tr>
<td>d_v [in]</td>
<td>0.375</td>
</tr>
<tr>
<td>Shear Bar #3</td>
<td></td>
</tr>
<tr>
<td>d_v [in]</td>
<td>0.375</td>
</tr>
<tr>
<td>clr [in]</td>
<td>1.5</td>
</tr>
</tbody>
</table>

### Manufacture FRP Input:

<table>
<thead>
<tr>
<th>Design limits:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ulf Tensile</td>
</tr>
<tr>
<td>Stress in steel</td>
</tr>
<tr>
<td>under service</td>
</tr>
<tr>
<td>load should be</td>
</tr>
<tr>
<td>limited to</td>
</tr>
<tr>
<td>80% of yield</td>
</tr>
<tr>
<td>strength</td>
</tr>
</tbody>
</table>

### Analysis:

- **Forces:**
  \[ F_{\text{Conc}} = 0.85 f_c a b \]
  \[ F_{\text{steel}} = A_s f_y \]
  \[ F_{\text{frp}} = A_{\text{frp}} f_{fe} \]
  \[ F_{\text{total}} = F_{\text{Conc}} + F_{\text{steel}} + F_{\text{frp}} \]

- **Strains:**
  \[ \varepsilon_c = 0.003 \]
  \[ \varepsilon_s = 0.0077 \]
  \[ \varepsilon_{fe} = 0.0085 \]
  \[ \varepsilon_{\text{frp}} = 0.0085 \]

- **Neutral Axis:**
  \[ c = 9.40 \text{ in} \]
  \[ \beta = 0.85 \]
  \[ a = 8.0 \text{ in} \]

- **Moment Arm:**
  \[ \text{Conc Arm} = (c - a / 2) \]
  \[ \text{Steel Arm} = (d - c) \]
  \[ \text{Fiber Arm} = (h - c) \]

- **Nominal Moment:**
  \[ \text{Conc Contr} = 2.476 \text{ kip-in} \]
  \[ \text{Steel Contr} = 5.799 \text{ kip-in} \]
  \[ \text{Fiber Contr} = 5.807 \text{ kip-in} \]

- **Factored Moment Strength:**
  \[ \phi \text{ Mn [k-in]} = 12.675 \]
  \[ \phi \text{ Mn [k-ft]} = 1.056 \]
Sensitivity Analysis

Several sensitivity analyses were performed to assess the necessity of obtaining a concrete strength core to determine a refined concrete strength of the existing Rumsey Bridge. Based on the sensitivity analyses results in the following pages, **QEI does not recommend obtaining sampling at this planning phase.** For more discussion, please see Section 2’s Similar Structure (Stevenson Bridge) study in this Feasibility Study.

Original / Baseline Model:

- Existing concrete strength: 2500 psi
- Existing bar reinforcing steel strength: 40 ksi
- Governing elastic D/C ratios:

![Diagram with color coding]
Sensitivity Model:

Sensitivity Modeling **Force** and Displacement D/C Results are approximately the same—as anticipated. Changing Concrete strength will have more impact on shear strength; however, most D/C ratios are governed by axial and flexural capacities. Therefore, **globally, the retrofit strategy is not affected by the concrete strength. During final design the concrete strength will affect the choice of the number of FRP layers required for a given member.**

Concrete strength: 2000 psi Model
Governing elastic D/C ratios:

Concrete strength: 1000 psi Model
Governing elastic D/C ratios:
Concrete strength: 4000 psi Model
Governing elastic D/C ratios:
Arch Rib Analysis:

As-built details of Arch sections & reinforcement:
Analysis below shows the limits where the Arch Rib exceed D/C ratio of 1.0 in Red:

Red block below indicates the approximate location of where the Arch will be retrofitted.
Moment diagram below indicates the weak access moments on the Arch Ribs.

Moment diagram below indicates the Strong access moments on the Arch Ribs.
D [in] = 36.00 in  3.00 ft

FC [ksi] = 2.5 ksi

Main Bars

# 9

Tot = 6

Main Reinforcement

phi_non-ps = 1.00

A_s [in^2] = 1

d_s [in] = 1.128

As [in^2] = 6

Shear Bars

# 3

A_v [in^2] = 0.11

d_v [in] = 0.375

Shear Confinement

clr [in] = 1.5

Analysis:

F_steel = As*fy = 240 kip

F_Conc = 0.85*fc*a*b 240 kip

a [in] = 4.18 in

beta = 0.85  (AASHTO 5.7.2.2)

x [in] = a/beta = 4.92 in

Check Steel Yield

e_s = 0.0175 > e_y = 0.0021

steel yields

Arm [in] = 31.47 in

\[ \phi \text{ Mn [k-in]} = 7,553 \text{ kip-in} \]

> Demand = 5,709  ok  D/C = 0.76

\[ \phi \text{ Mn [k-ft]} = 629 \text{ kip-ft} \]

> Demand = 476 kip-ft  ok
D [in] = 27.00 in  2.25 ft  
D [in] = 24.56 in  2.05 ft  
b [in] = 36.00 in  3.00 ft  
fc [ksi] = 2.5 ksi  
Fy [ksi] = 40 ksi  

Main Bars  
# 9  
Tot = 2  

PS [yes, no] = no  
phi_PS = 1.00  
phi_non-ps = 1.00  
ϕ = 1  

A_s [in^2] = 1  
d_s [in] = 1.128  
Main Reinforcement  
As [in^2] = 2  

A_v [in^2] = 0.11  
d_v [in] = 0.375  
Shear Confinement  
clr [in] = 1.5  

Analysis:  
F_steel = As*fy = 80 kip  
F_Conc = 0.85*fc*a*b  
80 kip  
a [in] = 1.05 in  
beta = 0.85  
x [in] = a/beta  
1.23 in  
(AASHTO 5.7.2.2)  

Check Steel Yield  
e_s = 0.0569  
e_y = 0.0021  
steel yields  

Arm [in] = 24.04 in  
ϕ Mn [k-in] = 1,923 kip-in  
ϕ Mn [k-ft] = 160 kip-ft  

Demand = 3,506  
NG  
D/C = 3.96  

Demand = 452 kip-ft  
NG
## Compression Member Strength

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_c'$</td>
<td>2500 psi</td>
<td>Main Bars #10 Main Reinforcement</td>
</tr>
<tr>
<td>$t$</td>
<td>36 in</td>
<td>Tot = 6</td>
</tr>
<tr>
<td>$b$</td>
<td>27 in</td>
<td>$A_s [in^2] = 1.27$</td>
</tr>
<tr>
<td>$d$</td>
<td>33.365 in</td>
<td>$d_s [in] = 1.27$</td>
</tr>
<tr>
<td>$d'$</td>
<td>2.635 in</td>
<td>$As [in^2] = 7.62$</td>
</tr>
<tr>
<td>$As = As'$</td>
<td>7.62 in$^2$</td>
<td>Shear Bars #3 Shear Confinement</td>
</tr>
<tr>
<td>$fy$</td>
<td>40,000 psi</td>
<td>Spacing [in] = 18</td>
</tr>
<tr>
<td>$\beta_1$</td>
<td>0.85</td>
<td>$A_v [in^2] = 0.11$</td>
</tr>
<tr>
<td>$Pu$</td>
<td>564 kip</td>
<td>Demand</td>
</tr>
<tr>
<td>$clr [in]$</td>
<td>2</td>
<td></td>
</tr>
</tbody>
</table>

### Pure Compression (LRFD 5.7.4.4)

$$Po = \phi [0.80 f_c' (tb - As - As') + (As + As') \cdot fy]$$

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$</td>
<td>0.85</td>
</tr>
<tr>
<td>$Po$</td>
<td>2,112,000 lbs</td>
</tr>
<tr>
<td>$\phi Po$</td>
<td>2,112 kips</td>
</tr>
</tbody>
</table>

### Lateral Reinforcement (BDS 8.18.2)

$$h_{wall} = 276 \text{ in}$$

$$h_c = 31.625 \text{ in}$$

$$Ag = 972 \text{ in}^2$$

$$Ac = 850 \text{ in}^2$$

$$0.30 s_f \cdot h_c' \cdot f_y \cdot (Ag/\text{Ac} - 1) = 1.54 \text{ in}^2$$

$$3.59 \text{ in}^2 > \text{Demand} = 1.54 \text{ in}^2 \text{ ok}$$
LRFD 5.8.3.3

Ecco Results:
\[ V_n = 48 \text{ kip} \quad \text{< Demand = 79 kip} \quad \text{NG} \]

\[ V_n = 0.25 f'c b v d_v + V_p \]

- \( f'c = 2,500 \text{ psi} \)
- \( b = 36 \text{ in} \)
- \( d_v = 24 \text{ in} \)
- \( v = 36 \text{ in} \)

\[ V = 48 \text{ kip} \quad < \quad \text{Demand = 79 kip} \quad \text{NG} \]

5.8.2.5 Minimum Transverse Reinforcement

\[ Av > 0.0316 \sqrt{f'c} \frac{b}{s} / fy \]

\[ Av = 0.01 \text{ in}^2 \quad \text{NG} \quad 0.81 \text{ in}^2 = 0.0316 \sqrt{f'c} \frac{b}{s} / fy \]

5.8.3.4.2 General Procedure

- \( Mu = 629 \text{ k-ft} \)
- \( Nu = 562 \text{ k} \)
- \( V_u = 78 \text{ k} \)
- \( V_p = 0 \text{ k} \)
- \( f_p = 0.7 f_{pu} \)
- \( Es = 29000 \text{ ksi} \)
- \( Ep = 29000 \text{ ksi} \)
- \( A_s = 6.00 \text{ in}^2 \)
- \( A_v = 0.01 \text{ in}^2 \)

\[ \text{Select: } \theta = 37 \text{ degrees} \quad \text{angle of inclination of diagonal comp. stress} \]

\[ Sxe = \frac{s x}{1.38} / (a g + 0.63) \]

- \( s x = \min \text{ of following} \)
- \( d_v = 24 \text{ in} \)
- \( d_v = 24 \text{ in} \)
- \( a g = 0.25 \text{ in} \)
- \( Sxe = 37.6 \)

If Min Transverse Reinforcement is met

\[ \beta = 4.8 / (1 + 750 e_s) \]

- \( \beta = 1.80 \quad \text{factor indicating ability of diagonal cracked concrete to transmit tension & shear} \)

If Min Transverse Reinforcement is NOT met

\[ \beta = 4.8 / (1 + 750 e_s) \times 51 / (39 + Sxe) \]

