APPENDIX G

FAILURE INVESTIGATION OF DECK SYSTEMS

Full-Depth Cast-in-Place Deck System with Conventional Reinforcement

Three modes of failure were checked in order to compare the theory with actual test results. These modes were: (1) two way shear failure, (2) one way shear failure, and (3) flexural failure. For each mode, three values were compared which were: (1) design value according to the material design assumptions ($f_c' = 4.0$ ksi and $f_y = 60$ ksi), (2) predicted value according to actual strength of the materials used ($f_c' = 6.5$ ksi and $f_y = 95$ ksi), and (3) failure value from the test. It should be noted that both the AASHTO Standard and AASHTO LRFD do not design the slab according to one-way or two-way shear but according to flexure only. Therefore, only for flexure, a moment value required by the specifications is given. For calculations of the design and predicted values, the 0.5 in. integral wearing surface is not considered. Fig. G.1. gives the shear force and bending moment diagram at failure.

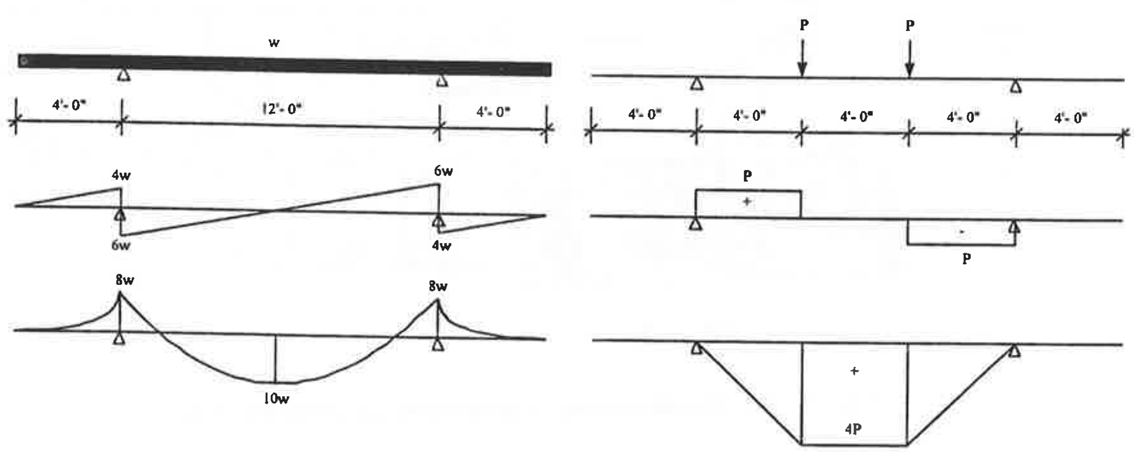


Fig. G.1. Shear Force and Bending Moment Diagrams at Failure
(w = 0.848 kip/ft/panel , P = 70 kips/panel)

Mode #1: Two-way shear failure (LRFD Art. 5.13.3.6.3)

The nominal shear resistance, V_n, is given by:

$$V_n = (0.063 + \frac{0.126}{\beta_c})\sqrt{f_c^{'}}(b_o d_v) \leq 0.126\sqrt{f_c^{'}}(b_o d_v) \quad \text{(LRFD Eq. 5.13.3.6.3-1)}$$

β_c = ratio of long side to short side of the rectangular through which the concentrated load or reaction is transmitted

b_o = perimeter of the critical section

d_v = effective shear depth, i.e distance between resultant of tensile and compression forces, (d_e - 0.5a), but not less than 0.9d_e or 0.72h

d_e = effective depth from the extreme compression fiber to the centroid of the tensile force

h = overall thickness or depth of member

AASHTO LRFD Specifications Art. 5.13.3.6.1 state that the critical section is the perpendicular section to the plane of the slab and is located so that its perimeter, b_o, is a minimum, but not closer than 0.5d to the perimeter to the concentrated load. Since the dimensions of the wheel print used in the test setup, (bxL), are 8.94 in. x 22.36 in., thus,

Design value: d_e = 9 - 0.5x0.625 - 1 = 7.69 in., a = 2.02 in.

d_v = (d_e - 0.5a) = 7.69-0.5x2.02 = 6.68 in. < 0.9d_e = 0.9x7.69 = 6.92 in. NG

> 0.72h = 0.72x9 = 6.48 in. OK

thus, d_v = 6.92 in.

b_o = 2(b+d_v) +2(L+ d_v) = 2(8.94+6.92) + 2(22.36+6.92) = 90.28 in.

$$\beta_o = (L/b) = 2.5\ ,\ (0.063 + \frac{0.126}{\beta_c} = 0.063 + \frac{0.126}{2.5} = 0.113) \leq 0.126 \qquad \text{OK}$$

$$V_n = (0.063 + \frac{0.126}{2.5})\sqrt{4.0}(90.28x6.92) = 141.7 \text{ kips}$$

Predicted value: $d_e = 7.69$ in., $a = 1.95$ in.

$d_v = (d_e - 0.5a) = 7.69\text{-}0.5x1.95 = 6.72$ in. $< 0.9d_e = 0.9x7.69 = 6.92$ in. NG

$> 0.72h = 0.72x9 = 6.48$ in. OK

thus, $d_v = 6.92$ in., $b_o = 90.28$ in.

$$V_n = (0.063 + \frac{0.126}{2.5})\sqrt{6.5}(90.28x6.92) = 180.6 \text{ kips}$$

Failure value: V = 70 kips

Mode #2: One-way shear failure (LRFD Art. 5.13.3.6.2)

Nominal shear resistance is given by:

$$V_n = V_c + V_s \leq 0.25\ f'_c b_v d_v \qquad \text{(LRFD Eq. 5.8.3.3-1)}$$

V_s = shear resistance provided by shear reinforcement = 0

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \qquad \text{(LRFD Eq. 5.8.3.3-3)}$$

In order to get the value of β, assume a value for the angle of inclination of diagonal compressive stresse, θ. Assume $\theta = 45^o$

The strain in flexural tesion reinforcement is

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5V_u\cot\theta - A_{ps}f_{po}}{E_sA_s + E_pA_p} \leq 0.002 \qquad \text{(LRFD Eq. 5.8.3.4.2-2)}$$

From Fig. 3.9, maximum M_u = 10w + 4P = 10x0.848 + 4x70 = 288.5 ft-kips/panel

= 288.5/7.5 = 38.5 ft-kips/ft

Maximum V_u = 6w + P = 6x0.848 + 70 = 75.1 kips/panel

= 75.1/7.5 = 10.0 kips/ft

Shear stress in concrete, v $= \dfrac{V_u - \phi V_p}{\phi b_v d_v}$ (LRFD Eq. 5.8.3.4.2-1)

$$= \frac{10.0 - 0}{0.9x12x6.52} = 0.142 \text{ ksi}$$

Design value: d_v = 6.92 in.

$$\varepsilon_x = \frac{\dfrac{38.5x12}{6.92} + 0 + 0.5x10(\cot 45) - 0}{29{,}000x0.53 + 0} = 4.669x10^{-3} > 0.002 \quad \text{NG}$$

Thus, use $\varepsilon_x = 0.002$

$$\frac{v}{f_c'} = \frac{0.142}{4.0} = 0.0355$$

From table 5.8.3.4.2-1 of LRFD, $\theta = 43°$, and $\beta = 1.72$

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$$

$$= 0.0316x1.72\sqrt{4.0}(12x6.92) = 9.0 \text{ kips/ft} < 0.25\sqrt{f_c'}\ b_v\ d_v \quad \text{OK}$$

$$= 9.0x7.5 = 67.5 \text{ kips/panel}$$

Predicted value: d_v = 6.92 in.

$$\varepsilon_x = \frac{\dfrac{38.5x12}{6.92} + 0 + 0.5x10(\cot 45) - 0}{29{,}000x0.53 + 0} = 4.669x10^{-3} > 0.002 \quad \text{NG}$$

Thus, use $\varepsilon_x = 0.002$

$$\frac{v}{f_c^{'}} = \frac{0.142}{6.5} = 0.0218$$

From table 5.8.3.4.2-1 of LRFD, $\theta = 43^o$, and $\beta = 1.72$

$$V_c = 0.0316\beta\sqrt{f_c^{'}}b_v d_v$$

$$= 0.0316x1.72\sqrt{6.5}(12x6.92) = 11.5 \text{ kips/ft} < 0.25\sqrt{f_c^{'}}\ b_v\ d_v$$

$$= 11.5x7.5 \qquad = 86.3 \text{ kips/panel}$$

Failure value: $V = P + 6w = 70.0 + 6x0.113x7.5 =$ 75.1 kips/panel

Mode #3: Flexural capacity (LRFD Art. 5.7.3.2)

At section of maximum positive mement between girders, top and bottom reinforcement are provided, 1.056 in^2/ft and 0.53 in^2/ft respectively. Thus, top reinforcement need to be investigated if it is in tension or in compression. Assume that the top reinforcement is in tension and does not yield, and check it later. If the distance from the extreme compression fiber to the neutral axis is c, thus:

$$f_s^{'} = (\frac{d^{'} - c}{c})\varepsilon_{cu}E_s, \qquad 0.85f_c^{'}b\beta_1 c = A_s f_y + A_{s'}f_s^{'}$$

$$d^{'} = 2.5 + 0.5x0.75 \qquad = 2.875$$

$$\beta_1 = 0.85 \qquad \text{if } f_c^{'} \leq 4.0 \text{ ksi}$$

$$= 0.85 - 0.05(f_c^{'} - 4) \geq 0.65 \quad \text{if } f_c^{'} \geq 4.0 \text{ ksi}$$

Design value: $f_s^{'} = (\frac{2.875 - c}{c})(0.003x29{,}000) = 87(\frac{2.875 - c}{c})$, $\beta_1 = 0.85$

$$(0.85x4.0x12x0.85)c = 0.53x60 + 1.056x87(\frac{2.875 - c}{c})$$

Solving for c, $c = 2.02$ in. $< (d' = 2.875$ in.) OK

$a = \beta_1 c = 0.85x2.02 \quad = 1.72$ in., $f_s' = 36.8$ ksi

$d = 7.69$ in., $d' = 2.875$ in.