- \( \beta = 1.22 \quad \text{factor indicating ability of diagonal cracked concrete to transmit tension & shear} \)

Select: \( \beta = 1.2 \text{ degrees} \quad \text{factor indicating ability of diagonal cracked concrete to transmit tension & shear} \)

Select: \( \alpha = 90 \text{ degrees} \quad \text{angle of inclination of transverse reinforcemen} \)

\[ V_n = V_c + V_s + V_p \]

- \( V_c = 0.0316 \beta \sqrt{f'c} b v d_v \)
- \( V_c = 53 \text{ kip} \)
- \( V_s = Av f_y \sin(\alpha) < 0.095 \sqrt{f'c} b v d_v \)
- \( V_s = 0.44 \text{ kip} \)

- \( V_n = 52 \text{ kip} \quad \text{Gov} \)

- \( V_n = 0.25 f'c b v d_v + V_p \)
- \( V_n = 54 \text{ kip} \)

Select: \( \phi = 0.90 \quad \text{Seismic phi for shear} \)

Select: \( \phi [\text{kip}] = 48 \text{ kip} \quad < \quad \text{Demand = 79 kip} \quad \text{NG} \)
LRFD 5.8.3.3
Vn = minimum of the following 2 equations
\[ V_n = V_c + V_s + V_p \]
\[ V_n = 0.25f'c bdv + V_p \]
Ecco Results:
\[ \phi V_{\text{kip}} = 42 \text{ kip} > \text{ Demand} = 37 \text{ kip} \text{ ok} \]

b = 27 in
\[ d = 36 \text{ in} \]
Ac = 972 in²
f'c [ksi] = 2,500 psi
bv = 27 in
\[ dv = 33 \text{ in} \]

5.8.2.5 Minimum Transverse Reinforcement
\[ Av > 0.0316 \sqrt{f'c} b_s / fy \]
\[ Av = 0.01 \text{ in}^2 \]

5.8.3.4.2 General Procedure
\[ es = (Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo) / (Es As + Ep Aps) \]
Mu = 629 k-ft
Nu = 562 k
Vu = 78 k
Vp = 0 k

Aps = 0.00 in²
fpo = 175 ksi
\[ f_y = 40 \text{ ksi} \]
Ep = 29000 ksi
As = 6.00 in²
\[ d_s = 1.13 \text{ in} \]

Sxe = sx * 1.38 / (ag + 0.63)
x = 33 in
\[ dv = 36 \text{ in} \]
ag = 0.25 in
\[ Sxe = 51.8 \]

theta = 29 + 3500 es
\[ \theta = 37 \text{ degrees} \]

If Min Transverse Reinforcement is met
\[ \beta = 4.8 / (1 + 750 es) \]
\[ \beta = 1.83 \]

If Min Transverse Reinforcement is NOT met
\[ \beta = 4.8 / (1 + 750 es) \times 51 / (39 + Sxe) \]
\[ \beta = 1.05 \]

Select: \[ \beta = 1.0 \text{ degrees} \]
Select: \[ \beta = 90 \text{ degrees} \]

\[ V_n = V_c + V_s + V_p \]
\[ V_c = 0.0316\beta \sqrt{f'c} \text{ kip} bdv \]
Vc = 47 kip
Vs = Av fy sin(\alpha) < 0.996 \sqrt{f'c} \text{ kip} bdv
Vs = 0.44 kip
Vn = 47 kip

Select: \[ \phi V_{\text{kip}} = 42 \text{ kip} > \text{ Demand} = 37 \text{ kip} \text{ ok} \]
\[ \phi = 0.90 \] Seismic phi for shear
## Project Description

**Project:**

**Job No.:**

**BY:**

**DATE:** 5/12/2014

---

### Concrete Section Input:

- Overall, h [in] = 36.00 in 3.00 ft
- d [in] = 33.56 in 2.80 ft
- Overall, b [in] = 27.00 in 2.25 ft
- Concrete f'c [ksi] = 2.5 ksi
- Concrete \( c \) = 0.003 conc strain limit
- Concrete \( \varepsilon_c \) = 0.003
- Photos shows 7 bars
- Concrete strain limit

### Steel Reinforcement input:

- Main Bars # 9 conservative of 1 1/8" SQ bar
- Shear Bars # 3
- Total = 6 conservative base on per as-built.
- Steel f_y = 40 ksi
- Elongation at Break = 0.6%
- Steel yield \( \varepsilon_y \) = 0.0021

### Manufacture FRP input:

- Tensile Modulus = 1.19E+07 psi
- Design limits:
  - Overall, h [in] = 36.00 in 3.00 ft
  - d [in] = 33.56 in 2.80 ft
  - Overall, b [in] = 27.00 in 2.25 ft
  - Concrete f'c [ksi] = 2.5 ksi
  - Concrete \( c \) = 0.003 conc strain limit
  - Concrete \( \varepsilon_c \) = 0.003
  - Photos shows 7 bars
  - Concrete strain limit
- Steel Reinforcement input:
- Manufacture FRP input:
- Tensile Strength
  - 121 ksi
  - 4.8 kip/in
  - Tensile Modulus = 1.19E+07 psi
- Tensile Modulus = 1.19E+04
  - \( \phi_{ps} \) = 1.00
  - \( \phi_{non-ps} \) = 0.90
  - Shear Bars # 3
  - Total thickness

### FRP Analysis:

- \( F_{\text{Conc}} \) = 0.85*f'c*a*b
- \( F_{\text{steel}} \) = A*s*f_y
- \( F_f\) = A_f*f_fe
- \( F_f\) within allowable stress
- \( F_f\) within allowable strain
- Neutral Axis:
  - c [in] = 12.42 in calculated distance between N.A. to extreme conc fiber
  - 606 kip

### Analysis Existing (without FRP) - Strong Axis:

- Conc Arm = (c - a / 2)
- Steel Arm = (d - c)
- Fiber Arm = (h - c)
- Conc Arm = 4.92 in
- Steel Arm = 23.58 in
- Fiber Arm = 21.14 in
- Conc Contr = 4,328 kip-in
- Steel Contr = 5,073 kip-in
- Fiber Contr = 8,626 kip-in
- Nominal Moment = 18,027 kip-in

### Factored Moment Strengths:

- \( \# Mn_{[\text{kip-in}]} \) = 16,224 kip-in
- \( \# Mn_{[\text{kip-ft}]} \) = 2400 kip-ft

### Steel and FRP Analysis:

- F_Conc = 0.85*f'c*a*b
- F_steel = A*s*f_y
- F_frp = A_f*f_fe
- \( \beta \) = 0.85 (AASHTO 5.7.2.2) function of f'c
- \( \alpha \) = 10.6 in vertical distance of Whitney stress block

### Assumptions:

- Plain section remains plane
- Max compressive strain in concrete is 0.003
- Stress in steel under service load should be limited to 80% of yield strength

---

### FRP Analysis Diagrams:

- Reinforced Concrete Section
- Strain Distribution
- Stress Distribution (Non-linear Concrete Stress Distribution)
- Stress Distribution (Equivalent Frictional Concrete Stress Distribution)
## Concrete Section Input:

- **Overall, h [in]**: 27.00 in, 2.25 ft
- **Overall, b [in]**: 36.00 in, 3.00 ft
- **Concrete f'c [ksi]**: 2.5 ksi
- **Concrete d [in]**: 0.003 conc strain limit

## Steel Reinforcement Input:

- **Main Bars**: #9 conservative of 1 1/8" SQ bar
- **Shear Bars**: #3
- **Concr f'c [ksi]**: 2.5 ksi
- **Elongation at Break**: 0.6%

## Manufacture FRP Input:

- **Design limits**:
  - **Ult Tensile Strength**: 40 ksi
  - **Tensile Modulus**: 1.19E+07 psi
- **number of layers [ea]**: 0.04 in
- **FRP composite material thickness per layer**: 0.32 in
- **width of FRP reinforcing layers**: 36.00 in
- **RPF within allowable stress**: FRP
- **RPF within allowable strain**: steel yields
- **Concrete e_c = 0.003**:
- **Steel e_s = 0.0038**:
- **RFP e_fe = 0.0045**:
- **Yield e_y = 0.0021**:

## Analysis Existing (without FRP) - Weak Axis:

- **Conc Arm**:
  - **Conc Arm = (c - a / 2)**: 7.14 in
  - **Nominal Moment = 14,886 kip-in**
  - **Nominal Moment = 1,240 kip-ft**
- **Steel Arm**:
  - **Steel Arm = (d - c)**: 21.14 in
  - **Nominal Moment = 5,001 kip-in**
  - **Nominal Moment = 846 kip-in**
- **Fiber Arm**:
  - **Fiber Arm = (b - c)**: 14.58 in
  - **Fiber Arm = 9,039 kip-in**
  - **Fiber Arm = 1,116 kip-ft**

## FRP Analysis:

- **F_Conc = 0.85*fc*a*b**
  - **700 kip**
- **F_steel = As*fy**
  - **40 kip**
- **F_frp = A frp * f fe**
  - **620 kip**
- **A frp = n * t_v * w_f**
  - **11.52 in sq**

## Assumptions:

- Plan section remains plane
- Max compressive strain in concrete is 0.003
- Stress in steel under service load should be limited to 80% of yield strength
Vertical Hanger Analysis:
As-built details of Portal sections & reinforcement:
Analysis below shows the limits where the entire Hanger set exceed D/C ratio of 1.0 in Red:

Analysis below shows the deformation from seismic excitation:

Red block below indicates the approximate location of where the Hanger will be retrofitted.
Hanger modeling (iteration only):
Hanger Moment Curvature Strong Axis:

![Graph showing moment curvature and strain diagram with numerical values for concrete strain (0.0111), steel strain (0.03), and neutral axis (9.6423).]
**Concrete Section input:**

- Overall, h [in] = 20.00 in, 1.67 ft
- d [in] = 17.49 in, 1.46 ft
- Concr fc [ksi] = 2.5 ksi
- Concrete i_s = 0.003, conc strain limit

**Steel Reinforcement input:**

- Overall, b [in] = 15.00 in, 1.25 ft
- A_s [in^2] = 1.27
- Photos shows 7 bars

**Manufacture FRP input:**

- Design limits:
  - Ult Tensile Strength = 121 ksi
  - in Primary Direction = 121.000 psi
  - Elongation at Break = 8.0%
  - Tensile Modulus = 1.18E+07 psi

**FRP Analysis:**

- F_Conc = 0.85*fc*a*b
- F_steel = As*fy
- F_frp = A_{frp}*f_{fe}
- F_Conc = 251 kip
- F_steel = 102 kip
- F_frp = 149 kip

**Neutral Axis:**

- c [in] = 9.26 in, calculated distance between N.A. to extreme conc fiber
- a [in] = 3.19 in, vertical distance of Whitney stress block

**Moment Arm:**

- Conc Arm = (c - a / 2)
- Steel Arm = (d - c)
- Fiber Arm = (h - c)

**Nominal Moment:**

- Conc Contr = 1,335 kip-in
- Steel Contr = 837 kip-in
- Fiber Contr = 1,603 kip-in

**Factored Moment Strength:**

- # Mn [k-ip] = 3,397 kip-in
- # Mn [k-ft] = 283 kip-ft

**Analysis Existing (without FRP) - Strong Axis (Informational only):**

- F_Conc = 0.85*fc*a*b
- F_steel = As*fy
- a [in] = 3.19 in
- beta = 0.85

**Check Steel Yield:**

- Steel yields

- a_y = 0.010
- Arm [in] = 15.90 in

**Assumptions:**

- Plane section remains plane
- Max compressive strain in concrete is 0.003
- Stress in steel under service load should be limited to 80% of yield strength

---

**QUINCY ENGINEERING, INC.**

**Project:** Rumsey

**Job No.**

**BY** JC

**DATE** 5-12-2014

**Description** Fiber Reinforcement Polymer

**Hangar**

**Strong axis (Retrofitted)**
Analysis Shear Strength (with FRP) - Along Strong Axis:

\[ V_s, \text{rank [k]} = \left( A_p, F_{p, \text{eff}} (\sin(a) + \cos(a)) \right) / s_f \]

- Number of layers: 1
- FRP spacing: 12.00 in
- FRP effective depth: 17.75 in
- FRP efficiency factor: 0.85
- FRP effective stress: 15.2 ksi

- FRP spacing: 12.00 in
- FRP effective depth: 17.75 in

Analysis Axial Tensile Strength:

\[ V_s = 13 \text{ kip} \]

- Demand: 35 kip
- D/C: 0.60

- Demand: 110 kip
- D/C: 0.60
Project: Rumsey  
Job No.  
BY JC  
DATE 5/12/2014  

Concrete Section input:
- Overall, h (in) = 15.00 in  
- d (in) = 12.49 in  
- Concrete e = 0.003 conc strain limit
- PS [yes, no] = no  
- phi_PS = 1.00  
- phi_non-ps = 0.90  
- ϕ = 0.9  

Steel Reinforcement input:
- Main Bars # 12  
- Shear Bars # 3  
- d_s [in] = 0.37 in  
- d_v [in] = 0.04 in  
- e_fe = 0.0043  
- E服役 = 1.19E+07 psi  
- A_f = 2.4 in sq

Concrete:
- e_c = 0.003
- f'c = 2.5 ksi
- f = 0.9, if e_s ≥ 0.005
- f = interp if 0.002 ≤ e_s < 0.005
- f = 0.65, if otherwise

Assumptions:
- Plain section remains plane
- Max compressive strain in concrete is 0.003
- Stress in steel under service load should be limited to 80% of yield strength

FRP Analysis:
- F_Conc = 224 kip
- F_steel = 51 kip
- F_frp = 12.2 kip
- FRP within allowable stress

Strains:
- Concrete e = 0.003
- Steel e_s = 0.0031
- RFP e_fe = 0.0043
- Yield E服役 = 0.0021

Neutral Axis:
- c [in] = 5.19 in  
- a [in] = 5.3 in

Moment Arm:
- Conc Arm = (c - a / 2)  
- Steel Arm = (d - c)  
- Fiber Arm = (h - c)

Nominal Moment:
- Conc Contr = 1,190 kip-in
- Steel Contr = 418 kip-in
- Fiber Contr = 701 kip-in  

Analysis Existing (without FRP) - Weak Axis (informational only):
- F_Conc = 102 kip
- F_steel = 102 kip
- a [in] = 2.39 in
- β = 0.85
- a [in] = 2.81 in

Check Steel Yield
- e_y = 0.0021
- Arm [in] = 16.29 in

Analysis: Strong axis (Retrofitted):
- F_Conc = 0.85*fc*a*b
- F_steel = As*fy
- F_frp = A_f* f_fe
- F_Conc = F_steel + F_frp
- 0 =  - F_Conc  + F_steel + F_frp
- 0 kip

Design Limits:
- Un Tensile Strength = 121 kip
- Tensile Modulus = 1.19E+07 psi

Assumptions:
- FRP composite material thickness per layer
- Total thickness = 0.12 in
- width of FRP reinforcing layers

Analysis: Strong axis (Retrofitted):
- F_Conc = 0.85*fc*a*b
- F_steel = As*fy
- F_frp = A_f* f_fe
- F_Conc = F_steel + F_frp
- 0 =  - F_Conc  + F_steel + F_frp
- 0 kip

Design Limits:
- Un Tensile Strength = 121 kip
- Tensile Modulus = 1.19E+07 psi

Assumptions:
- FRP composite material thickness per layer
- Total thickness = 0.12 in
- width of FRP reinforcing layers
### Rebar Steel Properties

- **Main Bars**: 1.125 in^2
- **Main Reinforcement**: 1.27 in^2
- **Total**: 5

### Tension

\[
T_n = F_y \cdot A \\text{s}
\]

- **\(T_n\)**: 253 kips
- **\(0.5T_n\)**: 253 kips

### Demand

- **\(P_u\)**: 115 kips
- **\(D/C\)**: 0.45
LRFD 5.8.3.3

\[ V_n = \min (V_{c} + V_{s} + V_{p}) \]

\[ V_n = 0.25 f'c b v d_v + V_{p} \]

**Ecco Results:**

\[ V_n = 1 \text{ kip} \]

\[ D = \text{Demand} = 15 \text{ kip} \]

**NG**

- **b** = 15 in
- **d** = 20 in
- **Ac** = 300 in²
- **f_c** = 0 ksi (worst case)
- **bv** = 15 in
- **dv** = 17 in

**5.8.2.5 Minimum Transverse Reinforcement**

\[ A_v > 0.0316 \sqrt{f_c} b \]

\[ A_v = 0.02 \text{ in}^2 \]

**OK**

**0.00 in²**

\[ = 0.0316 \sqrt{f_c} b \]

**5.8.3.4.2 General Procedure**

\[ es = \frac{(Mu + 0.5 Nu + (Vu - Vp) \cdot A_{ps})}{(Es \cdot A_s + Ep \cdot A_{ps})} \]

- **Mu** = 32184 k-ft
- **Nu** = 653 k
- **Vu** = 441 k
- **Vp** = 0 k

**Aps** = 0.00 in²

**Area of PS**

**Flexural steel info:**

- **Es** = 29000 ksi
- **Ep** = 29000 ksi
- **As** = 8.89 in²
- **A_s-bar** = 1.27 in²
- **d_s-bar** = 1.27 in

- **Fy = 40 ksi**
- **fpo = 175 ksi**

**Sxe = sx * 1.38 / (ag + 0.63)**

\[ sx = \min (17 \text{ in}, 17 \text{ in}) \]

\[ d_v = \frac{36 \text{ in}}{17 \text{ in}} \]

\[ ag = \frac{0.25 \text{ in}}{\text{max agg size assumed per photos}} \]

**Sxe = 26.7**

\[ \theta = 29 + 3500 es \]

\[ \theta = 65 \text{ degrees} \]

\[ \theta = \text{angle of inclination of diagonal comp. stress} \]

**Select:**

- **beta = 0.4 degrees**
- **theta = 65 degrees**
- **Select:**

\[ V_n = \frac{V_c + V_s + V_p}{f'c \cdot b v d_v + V_{p}} \]

\[ V_c = 0.0316 \cdot \text{beta} \cdot \sqrt{f_c} b v d_v \]

\[ V_s = 0.66 \text{ kip} \]

\[ V_{p} = 0 \text{ kip} \]

\[ V_n = 1 \text{ kip} \]

**Gov**

\[ \phi = 0.90 \]

Seismic phi for shear

**Select:**

\[ \phi = 1 \text{ kip} \]

\[ \text{Demand} = 15 \text{ kip} \]

**NG**

\[ 15.00 \]
LRFD 5.8.3.3

\[ V_n = \text{min} \left( V_c + V_s + V_p \right) \]

Ecco Results:

\[ f_V = 1 \text{ kip} \]

\[ \text{Demand} = 30 \text{ kip} \]

NG

5.8.2.5 Minimum Transverse Reinforcement

\[ A_V > 0.0316 \sqrt{f'c} \frac{b}{s} / f_y \]

\[ A_V = 0.02 \text{ in}^2 \]

ok

5.8.3.4.2 General Procedure

\[ es = \left( \frac{M_u}{d_v} + 0.5 \frac{N_u}{V_u-V_p} \right) \div \left( E_s \frac{A_s}{E_p} \right) \]

\[ M_u = 13063 \text{ k-ft} \]

Moment demand

\[ N_u = 653 \text{ k} \]

Axial demand

\[ V_u = 263 \text{ k} \]

Shear demand

\[ V_p = 0 \text{ k} \]

Component in direction of applied shear of the effective prestressing force positive if resisting the applied shear

\[ A_{ps} = 0.00 \text{ in}^2 \]

Area of PS

\[ f_{po} = 175 \text{ ksi} \]

0.7 fpu

Flexure steel info:

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

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

Flex Bars

\[ A_s = 5.08 \text{ in}^2 \]

Flexural steel

\[ A_{s_bar} = 1.27 \text{ in}^2 \]

\[ d_{s_bar} = 1.27 \text{ in} \]

approx effective

\[ S_{xe} = s_x \times 1.38 / \left( \text{ag} + 0.63 \right) \]

\[ s_x = \text{min of following} \]

\[ 12 \text{ in} \]

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

\[ \text{dist between layers of long crack control reinf} \]

\[ \text{ag} = 0.25 \text{ in} \]

Max agg size assumed per photos

\[ S_{xe} = 18.8 \]

\[ \theta = 29 + 3500 es \]

69 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met

\[ \beta = 4.8 / \left( 1 + 750 \text{ cs} \right) \]

0.50 factor indicating ability of diagonal cracked concrete to transmit tension & shear

If Min Transverse Reinforcement is NOT met

\[ \beta = 4.8 / \left( 1 + 750 \text{ cs} \right) \times 51 / \left( 39 + S_{xe} \right) \]

0.45 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: \[ \beta = 0.5 \text{ degrees} \]

Factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: \[ \alpha = 90 \text{ degrees} \]

Angle of inclination of transverse reinforcement

\[ V_n = V_c + V_s + V_p \]

\[ V_c = 0.0316 \beta \sqrt{f'c} \frac{b}{s} \]

\[ V_c = 0 \text{ kip} \]

\[ V_s = A_v f_y \sin(\alpha) < 0.096 \sqrt{f'c} \frac{b}{s} \]

\[ V_s = 0.79 \text{ kip} \]

\[ V_n = 1 \text{ kip} \]

Gov

\[ \phi = 0.90 \]

Seismic phi for shear

Select: \[ \phi = 1 \text{ kip} \]

< Demand = 30 kip

NG

Select: \[ 30.00 \]
Tie Girder Retrofit:

As-built details of Girder-Tie sections & reinforcement:
Analysis below shows the limits where the Girder-Ties exceed D/C ratio of 1.0 in Red:

Red block below indicates the approximate location of where the Girder Ties will be retrofitted.
Screenshot below indicates that the retrofitted Girder-Ties all have D/C’s lower than 1.0.
Moment diagram below indicates the weak access moments on the Girder-Ties.