Nominal flexural resistance is given by:

$$M_n \quad = A_s f_y (d - a/2) + A_s' f_s' (d' - a/2)$$

$$= [0.53x60(7.69\text{-}0.5x1.72) + 1.056x36.8(2.875\text{-}0.5x1.75)]/12$$

$$= 24.576 \text{ ft-kips/ft} \qquad = 24.576 \text{ x}7.5 = 184.320 \text{ ft-kips/panel}$$

Predicted value: $f_s' = (\frac{2.875 - c}{c})(0.003x29{,}000) = 87(\frac{2.875 - c}{c})$

$$\beta_1 = 0.85 - 0.05(f_c' - 4) = 0.85 - 0.05(6.5\text{-}4) = 0.725$$

$$(0.85x6.5x12x0.725)c = 0.53x95 + 1.056x87(\frac{2.875 - c}{c})$$

Solving for c, $c = 1.95$ in. $< (d' = 2.875$ in.) OK

$a = \beta_1 c = 0.725x1.95 = 1.41$ in., $f_s' = 41.3$ ksi

$$M_n \quad = A_s f_y (d - a/2) + A_s' f_s' (d' - a/2)$$

$$= [0.53x95(7.69\text{-}0.5x1.41) + 1.056x41.3(2.875\text{-}0.5x1.41)]/12$$

$$= 37.194 \text{ ft-kips/ft} \qquad = 37.194 \text{ x}7.5 = 278.955 \text{ ft-kips/panel}$$

Failure value: $M_u \quad = 4P + 10w = 4x70 + 10x0.848 \quad = \quad 288.5$ kips-ft

Required factored moment according to AASHTO LRFD Specifications (refer to Appendix F-1)

$$= 18.814 \text{ ft-kips/ft} \quad = 18.814 \times 7.5 \quad = \quad 141.1 \text{ ft-kips}$$

Based on a theoretical analysis of the possible failure modes, the conventionally reinforced CIP system was expected to have a flexural failure. The actual failure moment was 3% higher than the value predicted from theory. This implies that using theory, it is possible to accurately predict the type of failure for a conventionally reinforced CIP system.

Full-Depth Cast-in-Place Deck System with Welded Wire Fabric

Similar calculation was done for this sytem. The average concrete strength obtained from concrete cylinders was 6500 psi which was higher than the specified design strength (4000 psi). The 0.5 in. integral wearing surface is ignored. Bending moment and shear force diagrams are given in Fig. G.2. Note that self weight of the testing equipments are ignored.

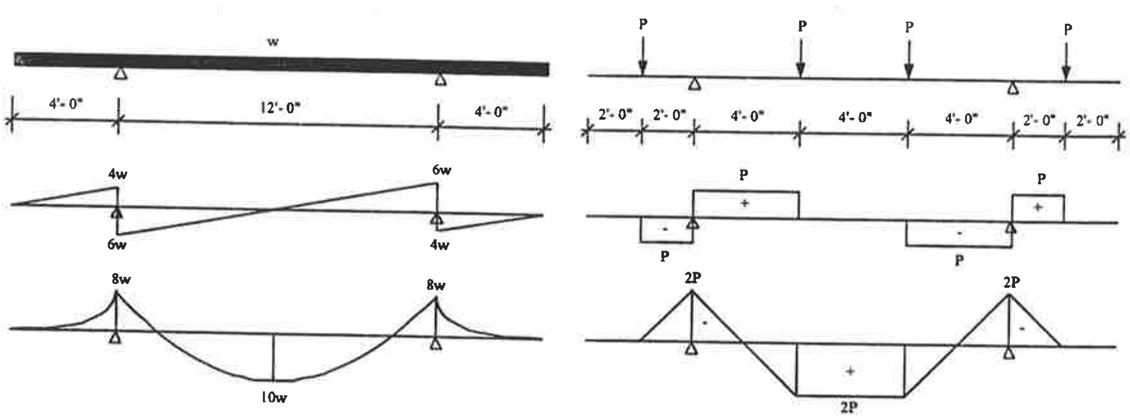


Fig. G.2. Bending Moment and Shear Force due to HS-25 Loading
(w = 0.848 kip/panel, P = 100 kips/panel)

Mode #1: Two-way shear strength: (AASHTO Art. 8.16.6.6.2)

Nominal shear resistance is given by:

$$V_n = (2 + \frac{4}{\beta_o})\sqrt{f_c'}\, b_o d \quad \leq \; 4\sqrt{f_c'}\, b_o d \qquad \text{(AASHTO Eq. 8.58)}$$

Where β_o = ratio of long side to short side of concentrated load

bo = the perimeter of the critical section

d = distance from extreme compression fiber to centroid of tension reinforcement

Art. 8.16.6.6.1 states that the critical section need not approach closer than d/2 from the perimeter of the concentrated load.

$b = 8.94$ in, $L = 22.36$ in, $\beta_o = (L/b) = 2.5$, $(2 + \frac{4}{\beta_o} = 2 + \frac{4}{2.5} = 3.6) < 4.0$ OK

At positive moment section: $d = 9 - 1.0 - 0.5 \text{x} 0.628 = 7.686$ in.

Thus, $b_o = 2(b+d) + 2(L+d) = 2(8.94+7.686) + 2(22.36+7.686) = 93.34$ in.

Design value: $V_u = (2 + \frac{4}{2.5})\sqrt{4000}(93.34 \text{x} 7.686) / 1000 = 163.3$ kips

Predicted value: $V_u = (2 + \frac{4}{2.5})\sqrt{6500}(93.34 \text{x} 7.686) / 1000 = 208.2$ kips

Failure value: V_u = 100 kips

Mode #2: One-way shear failure (AASHTO Art. 8.16.6.2)

Nominal shear resistance is given by:

$$V_c = (1.9\sqrt{f_c'} + 2{,}500\rho_w \frac{V_u d}{M_u})\, b_w d \leq 3.5\sqrt{f_c'}\, b_w d \qquad \text{(AASHTO Eq. 8-48)}$$

where $(V_u d / M_u) \leq 1.0$, and M_u is the moment occuring simultaneously with V_u

Thus, at maximum negative moment section, over the girder line:

$V_u = (6w + P) = 6x0.848 + 100 = 105.1$ kips/panel

$M_u = (8w + 2P) = 8x0.848 + 2x100 = 206.8$ kips/panel

$(V_u d / M_u) = (105.1)(7.686)/(206.8x12) = 0.326 < 1.0$ OK

$\rho_w = (1.24)/(12 \times 7.186) = 0.0144$

Design value: $V_n = (1.9\sqrt{4000} + 2{,}500x0.0144x0.326)(12x7.686)/1000$

$= 12.2$ kips/ft $< (3.5\sqrt{f'_c}\, b_w d = 3.5\sqrt{4000}(12x7.686)/1000 = 20.4$ kips$)$

$= 12.2x7.5 = 91.5$ kips/panel

Predicted value: $V_n = (1.9\sqrt{6500} + 2{,}500x0.0144x0.326)(12x7.686)/1000$

$= 15.2$ kips/ft $< (3.5\sqrt{f'_c}\, b_w d = 3.5\sqrt{6500}(12x7.686)/1000 = 26.0$ kips$)$

$= 15.2x7.5 = 114.0$ kips/panel

Failure value $V_n = 6w + P = 6x0.848+100.0 = 105.1$ kips/panel

Mode #3: Flexural capacity (Art. 8.16.3)

At section of maximum positive mement between girders, top and bottom reinforcement are provided, 1.24 in^2/ft and 1.24 in^2/ft respectively. Thus, top reinforcement need to be investigated if it is in tension or in compression. Assume that the top reinforcement is in tension and does not yield, and check it later. If the distance from the extreme compression fiber to the neutral axis is c, thus:

$$f_s' = \left(\frac{d' - c}{c}\right)\varepsilon_{cu}E_s, \qquad 0.85f_c'b\beta_1c = A_sf_y + A_{s'}f_s'$$

d' $= 2.5 + 0.5x0.628$ $= 2.814$ in., $d = 7.686$ in.

β_1 $= 0.85$ if $f_c' \leq 4.0$ ksi

$= 0.85 - 0.05(f_c' - 4) \geq 0.65$ if $f_c' \geq 4.0$ ksi

Design value: $f_s' = \left(\frac{2.814 - c}{c}\right)(0.003x29{,}000) = 87\left(\frac{2.814 - c}{c}\right)$, $\beta_1 = 0.85$

$$(0.85x4.0x12x0.85)c = 1.24x60 + 1.24x87\left(\frac{2.814 - c}{c}\right)$$

Solving for c, $c = 2.51$ in. $< (d' = 2.814$ in.$)$ OK

$a = \beta_1 c = 0.85x2.51 = 2.13$ in., $f_s' = 10.5$ ksi

Nominal flexural resistance is given by:

M_n $= A_s f_y (d - a/2) + A_s'f_s' (d' - a/2)$

$= [1.24x60(7.686\text{-}0.5x2.13) + 1.24x10.5(2.814\text{-}0.5x2.13)]/12$

$= 42.947$ ft-kips/ft $= 42.947 \times 7.5 = 322.103$ ft-kips/panel

Predicted value: $f_s' = \left(\frac{2.814 - c}{c}\right)(0.003x29{,}000) = 87\left(\frac{2.814 - c}{c}\right)$

$\beta_1 = 0.85 - 0.05(f_c' - 4) = 0.85 - 0.05(6.5\text{-}4) = 0.725$

$$(0.85x6.5x12x0.85)c = 1.24x60 + 1.24x87\left(\frac{2.814 - c}{c}\right)$$

Solving for c, $c = 2.19$ in. $< (d' = 2.875$ in.$)$ OK

$a = \beta_1 c = 0.725x2.19 = 1.59$ in., $f_s' = 24.8$ ksi

$$M_n = A_s f_y (d - a/2) + A_s' f_s' (d' - a/2)$$

$$= [1.24x60(7.686\text{-}0.5x1.59) + 1.24x24.8(2.814\text{-}0.5x1.59)]/12$$

$= 47.898$ ft-kips/ft $\quad = 47.898 \text{ x}7.5 = 359.235$ ft-kips/panel

Failure value $= 10w+2P = 10x0.848 + 2x100 \quad = 208.5$ ft-kips/ft

AASHTO Standard Specification ultimate design moment value (refer to Appendix D-2)

$= 21.423$ ft-kips/ft $=$ $21.423 \text{ x } 7.5 = 160.700$ ft-kips

The comparison with expected capacities based on the AASHTO Standards specifications showed that: (1) one way shear failure was expected to take place before flexural failure or two way shear failure; and (2) the actual one way shear failure load was 7% lower than that calculated using AASHTO Standards Specifications. The research team feels that this was because the positive reinforcement was not anchored over the girder lines which cause a stress concentration.