Moment diagram below indicates the Strong access moments on the Girder-Ties.

Photo of existing Girder-Ties.
**Analysis:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>D [in] =</td>
<td>63.00 in</td>
</tr>
<tr>
<td>d [in] =</td>
<td>60.44 in</td>
</tr>
<tr>
<td>b [in] =</td>
<td>23.00 in</td>
</tr>
<tr>
<td>f_c [ksi] =</td>
<td>2.5 ksi</td>
</tr>
<tr>
<td>F_y [ksi] =</td>
<td>40 ksi</td>
</tr>
<tr>
<td>A_s [in^2] =</td>
<td>1.27</td>
</tr>
<tr>
<td>d_s [in] =</td>
<td>1.125</td>
</tr>
<tr>
<td>A_v [in^2] =</td>
<td>0.2</td>
</tr>
<tr>
<td>d_v [in] =</td>
<td>0.5</td>
</tr>
<tr>
<td>clr [in] =</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Steel Yield

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>e_s =</td>
<td>0.0094</td>
</tr>
<tr>
<td>e_y =</td>
<td>0.0021</td>
</tr>
<tr>
<td>Arm [in] =</td>
<td>54.22 in</td>
</tr>
</tbody>
</table>

Moment

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mn [k-in] =</td>
<td>32,940 kip-in</td>
</tr>
<tr>
<td>Mn [k-ft] =</td>
<td>2,745 kip-ft</td>
</tr>
<tr>
<td>Demand =</td>
<td>31,025 kip-in</td>
</tr>
<tr>
<td>Demand =</td>
<td>2,585 kip-ft</td>
</tr>
<tr>
<td>D/C =</td>
<td>0.94</td>
</tr>
</tbody>
</table>
D [in] = 23.00 in  1.92 ft
D [in] = 20.44 in  1.70 ft
b [in] = 63.00 in  5.25 ft
Fc [ksi] = 2.5 ksi

Fy [ksi] = 40 ksi

Main Bars = 1 1/8 SQ
Tot = 11.9

Main Reinforcement

A_s [in^2] = 1.27
D_s [in] = 1.125
As [in^2] = 2.53125

PS [yes, no] = no

phi_PS = 1.00
phi_non-ps = 0.90

phi = 1.00 for seismic

Shear Bars = # 4

A_v [in^2] = 0.2
D_v [in] = 0.5

Shear Confinement

clr [in] = 1.5

Analysis:

F_steel = As*fy = 101 kip
F_Conc = 0.85*fc*a*b = 101 kip

a [in] = 0.76 in
beta = 0.85
x [in] = a/beta = 0.89 in

(AASHTO 5.7.2.2)

Check Steel Yield

Check = 0.0659 > e_y = 0.0021

steel yields

Arm [in] = 20.06 in

phi Mn [k-in] = 2,031 kip-in  <  Demand = 3,229 kip-in  NG  D/C = 1.59

phi Mn [k-ft] = 169 kip-ft  <  Demand = 269 kip-ft  NG
Concrete Section input:

- Overall, h [in] = 63.00 in 5.25 ft
- Overall, d [in] = 60.44 in 5.04 ft
- Concr f'c [ksi] = 2.5 ksi
- Concrete $\epsilon_c = 0.003$ conc strain limit

Steel Reinforcement input:

- Main Bars 1 1/8 SQ
- Photos shows 7 bars
- $d_s = 1.125$ Main Reinforcement
- $d_v = 0.5$ Shear Confinement
- $\phi = 0$ number of layers of FRP reinforcement

FRP Analysis:

\[
\begin{align*}
F_{\text{frc}} &= 0.85f'c*a*b \\
F_{\text{steel}} &= A_s*f_y \\
F_{\text{frp}} &= A_{frp}*f_{fe} \leq 0 \text{ kip}
\end{align*}
\]

- $\epsilon_{fe} = 0.0240$ RFP design longation limit exceeded!!!
- $\epsilon_{y} = 0.0021$ steel yields

Assumptions:

- Plain section remains plane
- Max compressive strain in concrete is 0.003
- Stress in steel under service load should be limited to 80% of yield strength

Nominal Moment:

- Conc Contr = 2,448 kip-in
- Steel Contr = 32,459 kip-in
- Fiber Contr = 0 kip-in

Analysis Existing (without FRP) - Strong Axis:

\[
\begin{align*}
F_{\text{frc}} &= 0.85f'c*a*b \\
F_{\text{steel}} &= A_s*f_y \\
F_{\text{frp}} &= A_{frp}*f_{fe} \leq 0 \text{ kip}
\end{align*}
\]

- $\epsilon_{fe} = 0.0229 > \epsilon_{y} = 0.0021$ steel yields
- $\epsilon_{c} = 0.003$ conc strain limit

Factored Moment Strength:

- $M_n [\text{k-in}] = 31,416 \text{ kip-in}$
- $M_d [\text{k-in}] = 2,618 \text{ kip-ft}$

Nominal Moment:

- Conc Contr = 2,448 kip-in
- Steel Contr = 32,459 kip-in
- Fiber Contr = 0 kip-in

Factored Strength:

- $M_n [\text{k-in}] = 31,416 \text{ kip-in}$
- $M_d [\text{k-ft}] = 2,618 \text{ kip-ft}$

Analysis Existing (without FRP) - Strong Axis:

- $\phi_{Mn} [\text{k-in}] = 31,416 \text{ kip-in}$
- $\phi_{Md} [\text{k-ft}] = 2,618 \text{ kip-ft}$
Concrete Section input:
Overall, h [in] = 48.00 in  4.00 ft
Overall, b [in] = 63.00 in  5.25 ft
Concr f'c [ksi] = 2.5 ksi
Concrete c_v = 0.003 conc strain limit

Steel Reinforcement input:
Main Bars Tot = 1 1/8" SQ
A_s [in^2] = 1.27

Manufacture FRP input:
Design limits:
ULT Tensile Strength 121 ksi in Primary Direction = 121,000 psi
Elongation at Break = 4.8 kip/in

Concrete e_c = 0.003

- Design:
  - 2
  - 0.04 in FRP composite material thickness per layer
  - 63.00 in width of FRP reinforcing layers

Assumptions:
Plain section remains plane
Max compressive strain in concrete is 0.003
Stress in steel under service load should be limited to 80% of yield strength

FRP Analysis:

Forces:
F_Conc = 0.85*fc*a*b  F_steel = As*fy = 350 kip
1.853 kip  51 kip

Strains:
\( \varepsilon_c / c = \varepsilon_s / (d - c) = \varepsilon_{frp} / (h - c) \)

Concrete e_c = 0.003
Steel e_s = 0.0054
Yield e_y = 0.0021
RFP e_{frp} = 0.0058
RFP within allowable stress

Neutral Axis:
c [in] = 16.29 in calculated distance between N.A. to extreme conc fiber

\( F_{Conc} = F_{steel} + F_{frp} \)
0 = -F_{Conc} + F_{steel} + F_{frp}
-1,402 kip

\( \beta = 0.85 \) (AASHTO 5.7.2.2) function of f_c
\( a [in] = 13.8 in \) vertical distance of whitney stress block

Moment Arm:
Conc Arm = (c - a / 2)  4.03 in
Steel Arm = (d - c)  53.43 in
Fiber Arm = (h - c)  40.99 in

Nominal Moment:
Conc Contr = 7,467 kip-in  Steel Contr = 2,705 kip-in  Fiber Contr = 14,364 kip-in

Factored Moment Strength:
\( \phi Mn [k-in] = 22,981 \) kip-in  > Demand = 13,800 kip-in OK D/C = 0.62
\( \phi Mn [k-ft] = 1,840 \) kip-ft  > Demand = 1,150 kip-ft OK

Analysis Existing (without FRP) - Weak Axis:

F_Conc = 0.85*fc*a*b  F_steel = As*fy = 101 kip
101 kip  101 kip

\( a [in] = 0.76 \) in
\( \beta = 0.85 \) (AASHTO 5.7.2.2)

Check Steel Yield
\( \varepsilon_y = 0.0021 \) steel yields

Arm [in] = 60.06 in
\( \phi Mn [k-in] = 5,473 \) kip-in  < Demand = 13,800 NG D/C = 2.52
\( \phi Mn [k-ft] = 456 \) kip-ft  < Demand = 1,150 kip-ft NG
<table>
<thead>
<tr>
<th>Description</th>
<th>Girder Tie</th>
<th>Concrete Axial (LRFD)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project:</strong></td>
<td>Rumsey</td>
<td></td>
</tr>
<tr>
<td><strong>Job No.:</strong></td>
<td>Y01-500</td>
<td></td>
</tr>
<tr>
<td><strong>BY:</strong></td>
<td>JC</td>
<td></td>
</tr>
<tr>
<td><strong>DATE:</strong></td>
<td>5/12/2014</td>
<td>(LRFD 5.7.4)</td>
</tr>
</tbody>
</table>

## Compression Member Strength

- **fc'** = 2500 psi
- **t** = 63 in
- **b** = 23 in
- **d** = 60.365 in
- **d'** = 2.635 in
- **As** = 7.62 in²
- **As'** = 7.62 in²
- **fy** = 40,000 psi
- **beta1** = 0.85
- **Pu** = 635 kip

### Main Bars
- **# 10 Main Reinforcement**
- **Tot** = 6
- **A_s [in^2]** = 1.27
- **d_s [in]** = 1.27
- **As [in^2]** = 7.62

### Shear Bars
- **# 4 Shear Confinement**
- **Spacing [in]** = 18
- **A_v [in^2]** = 0.2
- **d_v [in]** = 0.5

### Pure Compression (LRFD 5.7.4.4)

\[ \phi = \text{phi}[0.80*fc' \cdot (t \cdot b - As - As') + (As + As') \cdot fy] \]

- **\( \phi P_o \)** = 3,477,120 lbs
- **\( \phi P_o \)** = 3,477 kips

### Comp Bars
- **# 10 Main Reinforcement**
- **Tot** = 6
- **A_s [in^2]** = 1.27
- **d_s [in]** = 1.27
- **As [in^2]** = 7.62

### Reinforcement of Compression Members (LRFD 5.7.4.2)

**Max Reinforcement**

\[ \frac{Ast}{Ag} < 0.08 \]

- **Ast/Ag** = (As+As_prime)/(t*b)

**Min Reinforcement**

- **Ast/Ag > 0.0025** for pier wall, and 0.01 for comp. member
- **Ast/Ag** = (As+As_prime)/(t*b)

### Lateral Reinforcement (BDS 8.18.2)

- **spacing > min(12", t=24")**, Check

### Ties (BDS 8.18.3)

\[ h = \frac{276}{6.5333} \text{ in} \]

- **h_wall** = 276 in
- **Ash** = 6.5333 in²
- **h_c** = 58.5 in
- **Ag** = 1449 in²
- **Ac** = 1341 in²

\[ 0.30*s_t'h_c'fc' / fy \cdot (Ag/\text{Ac}-1) = 1.59 \text{ in}^2 \]

### Demand

\[ 6.53 \text{ in}^2 > \text{Demand} = 1.59 \text{ in}^2 \] ok
LRFD 5.8.3.3

Vn = min of the following 2 equations

\[ Vn = Vc + Vs + Vp \]
\[ Vn = 0.25f'c bv dv + Vp \]

\( b = 63 \text{ in} \)
\( d = 23 \text{ in} \)
\( Ac = 1449 \text{ in}^2 \)
\( f_c [ksi] = 2500 \text{ psi} \)
\( dv = 20 \text{ in} \)
\( bv = 63 \text{ in} \)

Ecco Results:

\( V [kip] = 29 \text{ kip} \)
\( \text{Demand} = 129 \text{ kip} \)

\( f = 1.00 \) Seismic phi for shear

Select: \( fV [kip] = 29 \text{ kip} \)
\( \text{Demand} = 129 \text{ kip} \)

5.8.2.5 Minimum Transverse Reinforcement

\[ Av > 0.0316 \sqrt{f'c} \frac{b}{s} / f_y \]

Analysis:

5.8.3.4.2 General Procedure

es = \( \frac{(Mu/dv + 0.5 Nu + |Vu-Vp| - Apsfpo)}{(Es As + Ep Aps)} \)

\( Mu = 32184 \text{ k-ft} \) Moment demand
\( Nu = 653 \text{ k} \) Axial demand
\( Vu = 441 \text{ k} \) Shear demand
\( Vp = 0 \text{ k} \) component in direction of applied shear of the effective prestressing force positive if resisting the applied shear

\( Aps = 0.00 \text{ in}^2 \) Area of PS
\( fpo = 175 \text{ ksi} \) 0.7 fpu
\( f_y = 40 \text{ ksi} \)
\( Es = 29000 \text{ ksi} \)
\( Ep = 29000 \text{ ksi} \)

Flexural steel info:

\( A_s_bar = 1.27 \text{ in}^2 \)
\( d_s_bar = 1.27 \text{ in} \)

Sxe = \( s_x * 1.38 / (ag + 0.63) \)

\( sx = \) min of following
\( dv = 20 \text{ in} \)
\( d = 36 \text{ in} \) dist between layers of long crack control reinf

ag = \( 0.25 \text{ in} \) max agg size assumed per photos

Sxe = \( 31.4 \)

theta = 29 + 3500 es

theta = 61 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met

\( \beta = 4.8 / (1 + 750 \text{ cs}) \)

\( \beta = 0.61 \) factor indicating ability of diagonal cracked concrete to transmit tension & shear

If Min Transverse Reinforcement is NOT met

\( \beta = 4.8 / (1 + 750 \text{ cs}) \times 51 / (39 + Sxe) \)

\( \beta = 0.45 \) factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: \( \beta = 0.4 \) degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: \( \alpha = 90 \) degrees angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp

\( Vc = 0.0316\beta\sqrt{f'c} \frac{bv}{dv} \)

\( Vc = 28 \text{ kip} \)

\( Vs = Av f_y \sin(\alpha) < 0.095 \sqrt{f'c} \frac{bv}{dv} \)

\( Vs = 0.79 \text{ kip} \) < 189 kip

Vn = \( 28 \text{ kip} \)

Vn = 0.25f'c bv dv + Vp

\( \phi = 1.00 \) Seismic phi for shear

Select: \( \phi V [kip] = 29 \text{ kip} \)
\( \text{Demand} = 129 \text{ kip} \)

\( f = 4.45 \)

\( f = 4.45 \)
LRFD 5.8.3.3
Ecco Results:

Vn = minimum of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

Demand = 69 kip
NG

Fy [ksi] = 40 ksi

5.8.2.5 Minimum Transverse Reinforcement

If Min Transverse Reinforcement is met

Select: beta = 0.5 degrees; alpha = 90 degrees

If Min Transverse Reinforcement is NOT met

Select: beta = 0.5 degrees; alpha = 90 degrees

Analysis:

5.8.3.4.2 General Procedure

es = (Mu/dv + 0.5 Nu + [Vu-Vp] · Apstp) / (Es As + Ep Aps)

Moment demand
Axial demand
Shear demand
Positive if resisting the applied shear

Area of PS
Flexural steel info:

Sxe = sx * 1.38 / (ag + 0.63)

beta = 4.8 / (1 + 750 es) factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: beta = 0.5 degrees; alpha = 90 degrees

angle of inclination of diagonal comp. stress

Sxe = 56.5

theta = 29 + 3500 es

Select: alpha = 90 degrees; beta = 0.5 degrees

angle of inclination of transverse reinforcement

Select: phi [kip] = 36 kip; < Demand = 69 kip

NG
Portal Bracing:
As-built details of Portal sections & reinforcement:
Analysis below shows the limits where the Portal exceed D/C ratio of 1.0 in Red:

Red block below indicates the approximate location of where the Portal will be retrofitted.
Portal modeling:
Portal Moment Curvature to determine $l_{\text{crack}}$:
Concrete Section input:

- Overall, h [in] = 36.00 in
- Overall, b [in] = 24.00 in
- Concrete f’c [ksi] = 2.5 ksi
- Steel Reinforcement input:
  - # 7 conservative of 1 1/8” SQ bar
  - Shear Bars # 3
  - Shear Confinement d_v [in] = 0.375
- Manufacture FRP input:
  - Design limits:
    - Overall, h [in] = 36.00 in
    - Overall, b [in] = 24.00 in
    - Concrete f’c [ksi] = 2.5 ksi
    - Envelope of FRP:
      - FRP composite material thickness per layer
      - FRP within allowable stress
      - RFP = 0.0079
- FRP Analysis:
  - Forces:
    - F_Conc = 0.85*f’c*a*b
    - F_steel = A_s*fy
    - F_frp = A_f*fe
  - Strains:
    - Concrete ε_c
    - Steel ε_s
    - Fiber ε_fe
  - Neutral Axis:
    - c [in] = calculated distance between N.A. to extreme conc fiber
  - Moment Arm:
    - Conc Arm = (c - a / 2)
    - Steel Arm = (d - c)
    - Fiber Arm = (h - c)
- Analysis Existing (without FRP) - Strong Axis:
  - F_Conc = 0.85*f’c*a*b
  - a [in] = 1.41 in
  - beta = 0.85
  - Check Steel Yield:
    - e_y = 0.0021
  - Arm [in] = 32.98 in

Steel Reinforcement input:

- Fy [ksi] = 40 ksi
- Photos shows 7 bars
- Photos shows 7 bars
- Main Bars # 7 conservative of 1 1/8” SQ bar
- Shear Bars # 3
- Shear Confinement d_v [in] = 0.375
- Shear Confinement d_v [in] = 0.375
- Shear Confinement d_v [in] = 0.375
- Shear Confinement d_v [in] = 0.375
- Conservation base on per as-built.

FRP within allowable stress

Assumptions:
- Plain section remains plane
- Max compressive strain in concrete is 0.003
- Stress in steel under service load should be limited to 80% of yield strength
Concrete Section input:  
Overall, \( h \) [in] = 24.00 in  
Overall, \( d \) [in] = 21.69 in  
Concrete \( f'c \) [ksi] = 2.5 ksi  
Concrete \( \epsilon_c \) = 0.003  
Photos shows 7 bars  

Steel Reinforcement input:  
Main Bars # 7  
\( A_s \) [in\(^2\)] = 0.67  
\( d \) [in] = 0.875  

Manufacture FRP input:  
Design limits:  
Ult Tensile Strength in Primary Direction = 121 ksi  

Elongation at Break = 0.85%  

Steel bars  
\( f_y \) [ksi] = 40 ksi  
\( f_{fe} \) [ksi] = 445 kip  
\( A_{frp} \) [in\(^2\)] = 5.76 in sq  

FRP Analysis:  
Strains:  
\( \epsilon_c / c \) = 0.003  
\( \epsilon_s / (d - c) \) = 0.0056  
\( \epsilon_{fe} / (h - c) \) = 0.0021  

Neutral Axis:  
\( c \) [in] = 7.58 in  
\( a \) [in] = 0.63 in  

Moment Arm:  
Conc Arm = (c - a / 2)  
Steel Arm = (d - c)  
Fiber Arm = (b - c)  

Nominal Moment:  
Conc Contr = 2,820 kip-in  
Steel Contr = 570 kip-in  
Fiber Contr = 6,256 kip-in  

Factored Moment Strength:  
\[ \# M_n [\text{kip-in}] = 8,581 \]  
\( \# M_n [\text{k-ft}] = 723 \)  