Use of epoxy coated deformed WWF is recommended because it would have better bonding capacity with concrete than epoxy coated smooth WWF. It is also recommended not to discontinue positive reinforcement (the bottom mat of reinforcement) over the girders in order to avoid creating stress concentration areas which initiate shear cracks.

Conventional Precast Deck Sub-panel System

Fig. G.3 gives the shear force and bending moment diagram at failure.

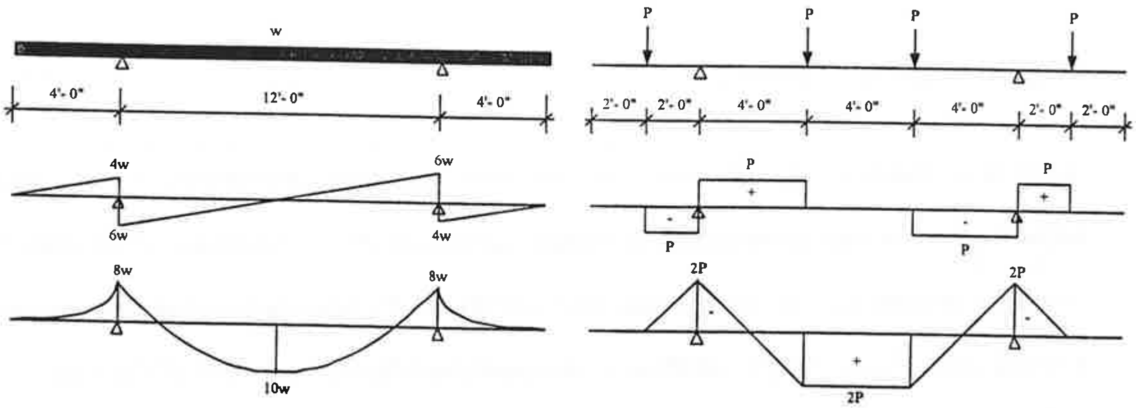


Fig. G.3. Shear Force and Bending Moment Diagram at Failure
(w = 0.848 kip/panel , P = 75 kips/panel)

Mode #1: Two-way shear strength: (AASHTO Art. 8.16.6.6.2)

Nominal shear resistance is given by:

$$V_n = \left(2+\frac{4}{\beta_o}\right)\sqrt{f_c'}\,b_o d \le 4\sqrt{f_c'}\,b_o d \qquad \text{(AASHTO Eq. 8.58)}$$

$b = 8.94$ in, $L = 22.36$ in, $\beta_o = (L/b) = 2.5$, $\left(2+\frac{4}{\beta_o} = 2+\frac{4}{2.5} = 3.6\right) < 4.0$ OK

At positive moment section, "d" is the distance from extreme compression fiber to the centroid of the prestressing reinforcement: $d = 9 - 1.5 = 7.5$ in. However, the top 6 in. of the depth "d" had a compressive strength different from that of the bottom 1.5 in. Therefore, the value of f_c' used in the above mentioned equation is calculated based on weigthed values.

Thus, $b_o = 2(b+d) + 2(L+d) = 2(8.94+7.5) + 2(22.36+7.5) = 92.60$ in.

Design value: $V_u = \left(2+\frac{4}{2.5}\right)\sqrt{\frac{4000x6+10{,}000x1.5}{6+1.5}}(92.60x7.5)/1000 = 180.3$ kips

Predicted value: $V_u = \left(2+\frac{4}{2.5}\right)\sqrt{\frac{6{,}500x6+9{,}500x1.5}{6+1.5}}(92.60x7.5)/1000 = 210.7$ kips

Mode #4: Development length of prestressed strands: (AASHTO Art. 9.27)

According to AASHTO Specifications, the minimum required length for full development of the strand is given by:

$$L = (f^{*}_{su} - 2\,f_{se}/3)\,D$$

$$= (174.9 - 2x165/3)(0.5) = 32.5 \text{ in.}$$

$$L_{available} = (48 - 3) = 45 \text{ in.}$$

Continuous Precast Deck Sub-panel System

Fig. G.4. gives the shear force and bending moment diagrams at failure.

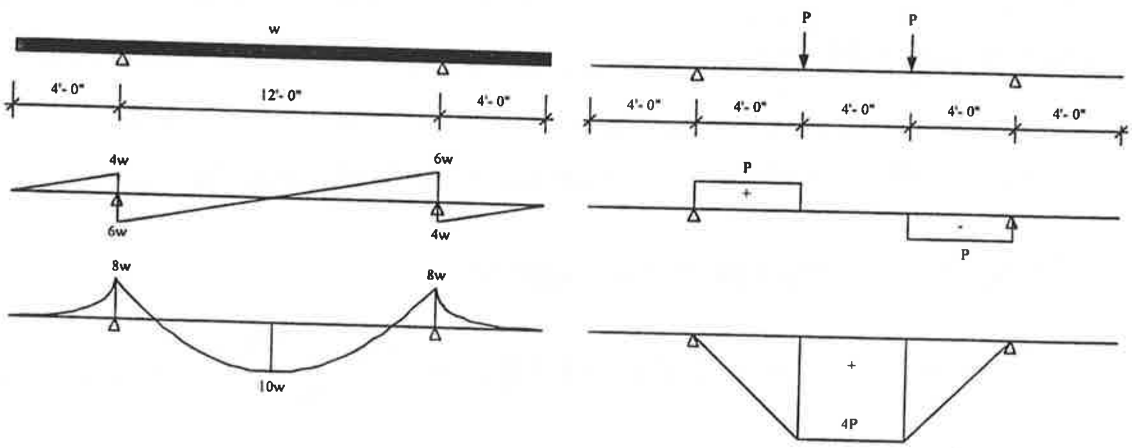


Fig. G.4. Shear Force and Bending Moment Diagrams at Failure
(w = 0.904 kip/ft/panel , P = 70 kips/panel)

Mode #1: Two-way shear strength: (AASHTO Art. 8.16.6.6.2)

Nominal shear resistance is given by:

$$V_n \quad = (2+\frac{4}{\beta_o})\sqrt{f'_c}\, b_o d \quad \leq \; 4\sqrt{f'_c}\, b_o d \qquad \text{(AASHTO Eq. 8.58)}$$

$b = 8.94$ in, $L = 22.36$ in, $\beta_o = (L/b) = 2.5$, $(2+\frac{4}{\beta_o} = 2+\frac{4}{2.5} = 3.6) < 4.0$ OK

At positive moment section, "d" is the distance from extreme compression fiber to the centroid of the prestressing reinforcement: $d = 9 - 2.25 = 6.75$ in. However, the top 4.5 in. of the depth "d" had a compresive strength different from that of the bottom 2.25 in. Therefore, the value of f'_c used in the above mentioned equation is calculated based on weigthed values.

Thus, $b_o = 2(b+d) + 2(L+d) = 2(8.94+6.75) + 2(22.36+6.75) = 89.6$ in.

$$\text{Design value: } V_u \quad = (2+\frac{4}{2.5})\sqrt{\frac{4000x4.5+10{,}000x2.25}{4.5+2.25}}(89.6x6.75)/1000 = 168.7 \text{ kips}$$

$$\text{Predicted value: } V_u = (2+\frac{4}{2.5})\sqrt{\frac{6{,}500x4.5+10{,}000x2.25}{4.5+2.25}}(89.6x6.75)/1000 = 190.6 \text{ kips}$$

Failure value: V_u = 70 kips

Mode #3: flexural capacity: (AASHTO Art. 9.17)

Based on gauge measurements placed on strands in the SIP panel, the effective stress in the strands before carrying on the ultimate test was 170 ksi measured at midspan.

At failure, the change in strain in the strand from the starting moment of applying load util the moment of failure was $1.27x10^{-3}$ in./in. Thus, the additional tensile stress added in the strands at failure was $(1.27x10^{-3})(28{,}500) = 36.2$ ksi.

Ignoring the effect of the top reinforcement, flexural resistance is given by:

$$M_n = A_s^* f_{su}^* d\left(1 - 0.6\frac{\rho^* f_{su}^*}{f_c^{'}}\right) \qquad \text{(AASHTO Eq. 9-13)}$$

$f_{su}^* = 170.0 + 36.2 = 206.2$ ksi , $A_s^* = (16x0.153) = 2.448$ in^2, d = 9 - 0.5 - 2.25 = 6.25 in.

$\rho^* = A_s^* / bd = (2.448)/(8.0x12x6.25) = 0.00408$

Design value: $$M_n = 2.448x206.2x6.25\left(1 - 0.6\frac{0.00408x206.2}{4.0}\right)/12 = 229.7 \text{ ft-kips}$$

Predicted value: $$M_n = 2.448x206.2x6.25\left(1 - 0.6\frac{0.00408x206.2}{5.5}\right)/12 = 238.8 \text{ ft-kips}$$

Failure value: $$M = 10w + 2P = 10x0.904 + 4x70 = 289.1 \text{ ft-kips}$$

Required design value according to AASHTO Standard Specifications

$$M_u = 26.3x8.0 = 210.4 \text{ ft-kips}$$

APPENDIX H

Construction Details of Full-Depth Precast Bridge Deck System

The research team has developed details for multistage deck construction with precast prestressed full-depth panels. Design and construction details using these precast panels were also refined, as a result of our experience. The following pages give the details of the multistage construction details developed.

Materials:

Concrete:

Concrete for Prestressed concrete panels shall have a minimum compressive strength f'c=7,500 psi at 28 days.

Minimum compressive strength at transfer of prestress force, f'ci= 5,000 psi.

Prestressing strands:

Prestressing Tendons shall be uncoated, seven wire, low relaxation steel strand , and shall conform to the requirements of AASHTO Designation M203.

Prestressing tendons shall strands will be placed to follow the slope of the deck and will therefore have a crown point. At the draped point, the prestressing strands will not be allowed to be held from the top but shall be supported from the bottom.

Mild reinforcing:

Deformed steel reinforcing bars shall conform to the requirements of ASTM Designation A615 or A617, Grade 60.

Deformed steel for welded wire fabric shall conform to the requirements of ASTM Designation A496. Welded deformed steel wire fabric shall conform to the requirements of ASTM Designation A497.

Precast Panels:

The top and bottom of the precast panel shall conform to the cross slope of the bridge.

Panel bedding grout stops shall be normal density expanded polystyrene foam. Polystyrene foam shall conform to the requirements of ASTM D-1621 with a maximum compressive strength of 10 psi, maximum water absorption of 0.125 pounds per square foot and a minimum oxygen index of 24.

Ducts:

Post-tensioning ducts shall meet the following requirements:

1. Post-tensioning ducts shall be non-metallic type.
2. Ducts shall be supported at intervals not to exceed 2 feet.
3. Ducts shall have vents at each end of the precast panels.
4. After installation, the ends of the ducts shall be sealed at all times to prevent entry of water and debris.
5. Before the panels are transported to the site, the contractor shall demonstrate to the satisfaction of the engineer that the ducts are not blocked.
6. Ducts must be placed using end-plate templates at the time of casting to assure accurate alignments between panels.

Grout:

Grout for transverse joints between panels and the voids between the top surface of the girders and the panels must be highly flowable at the time of placement and must attain a minimum strength of 2,000 psi in 3 hours. Set 45 as manufactured by Master Builders or an approved equal may be used for this purpose.

To achieve good adhesion between the panels at the transverse joints, it is recommended that the shear key shall be sand blasted, cleaned and wetted thoroughly before the grouting operation can be performed.

Post-Tensioning Sequencing:

Post-tensioning shall be accomplished by alternating symmetrical patterns across the roadway, beginning at the center.

Jacking Force:

The jacking force required at the jacking end of the post-tensioning tendons shall be 41.0 kips.

Initial prestress force for the 0.5" diameter pretensioning strands shall be 29 kips.

Shop Drawing:

The contractor shall submit shop drawings showing complete details of the precast panels. The precast panels shall be identified with piece marks. Piece marks shall be of a sequence that matches the erection sequence.

Shop drawings shall show complete details of the grout operation, and the post-tensioning operation.

Construction Sequence for new structure or structure closed to traffic:

1. Install grout stopper over the girder top flanges as shown on the plans.
2. Erect Prestressed concrete panels per the sequence indicated on the shop plans.
3. Adjust elevation of panels with adjustment screws.
4. Couple post-tensioning ducts at the transverse joints.
5. Clean transverse shear key as indicated previously.
6. Fill transverse joints between the panels with grout .
7. Post-tension tendons per the sequence as stated previously.
8. Fill voids around the threaded end welded studs at the ends of the panel and fill with grout.
9. Fill the voids between the panels and the girders top flange.
10. Grind top surface of deck joints as required to achieve a smooth riding surface.
11. Cast barriers.

12. Grout post-tensioning ducts.

Phasing Construction Sequence:

1. Perform existing deck removal to the limits as shown on the phasing plan.
2. Install grout stopper and preformed fabric pads over the girder top flanges as shown on the phasing plans for phase 1 construction.
3. Erect Prestressed concrete panels per the sequence indicated on the shop plans.
4. Adjust elevation of panels with adjustment screws.
5. Couple post-tensioning ducts at the transverse joints.
6. Clean transverse shear key as indicated previously.
7. Fill transverse joints between the panels with grout .
8. Post-tension tendons per the sequence as stated previously.
9. Cast barrier.
10. Fill voids around the threaded end welded studs at the ends of the panel and fill with grout.
11. Fill the voids between the panels and the girders top flange.
12. Switch traffic onto the newly constructed portion of this bridge as shown on the phasing plans.
13. Install grout stopper over the remaining girders.
14. Repeat steps 3 through 11.
15. Make closure pour.
16. Cast barrier.
17. Open bridge to two way traffic.
18. Post-tension closure pour.
19. Grout post-tensioning ducts.
20. Grind top surface of deck joints as required to achieve a smooth riding surface.

APPENDIX I

Test Program of

Concrete Girder-to-Deck Connection

Table I.1 Concrete Girder-to-Deck Horizontal Shear Specimens

Series No.	Size	No	Specimen Designation	Description	Clamping Stress (psi)
1	24" x 48"	1	BR-T-0.50	**bonded roughened with 2-#4 bars at 12" oc, double shear**	42
		2	UK-H-1.00-a	debonded shear key with 2 - 1" dia threaded rod at 12" oc, double shear	134
		3	UK-H-1.00-b	same	134
		4	UK-H-1.00-c	same	134
		5	UK-H-1.00-d	same	134
		6	UK-H-0.75-a	debonded smooth with 2 - $^3/_4$" dia rod at 12" oc, double shear	77
		7	UK-H-0.75-b	same	77
2	24" x 48"	8	UK-H-1.00-a	debonded shear key with 1 - 1" dia rod 24" oc, double shear	134
		9	UK-H-1.00-b	same	134
		10	UK-H-1.00-c	debonded shear key with 2 - 1" dia rod 24" oc, double shear	267
		11	UK-H-1.00-d	same	267
		12	UK-H-0.75-a	debonded shear key with 2 - $^3/_4$" dia rod 24" oc, double shear	154
		13	UK-H-0.75-b	same	154

Legend:

B = bonded surface
R = roughened surface
K = shear key interface
U = debonded surface
S = smooth surface

H = threaded bar
T = extended stirrups
N= new connector

a = specimen 1
b = specimen 2
c = specimen 3
d = specimen 4
e = specimen 5

(1 in. = 25.4 mm)

Table I.1 (continued)

Series No.	Size	No	Specimen Designation	Description	Clamping Stress (psi)
3	20" x 24"	14	US-0	debonded smooth surface, applied external clamping stress, single shear	0
		15	US-50	same	50
		16	US-100	same	100
		17	US-125	same	125
		18	US-150	same	150
		19	US-175	same	175
		20	US-200	same	200
		21	US-250	same	250
4	20" x 24"	22	UK-0	debonded shear key applied external clamping stress, single shear	0
		23	UK-100	same	100
		24	UK-125	same	125
		25	UK-150	same	150
		26	UK-175	same	175
		27	UK-200	same	200
		28	UK-210	same	210
		29	UK-250	same	250

Legend:

B = bonded surface
R = roughened surface
K = shear key interface
U = debonded surface
S = smooth surface

H = threaded bar
T = extended stirrups
N= new connector

a = specimen 1
b = specimen 2
c = specimen 3
d = specimen 4
e = specimen 5

(1 in. = 25.4 mm)

Table I-1 (continued)

Series No.	Size	No	Specimen Designation	Description	Clamping Stress (psi)
5	20" x 24"	30	UK-H-0.75-a	debonded shear key with 1 - $^3/_4$" dia rod, 24" oc, single shear	92
		31	UK-H-0.75-b	same	92
		32	UK-H-1.00-a	debonded shear key with 1 - 1" dia rod, 24" oc, single shear	160
		33	UK-H-1.00-b	same	160
		34	UK-H-1.25-a	debonded shear key with 1 - 1-$^1/_4$" dia rod, 24" oc, single shear	255
		35	UK-H-1.25-b	same	255
		36	UK-H-1.50-a	debonded shear key with 1 - 1-$^1/_2$" dia rod, 24" oc, single shear	368
		37	UK-H-1.50-b	same	368
6	20" x 24"	38	UK-T#4-a	debonded shear key with 2 - #4 double leg stirrups at 12" oc, single shear	100
		39	UK-T#4-b	same	100
		40	UK-T#4-c	debonded shear key with 3 - #4 double leg stirrups, at 8" oc, single shear	150
		41	UK-T#4-d	same	150
		42	UK-T#5-a	debonded shear key with 3 - #5 double leg Gr. 60 stirrups at 8" oc, single shear	230
		43	UK-T#5-b	same	230
		44	UK-T#6-a	debonded shear key with 3 - #6 double leg stirrups at 8" oc, single shear	330
		45	UK-T#6-b	same	330

<u>Legend:</u>

B = bonded surface
R = roughened surface
K = shear key interface
U = debonded surface
S = smooth surface

H = threaded bar
T = extended stirrups
N= new connector

a = specimen 1
b = specimen 2
c = specimen 3
d = specimen 4
e = specimen 5

(1 in. = 25.4 mm)

Table I.1 (continued)

Series No.	Size	No	Specimen Designation	Description	Clamping Stress (psi)
7	20" x 48"	46	UK-H-1.00-a	debonded shear key with 4 - 1" dia bar and insert at 24" oc, 100 ksi threaded bar 3 headless and 1 headed bars, single shear	16
		47	UK-H-1.00-b	same	16
		48	BR-H-1.00	bonded roughened with 1 - 1" dia bar and insert at 96" oc, 100 ksi threaded bar single shear	40
		49	BR-H-0.50	bonded roughened with 2 - $^1/_2$" dia bar and insert at 48" oc, 100 ksi threaded bar single shear	20
		50	UK-H-1.50-a	debonded shear key with 1 - 1-$^1/_2$" dia bar and insert at 96" oc, 100 ksi bar single shear	92
		51	UK-H-1.50-b	same	92
		52	UR-H-1.50-a	debonded roughened with 1 - 1-$^1/_2$" dia bar and insert at 96" oc, 100 ksi bar single shear	92
		53	UR-H-1.50-b	same	92