Analysis Existing (without FRP) - Weak Axis:  
\( F_{Conc} = 48 \) kip  
\( a \) [in] = 0.63 in  
\( \beta \) = 0.85  
\( \delta \) [in] = 0.74 in  

Check Steel Yield  
\( \epsilon_s = 0.1339 \)  
Steel yields  
\( \epsilon_y = 0.0021 \)  

Assumptions:  
Plain section remains plane  
Max compressive strain in concrete is 0.003  
Stress in steel under service load should be limited to 80% of yield strength
LRFD 5.8.3.3
Vn = minimum of the following 2 equations
Vn = Vc + Vs + Vp
Vn = 0.25f'c bv dv + Vp

**Ecco Results:**
Vn = 14 kip < Demand = 62 kip NG

**Analysis:**
5.8.3.4.2 General Procedure
es = (Mu/dv + 0.5 Nu + [Vu-Vp) - Apsfpo) / (Es As + Ep Aps)
Mu = 7516 k-ft Moment demand
Nu = 14 k Axial demand
Vu = 62 k Shear demand
Vp = 0 k component in direction of applied shear of the effective prestressing force

Aps = 0.00 in² Area of PS
Es = 29000 ksi Flexural steel info;
Ep = 29000 ksi Flex Bars # 10

As = 8.89 in² Flexural steel
A_s_bar = 1.27 in²
d_s_bar = 1.27 in

es = 0.0027
Sxe = sx * 1.38 / (ag + 0.63)
sx = min of following
12 in
dv = 12 in
d = 36 in dist between layers of long crack control reinf
ag = 0.25 in max agg size assumed per photos

Sxe = 18.8
theta = 29 + 3500 es
theta = 36 degrees angle of inclination of diagonal comp. stress

If Min Transverse Reinforcement is met
beta = 4.8 / (1 + 750 cs)

beta = 1.59 factor indicating ability of diagonal cracked concrete to transmit tension & shear
If Min Transverse Reinforcement is NOT met
beta = 4.8 / (1 + 750 cs) x 51 / (39 + Sxe)
beta = 1.43 factor indicating ability of diagonal cracked concrete to transmit tension & shear

Select: beta = 1.4 degrees factor indicating ability of diagonal cracked concrete to transmit tension & shear
theta = 38 degrees angle of inclination of diagonal comp. stress
Select: alpha = 90 degrees angle of inclination of transverse reinforcemen

Vn = Vc + Vs + Vp
Vc = 0.316*beta*sqrt(f'c) bv dv
Vc = 15 kip
Vs = Av fy sin(alpha) < 0.095 sqrt(f'c) bv dv
Vs = 0.66 kip < 32 kip
Vn = 16 kip Gov

Vn = 0.25f'c bv dv + Vp
Vn = 135 kip

phi = 0.90 Seismic phi for shear

Select: phi V [kip] = 14 kip < Demand = 62 kip NG 4.43
LRFD 5.8.3.3

\[ V_n = \min \{ V_c + V_s + V_p \} \]

**Ecco Results:**

\[ \begin{align*} \text{F} V \text{ [kip]} & = 11 \text{ kip} < \text{Demand} = 36 \text{ kip} \quad \text{NG} \\
\end{align*} \]

**Analysis:**

5.8.3.4.2 General Procedure

\[ \text{es} = \left( \frac{\text{Mu} / \text{dv} + 0.5 \text{Nu} \cdot [\text{Vu-Vp} + \text{Aps} / \text{fpo}]}{\text{(Es} \cdot \text{As})} \right) \]

\[ \begin{align*} \text{Mu} & = 7516 \text{ k-ft} \quad \text{Moment demand} \\
\text{Nu} & = 14 \text{ k} \quad \text{Axial demand} \\
\text{Vu} & = 62 \text{ k} \quad \text{Shear demand} \\
\text{Vp} & = 0 \text{ k} \quad \text{component in direction of applied shear of the effective prestressing force} \\
\text{Aps} & = 0.00 \text{ in}^2 \quad \text{Area of PS} \\
\text{Es} & = 29000 \text{ ksi} \\
\text{Ep} & = 29000 \text{ ksi} \\
\text{As} & = 5.08 \text{ in}^2 \quad \text{Flexural steel info:} \\
\text{fpo} & = 0.00 \text{ in} \\
\end{align*} \]

**Shear**

\[ \begin{align*} \text{A}_{v,\text{bar}} & = 0.20 \text{ in}^2 \\
\text{A}_{v,\text{bar}} & = 0.50 \text{ in} \\
\text{d}_{v,\text{bar}} & = 15 \text{ in} \quad \text{spacing of stirrup} \\
\text{dv} & = 15 \text{ in} \quad \text{eff shear depth} \\
\text{b} & = 15 \text{ in} \\
\text{f} & = 40 \text{ ksi} \\
\text{fy} & = 40 \text{ ksi} \\
\text{Spacing} & = 18.00 \text{ in} \\
\text{2} & = \text{number of bars per plane} \\
\text{Av} & = 0.02 \text{ in} \quad \text{area of shear reinforcement w/in dist s} \\
\end{align*} \]

5.8.2.5 Minimum Transverse Reinforcement

\[ Av > 0.0316 \sqrt{f'c \cdot b} \cdot s / fy \]

\[ \text{Av} = 0.02 \text{ in}^2 \quad \text{NG} \quad 0.34 \text{ in}^2 = 0.0316 \sqrt{f'c \cdot b} \cdot s / fy \]

\[ \begin{align*} \text{es} = (\text{Mu} / \text{dv} + 0.5 \text{Nu} \cdot [\text{Vu-Vp} + \text{Aps} / \text{fpo}]) / (\text{Es} \cdot \text{As}) \\
\text{Mu} & = 7516 \text{ k-ft} \quad \text{Moment demand} \\
\text{Nu} & = 14 \text{ k} \quad \text{Axial demand} \\
\text{Vu} & = 62 \text{ k} \quad \text{Shear demand} \\
\text{Vp} & = 0 \text{ k} \quad \text{component in direction of applied shear of the effective prestressing force} \\
\text{Aps} & = 0.00 \text{ in}^2 \quad \text{Area of PS} \\
\text{Es} & = 29000 \text{ ksi} \\
\text{Ep} & = 29000 \text{ ksi} \\
\text{As} & = 5.08 \text{ in}^2 \quad \text{Flexural steel info:} \\
\text{fpo} & = 0.00 \text{ in} \\
\end{align*} \]

**Flexural steel info:**

\[ \begin{align*} \text{A}_{s,\text{bar}} & = 1.27 \text{ in}^2 \\
\text{d}_{s,\text{bar}} & = 1.27 \text{ in} \\
\text{sx} & = \min \{ 15 \text{ in}, 0.25 \text{ in} \} \quad \text{max agg size assumed per photos} \\
\text{Sxe} & = 23.5 \quad \text{approx effective} \\
\text{theta} & = 29 + 3500 \text{ es} \quad \text{angle of inclination of diagonal comp. stress} \\
\text{theta} & = 43 \text{ degrees} \quad \text{angle of inclination of diagonal comp. stress} \\
\text{beta} & = 1.23 \quad \text{factor indicating ability of diagonal cracked concrete to transmit tension & shear} \\
\text{beta} & = 1.02 \quad \text{factor indicating ability of diagonal cracked concrete to transmit tension & shear} \\
\text{Select: beta} & = 1.0 \text{ degrees} \quad \text{factor indicating ability of diagonal cracked concrete to transmit tension & shear} \\
\text{Select: theta} & = 43 \text{ degrees} \quad \text{angle of inclination of diagonal comp. stress} \\
\text{Select: alpha} & = 90 \text{ degrees} \quad \text{angle of inclination of transverse reinforcement} \\
\text{Vn} & = V_c + V_s + V_p \\
\text{Vn} & = 12 \text{ kip} \quad \text{Gov} \\
\text{Vn} & = 0.25f'c \cdot dv + Vp \\
\text{Vn} & = 141 \text{ kip} \\
\text{phi} & = 0.90 \quad \text{Seismic phi for shear} \\
\text{Select: phi} & = 0.90 \text{ kip} < \text{Demand} = 36 \text{ kip} \quad \text{NG} \\
\end{align*} \]
EVALUATION OF REINFORCING BARS IN OLD REINFORCED CONCRETE STRUCTURES

INTRODUCTION

Most practicing structural engineers sooner or later face the task of evaluating old structures. This task is always an interesting challenge, because it is never a routine application of the current practice in design. Owners commonly require re-evaluation when planning a change in building usage, restoration, additional stories, or lateral additions in any combination. Frequently, the original contract documents, the “as-built” revisions, and so on, cannot be found.

The structural engineering challenge is two-fold. First, the material properties must be determined for the concrete. The concrete can and usually does gain 25 percent or more strength than it had at 28 days, but the concrete can also have deteriorated under fire or chemical exposures. The second challenge concerns the reinforcing bars — determining the yield strength, the bar sizes and their cross-sectional areas, the locations of the bars, effective depths of structural members, the bending and cut-off details of the bars, and development lengths (bond and anchorage).

Where documentation is lacking for the existing structure, the following abbreviated history of reinforcing bars may be a useful starting point.

Reference 1 is an excellent presentation on the history of reinforced concrete. Included in the article are illustrations of a variety of patented reinforcing bars, and an extensive list of references regarding codes, design and construction, and reports on landmark tests.

REINFORCING BARS — SPECIFICATIONS, BAR SIZES AND ALLOWABLE STRESSES

Specifications. Reinforcing bars, as we know them today, came about in 1900. Specifications were first developed by the Association of American Steel Manufacturers in 1910. The American Society for Testing and Materials (ASTM) adopted standard specification A15 for billet-steel concrete reinforcing bars in 1911. Reinforcing bars were plain and deformed in structural, intermediate and hard grades (minimum yield strengths), or deformed, cold-twisted. Structural grade (minimum \( f_y = 33,000 \) psi) was normally used, unless otherwise specified. The specified minimum yield strengths of structural, intermediate, and hard grades were 33,000, 40,000, and 50,000 psi, respectively. The minimum yield strength of cold twisted bars was specified at 55,000 psi.

ASTM also issued similar specifications for rail-steel (A16) and axle-steel (A160) reinforcing bars. The minimum yield strength for rail-steel bars was 50,000 psi, and for axle-steel bars the same as for billet steel bars.

Table 1 summarizes the ASTM specifications for reinforcing bars from 1911 to the present.