Legend:

B = bonded surface
R = roughened surface
K = shear key interface
U = debonded surface
S = smooth surface

H = threaded bar
T = extended stirrups
N= new connector

a = specimen 1
b = specimen 2
c = specimen 3
d = specimen 4
e = specimen 5

(1 in. = 25.4 mm)

Table I.1 (continued)

Series No.	Size	No	Specimen Designation	Description	Clamping Stress (psi)
8	20" x 48"	54	BR-T#3-a	bonded roughened with 3 - #3 double leg stirrups at 12" oc, single shear	41
		55	BR-T#3-b	same	41
		56	BR-T#3-c	bonded roughened with 3 - #3 double leg stirrups at 12" oc, single shear remove deck and place a new bonded deck on top	41
		57	BR-T#3-d	same	41
		58	UK-T#3-a	debonded shear key with 3 - #3 double leg stirrups at 12" oc, single shear remove deck and place a new debonded deck on top	41
		59	UK-T#3-b	same	41
		60	UR-T#3-a	debonded roughened with 3 - #3 double leg stirrups at 12" oc, single shear remove deck and place a new debonded deck on top	41
		61	UR-T#3-b	same	41
9	20" x 24	62	BS-0	bonded smooth surface external clamping stress	0
		63	BS-100	same	100

Legend:

B = bonded surface
R = roughened surface
K = shear key interface
U = debonded surface
S = smooth surface

H = threaded bar
T = extended stirrups
N= new connector

a = specimen 1
b = specimen 2
c = specimen 3
d = specimen 4
e = specimen 5

(1 in. = 25.4 mm)

Table I.1 (continued)

Series No.	Size	No	Specimen Designation	Description	Clamping Stress (psi)
10	24" x 32"	64	UK-N-Gr.60-a	debonded shear keys with 1 - #5 rebar connector, Gr. 60	97
		65	UK-N-Gr.60-b	same	97
		67	UK-N-Gr.60-c	same	97
		68	UK-N-Gr.60-d	same	97
		69	UK-N-Gr.60-e	same	97
		70	UK-N-Gr.100-a	debonded shear keys with 1 - #5 rebar connector, Gr. 100	162
		71	UK-N-Gr.100-b	same	162
		72	UK-N-Gr.100-c	same	162
		73	UK-N-Gr.100-d	same	162
		74	UK-N-Gr.100-e	same	162

Legend:

B = bonded surface
R = roughened surface
K = shear key interface
U = debonded surface
S = smooth surface

H = threaded bar
T = extended stirrups
N= new connector

a = specimen 1
b = specimen 2
c = specimen 3
d = specimen 4
e = specimen 5

(1 in. = 25.4 mm)

Test Variables Several test variables were evaluated in these tests. The variables include the type of shear interface and steel connectors crossing the interface. The types of interface between the girder and the deck are shown in Fig. I.1. The types of steel connectors crossing the interface are shown in Fig. I.2.

Description of Interface	Detail No.
Debonded Shear Key Interface - Stamped Application	1
Debonded Shear Key Interface - Roller Application	2
Debonded Shear Key Interface - Form Application	3
Intentionally Roughened Bonded Interface. 1/4 in. (6.4 mm) Amplitude	-
Intentionally Roughened Unbonded Interface. 1/4 in. (6.4 mm) Amplitude	-
Debonded Smooth Interface	-
Bonded Smooth Interface	-

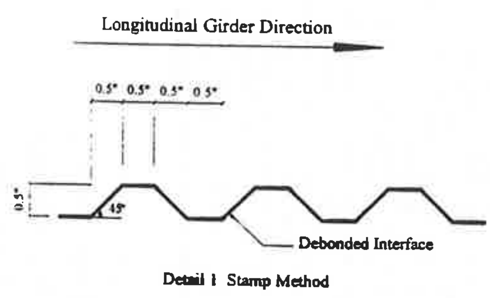
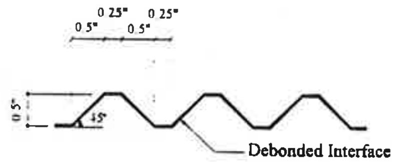
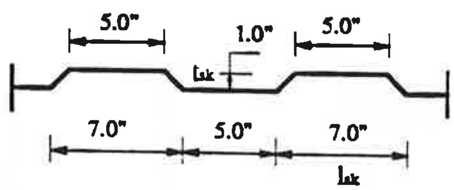


Fig. I.1 Concrete Interface Test Variables



Detail 2 Roller Method
(1 in. = 25.4 mm)



Detail 3 Formed Shear Key Method

Fig. I.1 (continued). Concrete Interface Test Variables

Description of Steel Connector	Detail No.
High Strength Coupled Threaded Rod, f_y = 100 ksi, 6 in. Round Plate at Top	1
High Strength Threaded Rod, f_y = 100 ksi, with 4 x 4 x $^3/_8$ in. Plate at Top	2
High Strength Coupled Threaded Rod with Nut at Top	3
Gr. 60 Extended Reinforcement Stirrups	4
High-Strength Threaded Rod, f_y = 100 ksi, with nut only	5
Gr. 100 & Gr. 60 #5 connector	6
No Steel Crossing the Interface	-

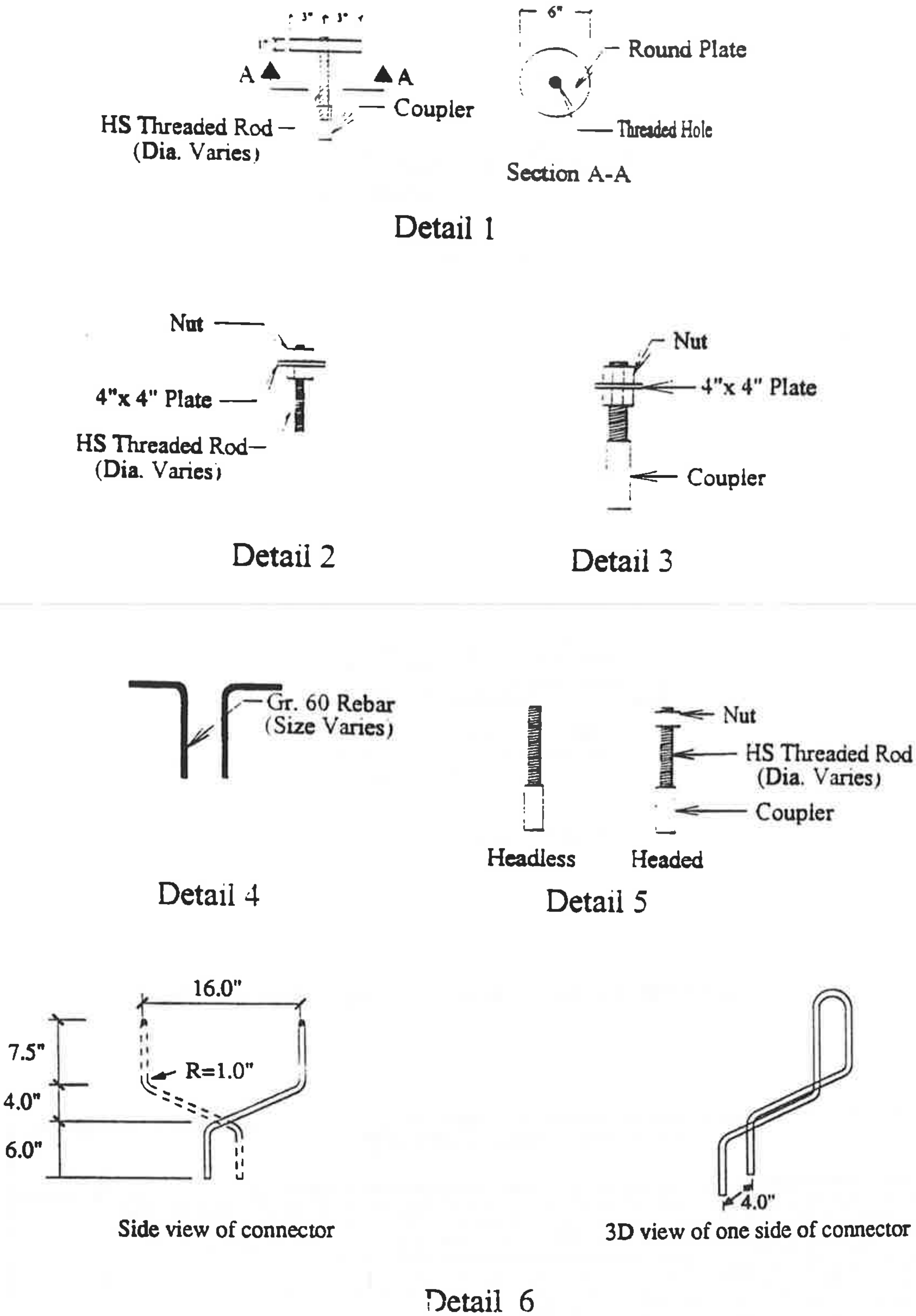


Fig. I.2 Steel Shear Connector Test Variables

Purpose and Description of Push-off Specimens

Series 1 Specimens: The purpose of Series 1 was to investigate the validity of using debonded shear keys at the surface of the girder top flange. Seven double shear push-off specimens were constructed. The area of the interface was 24 in. (609.6 mm) wide by 48 in. (1219.2 mm) long. Shear reinforcement crossing the interface was provided by high-strength threaded rods with a large round plate at both ends. One specimen was tested with Gr. 60 reinforcement crossing a roughened bonded interface. The performance and the method of making the shear keys was specifically of interest. These specimens utilized a stamp with the shear key imprint (see Fig. I.1. Detail 1) that was pressed into the plastic concrete of one of the layers at each interface. Fig. I.3 shows the specimens details.