Bar Sizes. Table 2 shows the standard reinforcing bar sizes recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in its 1924 Report (Reference 2).

Allowable Stresses. Some early authorities stated that allowable stresses in tension in the reinforcement higher than 12,000 psi show “very little to be gained in economy” and recommended a maximum of 14,000 psi (Reference 3). Recommended allowable stresses in tension in the 1924 Joint Committee Report (Reference 2) were:

- 16,000 psi for structural grade and rail-steel bars
- 18,000 psi for intermediate and hard grade bars and twisted bars.

In its 1940 Report, the Joint Committee increased its recommended allowable stresses to:

Tension

- 18,000 psi for structural grade bars
- 20,000 psi for intermediate and hard grades or rail-steel bars
- 16,000 psi for all web reinforcement

Compression

- 16,000 psi for intermediate grade bars
- 20,000 psi for hard grade or rail-steel bars
### Table 1—Reinforcing Bars 1911 to Present; ASTM Specifications; Minimum Yield and Tensile Strengths in psi

<table>
<thead>
<tr>
<th>ASTM Spec</th>
<th>Start</th>
<th>End</th>
<th>Steel Type</th>
<th>Grade 33 (Structural)</th>
<th>Grade 40 (Intermediate)</th>
<th>Grade 50 (Hard)</th>
<th>Grade 60</th>
<th>Grade 75</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Min. Yield</td>
<td>Min. Tensile</td>
<td>Min. Yield</td>
<td>Min. Tensile</td>
<td>Min. Yield</td>
</tr>
<tr>
<td>A15</td>
<td>1911</td>
<td>1966</td>
<td>Billet</td>
<td>33,000</td>
<td>55,000</td>
<td>40,000</td>
<td>70,000</td>
<td>50,000</td>
</tr>
<tr>
<td>A408</td>
<td>1957</td>
<td>1966</td>
<td>Billet</td>
<td>33,000</td>
<td>55,000</td>
<td>40,000</td>
<td>70,000</td>
<td>50,000</td>
</tr>
<tr>
<td>A432</td>
<td>1959</td>
<td>1966</td>
<td>Billet</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A431</td>
<td>1959</td>
<td>1966</td>
<td>Billet</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A615</td>
<td>1968</td>
<td>1972</td>
<td>Billet</td>
<td>40,000</td>
<td>70,000</td>
<td>60,000</td>
<td>90,000</td>
<td>75,000</td>
</tr>
<tr>
<td>A615</td>
<td>1974</td>
<td>1986</td>
<td>Billet</td>
<td>40,000</td>
<td>70,000</td>
<td>60,000</td>
<td>90,000</td>
<td>75,000</td>
</tr>
<tr>
<td>A615</td>
<td>1987</td>
<td>Present</td>
<td>Billet</td>
<td>40,000</td>
<td>70,000</td>
<td>60,000</td>
<td>90,000</td>
<td>75,000</td>
</tr>
<tr>
<td>A16</td>
<td>1913</td>
<td>1966</td>
<td>Rail</td>
<td></td>
<td></td>
<td></td>
<td>50,000</td>
<td>80,000</td>
</tr>
<tr>
<td>A61</td>
<td>1963</td>
<td>1966</td>
<td>Rail</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A616</td>
<td>1968</td>
<td>1999</td>
<td>Rail</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A160</td>
<td>1936</td>
<td>1964</td>
<td>Axle</td>
<td>33,000</td>
<td>55,000</td>
<td>40,000</td>
<td>70,000</td>
<td>50,000</td>
</tr>
<tr>
<td>A160</td>
<td>1965</td>
<td>1966</td>
<td>Axle</td>
<td>33,000</td>
<td>55,000</td>
<td>40,000</td>
<td>70,000</td>
<td>50,000</td>
</tr>
<tr>
<td>A617</td>
<td>1968</td>
<td>1999</td>
<td>Axle</td>
<td>40,000</td>
<td>70,000</td>
<td>60,000</td>
<td>90,000</td>
<td></td>
</tr>
<tr>
<td>A996</td>
<td>2000</td>
<td>Present</td>
<td>Rail, Axle</td>
<td>40,000</td>
<td>70,000</td>
<td>60,000</td>
<td>90,000</td>
<td></td>
</tr>
<tr>
<td>A706</td>
<td>1974</td>
<td>Present</td>
<td>Low-Alloy</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A955M</td>
<td>1996</td>
<td>Present</td>
<td>Stainless</td>
<td>40,000</td>
<td>70,000</td>
<td>60,000</td>
<td>90,000</td>
<td>75,000</td>
</tr>
</tbody>
</table>

### BOND AND ANCHORAGE

After establishing the yield strength of the reinforcing bars, the next important property required for evaluation of old structures concerns bond and anchorage. Steel mills in the USA completed conversion of their production to “high-bond” deformations about 1947, which continue virtually unchanged to the present day. In 1947, ASTM issued a specification, designated as A305, which prescribed requirements for deformations on reinforcing bars. The A305 specification existed from 1947 to 1968. In 1968, the requirements for deformations were merged into the specifications for reinforcing bars—A615 (billet-steel), A616 (rail-steel), and A617 (axle-steel).

For older structures, it is prudent to consider all varieties of reinforcing bars—plain round, old-style deformed, twisted square, and so on—conservatively and simply as 50 percent as effective in bond and anchorage as current bars. In other words, the tension development lengths, \( l_d \), for the old bars would be twice (double) the \( l_d \) required for modern reinforcing bars. Since most strength design reviews for flexure will be based on a yield strength, \( f_y = 33,000 \text{ psi} \) instead of today’s 60,000 psi, the tension development lengths for the old bars can be determined by adding 10 percent to any current table of tension development lengths, \( l_d \), for modern reinforcing bars. The main deficiencies encountered in old structures will be in tension lap splice lengths provided for bars larger than #6, and typical details with top bars larger than #6 cut off at 0.25 times clear span.

Standard end hooks, 90° or usually 180°, on old-style bars in earlier codes were considered to develop half the allowable tension stress. Under today’s strength design method, this value would approximate \( \phi f_y / 2 = (0.90)(33,000 \text{ psi}) / 2 = 15,000 \text{ psi} \).

### DETAILS OF REINFORCING BARS

**Flexural Members.** For structures built during the period 1900 to 1940, design standards and accompanying typical details of reinforcing bars evolved gradually, beginning with a bewildering variety of patented systems. Where design drawings or project specifications are not available, and no clue remains to the system used, caution is particularly prudent. Many of the older patented systems would be considered much less effective today—some were theoretically sound and went out of style because of high costs, but others were based upon theory not acceptable today. In two-way slabs, do not assume that there was only two-way reinforcement. Especially, if the topmost layer is disappointingly light, it may be part of a four-way system, with four layers instead of two. Look for diagonal bands of bars.

Where original design drawings are not available, typical details for reinforcing bars as shown in ACI Detailing Manuals (Reference 4) were commonly used since 1947. These typical details can be assumed and used for initial calculations if original service loads are known. In any case, these calculations should be confirmed or modified as soon as data on bar sizes, bar spacings, and effective depths of structural members can be checked in the field.

Particularly for flexural members, load tests are especially convincing when used to check calculated capacity based upon material tests and reconstituted...
placing drawings. In particular, even non-destructive load tests can thus be used to validate calculated deflections before and after cracking. (Reference 5).

**Columns.** Non-destructive surface tests should be employed at numerous locations to evaluate the concrete. If it is necessary, column concrete cover can be removed to observe vertical bar sizes, splice details, ties or spirals, etc., and replaced with little or no impairment of the structural capacity. Load tests on columns are generally not feasible, and so evaluation of column strength must be analytical. Even cutting out sample test cores to determine concrete strength is not generally advisable, since vertical reinforcing bars may be damaged and replacing removed concrete is not likely to be effective.

Under present codes, the contribution of spiral reinforcement to column capacity is considerably less than under old codes. In a present day evaluation, therefore, spiral columns, especially square or rectangular, are more likely to limit the total capacity than tied columns.

**Locating Reinforcing Bars.** Instruments now available permit the user to locate and follow individual reinforcing bars inside concrete slabs or beams. Some give accurate indications for the depth of concrete cover and even relative size of bar. Again, it is desirable to calibrate such readings by exposing the bars at some non-critical locations. These readings are particularly valuable in re-constrcuting the design details—bend points, cut-off points, and bar spacings—at least for the outside layers of bars.

**CONCRETE PROPERTIES**

The present day concrete properties in place should be determined by tests. Even if original project specifications are available, the specified concrete compressive strength, $f'_c$, is not a reliable value years later. Evaluation of present in-place concrete strength may be demonstrated by several more or less non-destructive methods. The ASTM standard test methods are:

(a) Test of cast-in-place cylinders, ASTM C873 (limited to use in slabs)
(b) Pulse velocity testing, ASTM C597
(c) Rebound number, ASTM C805
(d) Penetration resistance, ASTM C803
(e) Pullout strength, ASTM C900

It should be noted that all these methods require correlation with strength tests on drilled cores. The measurements of these various properties of concrete are related to compressive strength, tensile strength, or modulus of elasticity which can be converted to compressive strength of standard cylinders for design strength. Even instruments purporting to read “psi” or with “conversions provided” must be calibrated with the tests on cores from the actual concrete in question.

---

**Table 2—Standard* Reinforcing Bar Sizes (1924)**

<table>
<thead>
<tr>
<th>Size, in.</th>
<th>Area, in.$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Round</td>
</tr>
<tr>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>3/8</td>
<td>0.11</td>
</tr>
<tr>
<td>1/2</td>
<td>0.20</td>
</tr>
<tr>
<td>5/8</td>
<td>0.31</td>
</tr>
<tr>
<td>3/4</td>
<td>0.44</td>
</tr>
<tr>
<td>7/8</td>
<td>0.60</td>
</tr>
<tr>
<td>1</td>
<td>0.79</td>
</tr>
<tr>
<td>1-1/8</td>
<td>–</td>
</tr>
<tr>
<td>1-1/4</td>
<td>–</td>
</tr>
</tbody>
</table>

* Recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in its 1924 Report.

** Most suppliers offered a ¼ inch round bar, as well as the recommended standard sizes.

† The ¼ inch square bar was used, but to a lesser extent. Square bars were usually deformed, or if plain in structural grade, twisted to enhance bond and yield strength properties.

1. Round bars were plain or deformed.
2. A number of producers offered additional sizes, in 1/16-inch increments, prior to adoption of this reduced list of standard sizes.