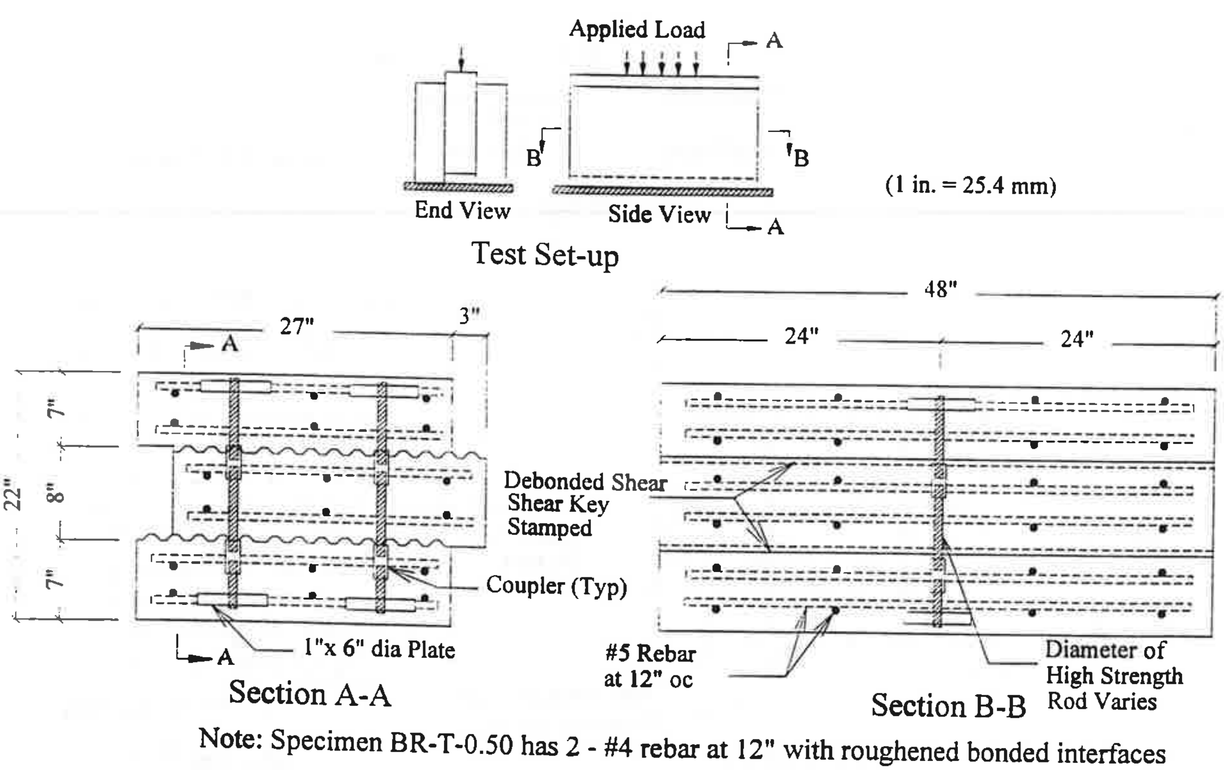


Fig. I.3 Series 1 Push-off Specimen Detail

Series 2 Specimens: The purpose of this series was to determine the effectiveness of a shear key interface with the shear key formed by a roller instead of the stamp method used for Series 1. See Fig. I.1, Detail 2, for a description of the shear key. The specimens were double shear push-off specimens with high-strength threaded rods crossing the interface. A 4 in. x 4 in. x $^3/_8$ in. (101.6 mm x 101.6 mm x 9.5 mm) square steel plate was held onto the rod on each end by two nuts. Details of the specimens are shown in Fig. I.4.

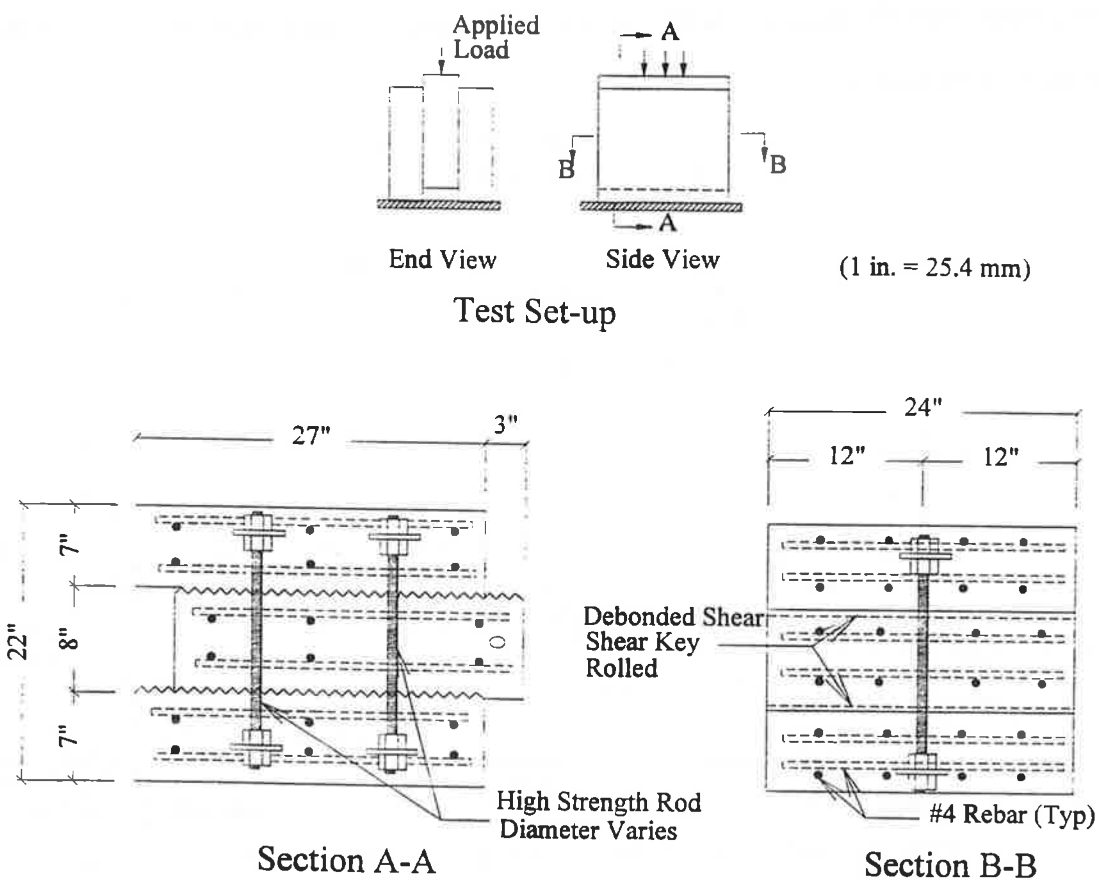


Fig. I.4 Series 2 Push-off Specimen Details

Series 3 Specimens: The purpose of this series of tests was to evaluate the coefficient of friction between two debonded smooth concrete surfaces. The smooth interface was created by a steel trowel on the wet concrete surface. The friction coefficient for a smooth concrete interface has been determined by Shiakh (1978) as 0.4. The results will be compared to Shiakh's value of 0.4, and used to obtain parameters to help develop a relationship to describe the behavior of the debonded shear keys. A controlled external clamping pressure was applied to the specimen throughout the application of the horizontal shearing force. For each of the eight specimens, the maximum horizontal shear stress at the corresponding external clamping stress will be used to determine the friction coefficient. Each specimen had a 20 in. (508 mm) x 24 in. (609.6 mm) shear interface. The reinforcement details for all specimens in Series 3 and Series 4, which are similar, are shown in Fig. I.5.

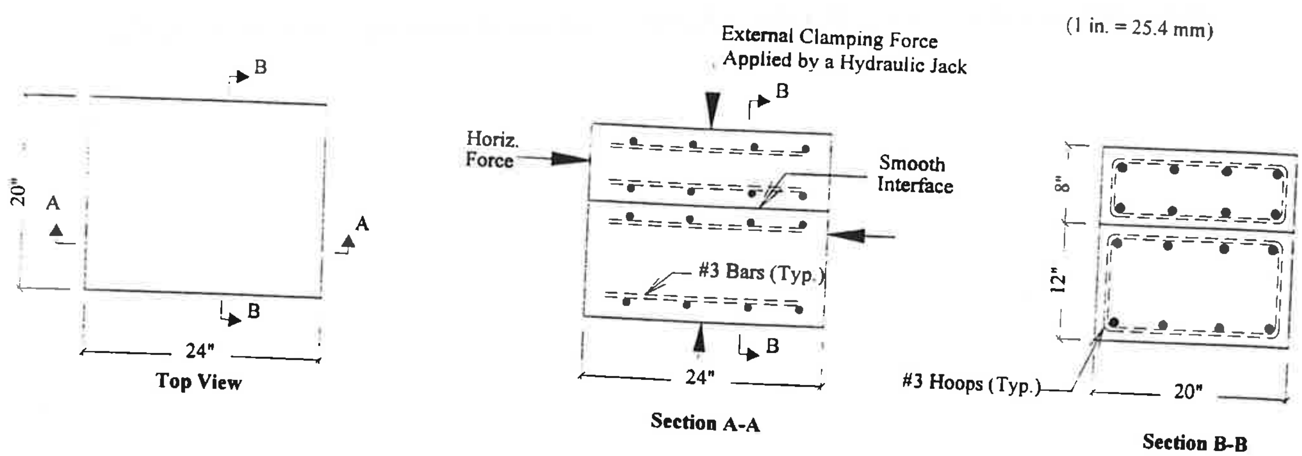


Fig. I.5 Series 3 and 4 Push-off Specimen Details

Series 4 Specimens: The purpose of this series of specimens was to determine the effect of the debonded shear key interface under different amounts of external clamping stress. In an actual bridge girder connection, the clamping stress would be provided by the reinforcement crossing the interface. This series of specimens is identical to Series 3 except for the interface. While the interface in Series 3 was a smooth finish, the interface in this series of specimens consisted of shear keys similar to those shown in Fig. I.1, Detail 2. The shear keys were applied with the roller method.

Series 5 Specimens: The purpose of this series was to evaluate the debonded shear key interface with the clamping stress on the interface, $\rho_v f_y$, provided by high strength bars placed at the center of the shear interface. The shear interface was the same size as Series 3 and 4. At both ends of the high-strength bar was a 4 in. x 4 in. x $^3/_8$ in. (101.6 mm x 101.6 mm x 9.5 mm) steel plate which was used to provide the clamping area. The variation of clamping stress was similar to that of specimens in Series 4. Two specimens for each clamping stress were constructed. Part of the bottom layer of concrete was intentionally wider with embedded hardware to allow for clamping the specimen down for stability during testing. Details of Series 5 and 6 test specimens are shown in Fig. I.6.

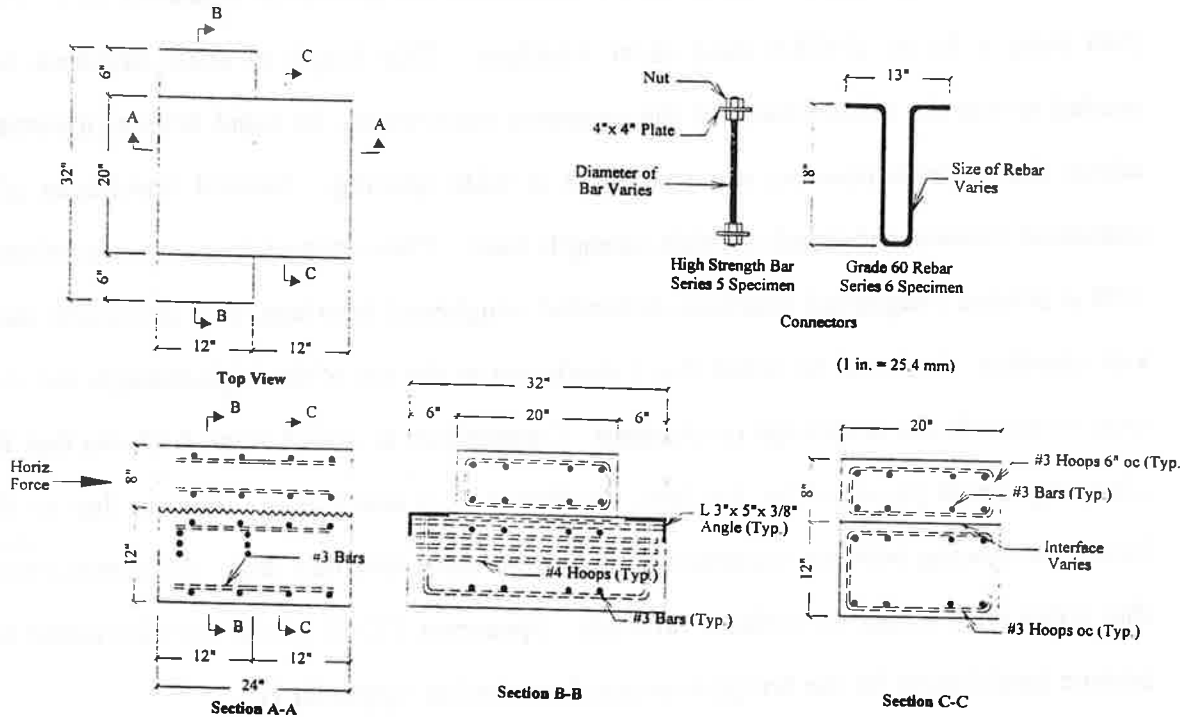


Fig. I.6 Series 5 and 6 Push-off Specimen Details

Series 6 Specimens: The purpose of this series was similar to Series 5, with the exception of the clamping stress being provided by extended double leg reinforcement stirrups. Details of the test specimens are similar to those of Series 5. The extended stirrups are similar to the existing AASHTO Code (15th Edition) horizontal shear design which requires that all vertical shear stirrups in the bridge girder extend into the CIP decks. Two push-off specimens were constructed and tested for each clamping stress. Since the yield stress of the steel crossing the interface in the specimen was less than that of Series 5, a larger amount of steel cross-sectional area was needed to provide the same ultimate clamping force.

Series 7 Specimens: The push-off specimens for this series consisted of a 20 in. (508 mm) x 96 in. (2438.4 mm) shear interface. This length of shear interface was needed to test the effectiveness of the proposed connections for rapid deck replacement where the shear connectors are spaced at a wide spacing. Several specimens also contained headed and headless high-strength bars. These connections were combined with a bonded roughened interface, debonded roughened interface, or a debonded shear key interface. It should be noted that a single nut at the top of the high-strength rod was used to provide the anchorage mechanism. Comparison to Series 5 and 6 shows that the clamping stress provided by the bars for Series 7 is much less. This is due to the increased spacing between connectors and a large shear interface area. All specimens in this series were tested for ultimate strength. Specimen UK-H-1.00-a was also tested for serivce load fatigue for the bridge example described in Appendix H.

The purpose of these specimens was to evaluate the effectiveness of the girder-to-deck connections which would facilitate future rapid deck replacement. The AASHTO Standard Code (15th Edition) limits the spacing of connectors to 24 in. (609.6 mm). This provision was purposely violated in order to test the performance of connectors at an increased spacing of 48 (1219.2 mm) and 96 in. (2438.4 mm) If the connectors are placed at a wider spacing, the effort required to remove the concrete around the connectors would be concentrated at fewer locations along the length of the bridge girder. Combined with the use of a debonded shear key interface, deck removal time would be considerably less than a bonded interface.

Specimens UK-H-1.00-a&b contained connectors at 24 in. (609.6 mm) o.c.; however, three headless and one headed high strength threaded bars (f_y = 100 ksi) were used with the debonded shear key. This scheme could allow the deck to be lifted from the girder and around the headless bolts aftèr the deck is destroyed from around the headed bolts.

Details of the push-off specimens for Series 7 are shown in Fig. I.7. A high strength coupler was used to connect the threaded rods at the interface. The rational for using a coupler a coupler was if the top connector in the deck were to be damaged or destroyed during bridge deck removal, the connector could be easily replaced through the use of the coupler.

Reinforcement in both layers of these specimens consisted of #4 bars in the longitudinal direction and #3 hoops in the transverse direction. A #2 grade 40 confinement spiral was provided around the shear connector in both the top and bottom layers of specimens UK-H-1.50-a&b, and UR-H-1.50-a&b. These spirals provided concrete confinement around the connector in order to help resist the large amount of concentrated horizontal shear force on a single connector.

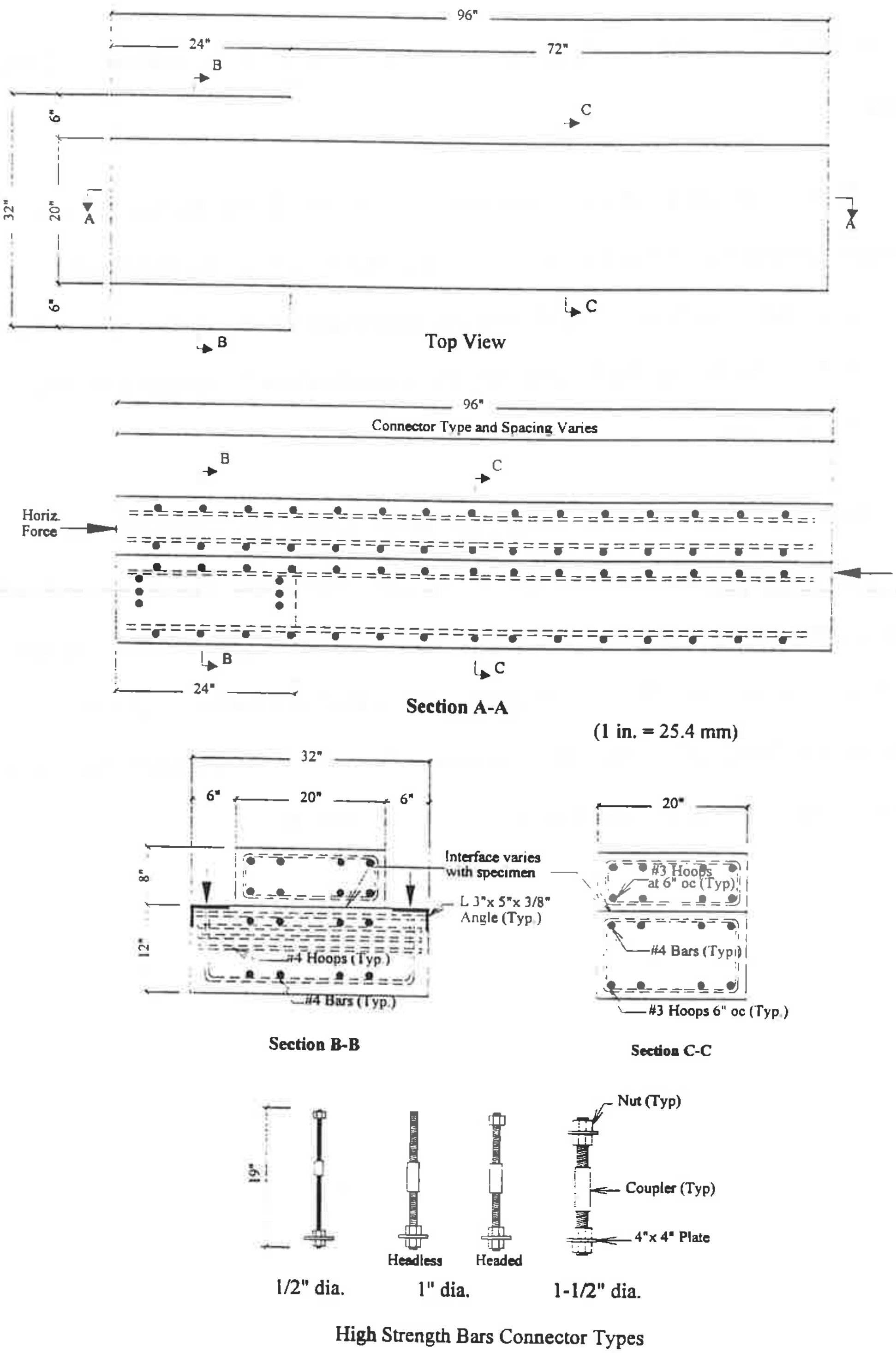


Fig. I.7 Series 7 Push-off Specimen Details

Series 8 Specimens: The purpose of this series was to evaluate the effect of the various types of interface and debonding on the difficulty of removing the top layer of concrete (which simulates the deck). Three different types of interfaces were also used to determine if deck replacement affects the horizontal shear strength of the interface. These surfaces consisted of: 1) a bonded roughened interface, 2) a debonded interface with shear keys, and 3) a debonded roughened interface.