**SELECTED REFERENCES**


2. “Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete” by Joint Committee on Standard Specifications for Concrete and Reinforced Concrete; the committee was composed of representatives of ACI, AIA, AREA, ASCE, ASTM and PCA. Reports were published in 1916, 1924 and 1940.


**OTHER RESOURCES**


“Strength Evaluation of Existing Structures”, Chapter 20, ACI 318-77, ACI 318-83, ACI 318-89 . . .


“Strength Evaluation of Existing Concrete Buildings (ACI 437R-91)”, by ACI Committee 437, 24 pp.

This report No. 48 replaces EDR No. 11.
SOFT METRIC REINFORCING BARS

While the focus of this report is on the past, it is important for readers of this document to be aware of current industry practice regarding soft metric reinforcing bars. The term “soft metric” is used in the context of bar sizes and bar size designations. “Soft metric conversion” means describing the nominal dimensions of inch-pound reinforcing bars in terms of metric units, but not physically changing the bar sizes. In 1997, producers of reinforcing bars (the steel mills) began to phase in the production of soft metric bars. Within a few years, the shift to exclusive production of soft metric reinforcing bars was essentially achieved. Virtually all reinforcing bars currently produced in the USA are soft metric. The steel mills’ initiative of soft metric conversion enables the industry to furnish the same reinforcing bars to inch-pound construction projects as well as to metric construction projects, and eliminates the need for the steel mills and fabricators to maintain a dual inventory. Thus, USA-produced reinforcing bars furnished to any construction project most likely will be soft metric.

Designations of Bar Sizes. The sizes of soft metric reinforcing bars are physically the same as the corresponding sizes of inch-pound bars. Soft metric bar sizes, which are designated #10, #13, #16, and so on, correspond to inch-pound bar sizes #3, #4, #5, and so on. The metric bar designations are simply a re-labeling of the inch-pound bar designations. The following table shows the one-to-one correspondence of the soft metric bar sizes to the inch-pound bar sizes.

### Soft Metric Bar Sizes vs. Inch-Pound Bar Sizes

<table>
<thead>
<tr>
<th>Soft Metric Bar Size Designation</th>
<th>Inch-Pound Bar Size Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>#10</td>
<td>#3</td>
</tr>
<tr>
<td>#13</td>
<td>#4</td>
</tr>
<tr>
<td>#16</td>
<td>#5</td>
</tr>
<tr>
<td>#19</td>
<td>#6</td>
</tr>
<tr>
<td>#22</td>
<td>#7</td>
</tr>
<tr>
<td>#25</td>
<td>#8</td>
</tr>
<tr>
<td>#29</td>
<td>#9</td>
</tr>
<tr>
<td>#32</td>
<td>#10</td>
</tr>
<tr>
<td>#36</td>
<td>#11</td>
</tr>
<tr>
<td>#43</td>
<td>#14</td>
</tr>
<tr>
<td>#57</td>
<td>#18</td>
</tr>
</tbody>
</table>

Minimum Yield Strengths or Grades. Virtually all steel mills in the USA are currently producing reinforcing bars to meet the metric requirements for tensile properties in the ASTM specifications. Minimum yield strengths in metric units are 300, 350, 420 and 520 MPa (megapascals), which are equivalent to 40,000, 50,000, 60,000 and 75,000 psi, respectively. Metric Grade 420 is the counterpart of standard Grade 60.

Bar Marking. Soft metric reinforcing bars are required to be identified with the Producer's mill designation, bar size, type of steel, and minimum yield strength or grade. For example, consider the marking requirements for a #25, Grade 420 metric bar, which is the counterpart of an inch-pound #8, Grade 60 bar. Regarding the bar size and grade, the ASTM specifications require the number “25” to be rolled onto the surface of the metric bar to indicate its size. For identifying or designating the yield strength or grade, the ASTM specifications provide an option. A mill can choose to roll a “4” (the first digit in the grade number) onto the bar, or roll an additional longitudinal rib or grade line to indicate Grade 420.

The 27th Edition of the CRSI Manual of Standard Practice was published in March 2001. Chapter 1 in the Manual includes a detailed presentation of the inch-pound and metric requirements in the ASTM specifications for reinforcing bars. Appendix A in the Manual shows the bar marks used by USA producers to identify Grade 420 soft metric bars.

More information about soft metric reinforcing bars is also provided in Engineering Data Report No. 42, “Using Soft Metric Reinforcing Bars in Non-Metric Construction Projects”. EDR No. 42 can be found on CRSI’s Website at [www.crsi.org](http://www.crsi.org).

Readers of this report are also encouraged to visit the CRSI Website for:
- Descriptions of CRSI publications and software, and ordering information
- Institute documents available for downloading
- Technical information on epoxy-coated reinforcing bars
- Technical information on continuously reinforced concrete pavement
- Membership in CRSI and member web links
- General information on the CRSI Foundation
- Information on the CRSI Design Awards competition

CONCRETE REINFORCING STEEL INSTITUTE
933 N. Plum Grove Road, Schaumburg, Illinois 60173-4758
Phone: 847/517-1200 Fax: 847/517-1206
[www.crsi.org](http://www.crsi.org)

This publication is intended for the use of professionals competent to evaluate the significance and limitations of its contents and who will accept responsibility for the application of the material it contains. The Concrete Reinforcing Steel Institute reports the foregoing material as a matter of information and, therefore, disclaims any and all responsibility for application of the stated principles or for the accuracy of the sources other than material developed by the Institute.
**Rumsey**

**Description**: Column Shear

**Seismic Shear Capacity (SDC 3.6)**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_c$</td>
<td>2,300 kip</td>
<td>Column axial force</td>
</tr>
<tr>
<td>$\phi$</td>
<td>0.90</td>
<td>Shear Strength reduction factor SDC 3.2.1</td>
</tr>
<tr>
<td>$R_{col}$</td>
<td>84 in</td>
<td>Column radius</td>
</tr>
<tr>
<td>$cl_r$</td>
<td>3 in</td>
<td>Clear cover</td>
</tr>
<tr>
<td>bar_s</td>
<td>9 in</td>
<td>Shear confinement reinforcement</td>
</tr>
<tr>
<td>$s$</td>
<td>9 in</td>
<td>Shear confinement reinforcement spacing</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>0.01</td>
<td>Ratio of volume of spiral or hoop reinforcement to the core column confined by the spiral or hoop reinforcement (measured out-to-out)</td>
</tr>
<tr>
<td>$f_{yh}$</td>
<td>60 ksi</td>
<td>Nominal yield stress of transverse column reinforcement</td>
</tr>
<tr>
<td>$\mu_d$</td>
<td>1.00</td>
<td>Local displacement ductility demand, limit set by SDC 2.2.4 on page 2-9</td>
</tr>
<tr>
<td>$f'c$</td>
<td>3.6 ksi</td>
<td>Assumed 1 to not yield</td>
</tr>
<tr>
<td>$A_b$</td>
<td>1.56 in$^2$</td>
<td>Area of individual reinforcing steel bar</td>
</tr>
<tr>
<td>$A_g$</td>
<td>22,167 in$^2$</td>
<td>Gross section area of column</td>
</tr>
<tr>
<td>$A_e$</td>
<td>17,734 in$^2$</td>
<td>$A_e = 0.8 \cdot A_g$ (SDC Eqn 3.17)</td>
</tr>
<tr>
<td>$D'$</td>
<td>81 in</td>
<td>Cross sectional dimension of confined concrete core measured between the centerline of the peripheral hoop or spiral</td>
</tr>
</tbody>
</table>

\[
F_1 = \frac{\rho_s f_{yh}}{0.150 + 3.67 \cdot \mu_d} \quad (SDC \text{ Eqn 3.20})
\]

\[
F_1 \text{ cal} = 8.39 \quad , \quad 0.3 \leq F_1 \leq 3
\]

\[
F_1 = 3.00
\]

\[
F_2 = 1 + \frac{P_c}{2000 A_g} \quad (SDC \text{ Eqn 3.21})
\]

\[
F_2 \text{ cal} = 1.05 \quad , \quad F_w \leq 1.5
\]

\[
F_2 = 1.05
\]

<table>
<thead>
<tr>
<th>Inside Plastic Hinge Zone</th>
<th>Inside Plastic Hinge Zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>$v_c = F_1 \cdot F_2 \cdot \sqrt{f'c} \leq 4 \cdot \sqrt{f'c}$ (SDC Eqn 3.18)</td>
<td>$v_c = 3 \cdot F_2 \cdot \sqrt{f'c} \leq 4 \cdot \sqrt{f'c}$ (SDC Eqn 3.19)</td>
</tr>
<tr>
<td>$v_{c\text{ cal}}$</td>
<td>5.99</td>
</tr>
<tr>
<td>$4 \cdot \sqrt{f'c}$</td>
<td>7.59</td>
</tr>
<tr>
<td>$v_c$</td>
<td>5.99</td>
</tr>
</tbody>
</table>

\[
V_c = v_c \cdot A_e \quad (SDC \text{ Eqn 3.16})
\]

For Inside Plastic Hinge Zone

\[
V_c = 106 \text{ kip}
\]

\[
A_v = n \cdot (\pi /2) \cdot A_b
\]

\[
A_v = 2.45 \text{ in}^2
\]

\[
V_s = A_v \cdot f_{yh} \cdot D' / s \quad (SDC \text{ Eqn 3.16}) \quad \text{For confined circular columns}
\]

\[
V_s = 1,318 \text{ kip}
\]

\[
V_n = V_c + V_s \quad (SDC \text{ Eqn 3.15})
\]

\[
V_n = 1,424 \text{ kip}
\]

\[
\phi V_n = 1,282 \text{ kip}
\]

\[
\phi V_n \geq V_o \quad (SDC \text{ Eqn 3.14})
\]

\[
V_{ocol} = 1,010 \text{ kip} \quad \text{Obtained by } (M_o^{col} + M_o^{col}) / L^{col}
\]

Assumed $L = 10'$, conservative

\[
D/C = 0.79 \quad \text{ok}
\]
Questions concerning the VERTCON process may be mailed to NGS

Latitude: 38.89022308

Longitude: 122.2384572

NGVD 29 height:

Datum shift (NAVD 88 minus NGVD 29): 0.831 meter
Patch Spalled or Delaminated Concrete:
Details below indicate the work required to repair spalled surface area. Photos following indicate several locations of concrete surface repair are required.