Eight specimens were constructed. After the specimens cured, the top layer for six of the specimens was removed using a 60 pound pneumatic jack-hammer, which is representative of actual field deck removal methods above the girder top flanges. The remaining two specimens were used as a baseline for comparison. The steel connectors were #3 Gr. 60 double leg stirrups. Details of the specimens are shown in Fig. I.2. After the top layer was removed, a new layer was cast on the removed interface. The specimens were then tested for ultimate strength and the test results were compared with those of the two baseline push-off specimens.

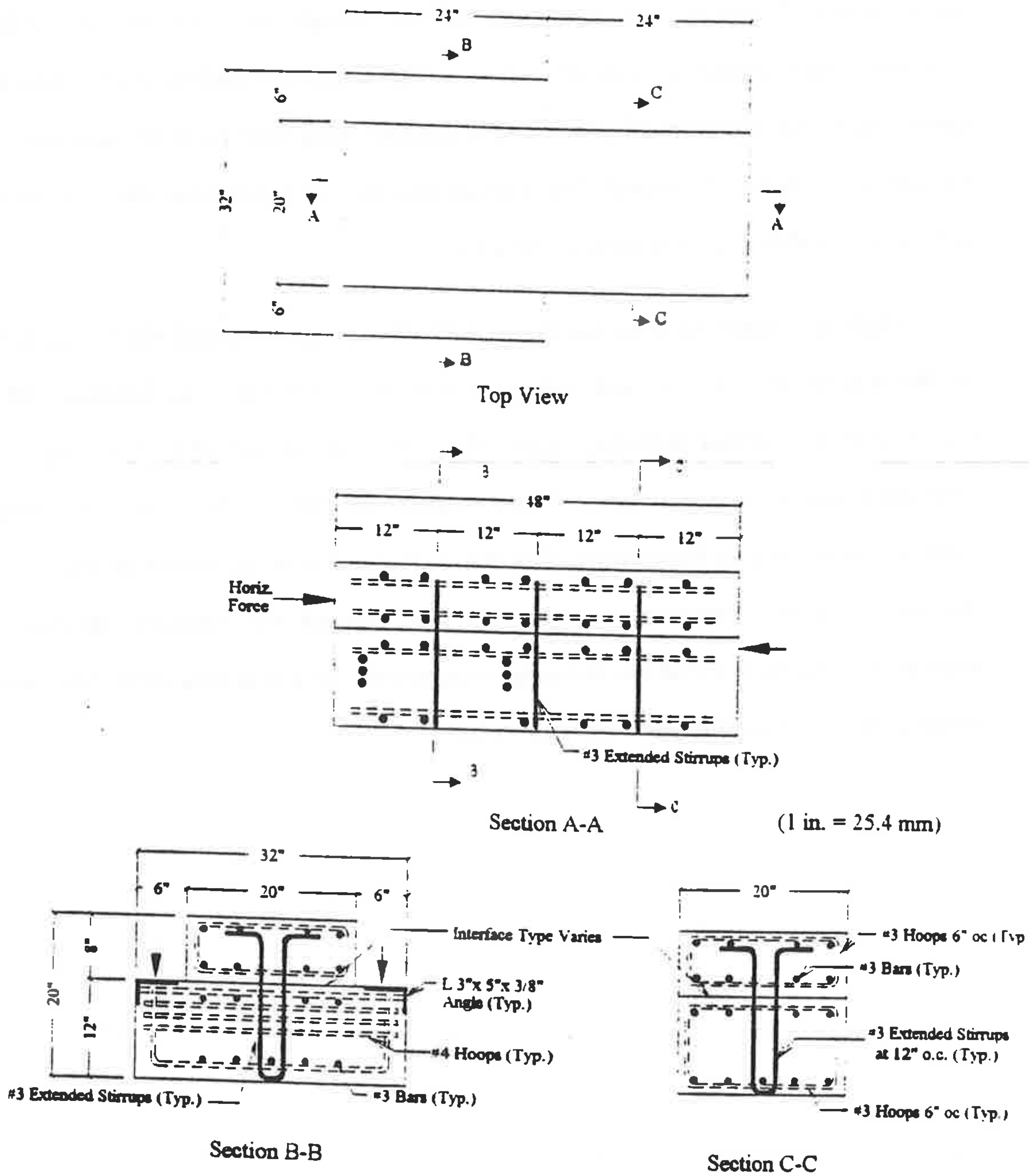
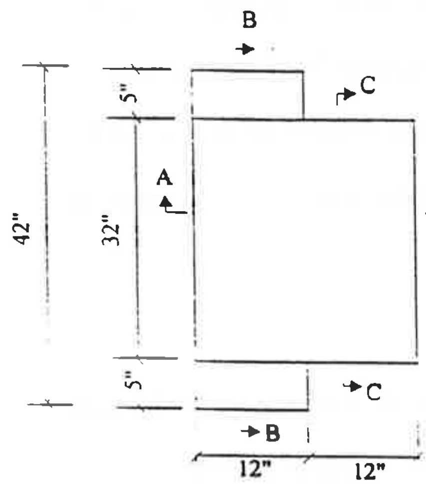


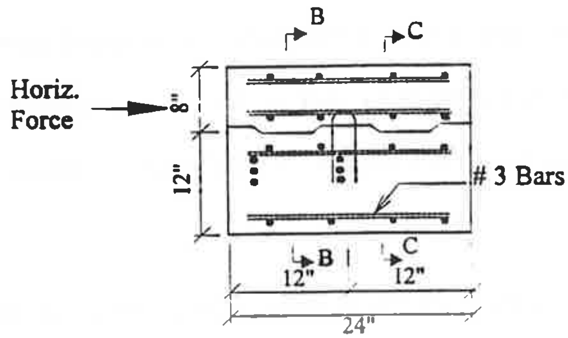
Fig. I.8 Series 8 Push-off Specimen Details

Series 9 Specimens: The purpose of this series was to measure the effect of bond (cohesion) with a smooth interface as compared with that of the debonded smooth interface used in the Series 3 specimens. Evaluation of the chemical bond (cohesion) was mainly targeted in these tests. An external clamping stress was applied at the top of the specimen similar to that of Series 3 specimens. The specimen details are the same as Series 3 with the exception of a bonded smooth interface.

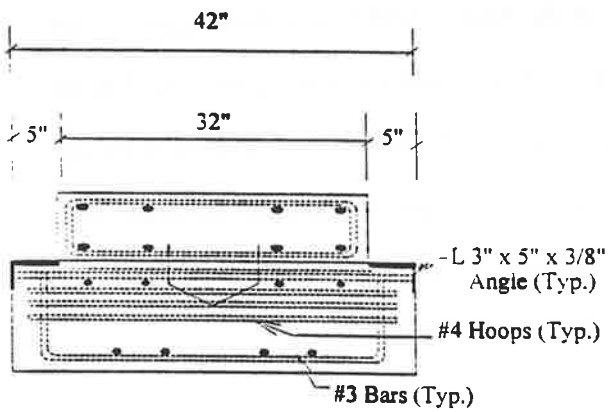
Series 10 Specimens: The purpose of this series was to evaluate an unbonded interface with shear keys protruding from precast girders and a clamping stress on the interface, $\rho_v f_y$, provided by steel connectors crossing the interface. Another purpose was to evaluate the coefficient of friction between the unbonded interface with shear keys in the precast girder (the first layer of concrete) with the bridge deck (the second layer). Ten shear push-off specimens were constructed. The area of interface was 32 in. (812.8 mm) wide by 24 in. (609.6 mm) long. A new connector was developed to be used with the proposed system. Five specimens were tested with Gr. 60 reinforcement, the other five specimens were tested with Gr. 100 reinforcement. Details of series 10 push-off specimens are shown in Fig. I.9.



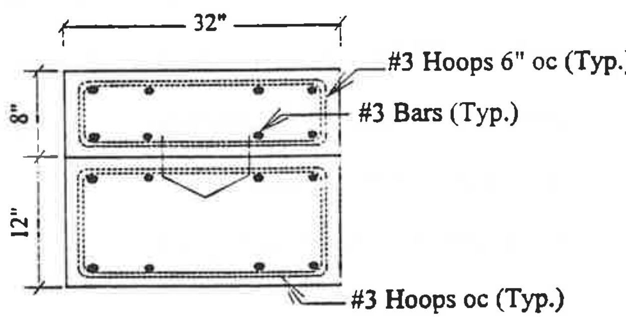
Top View



Section A-A



Section B-B



Section C-C

Fig. I.9 Series 10 Push-off Specimen Details

Material Properties

Tensile Strength of Steel Connectors: The connectors were tested for tensile strength. A typical stress-strain curve for Gr. 60 steel reinforcement used as one of the shear connectors is shown in Fig. I.10.

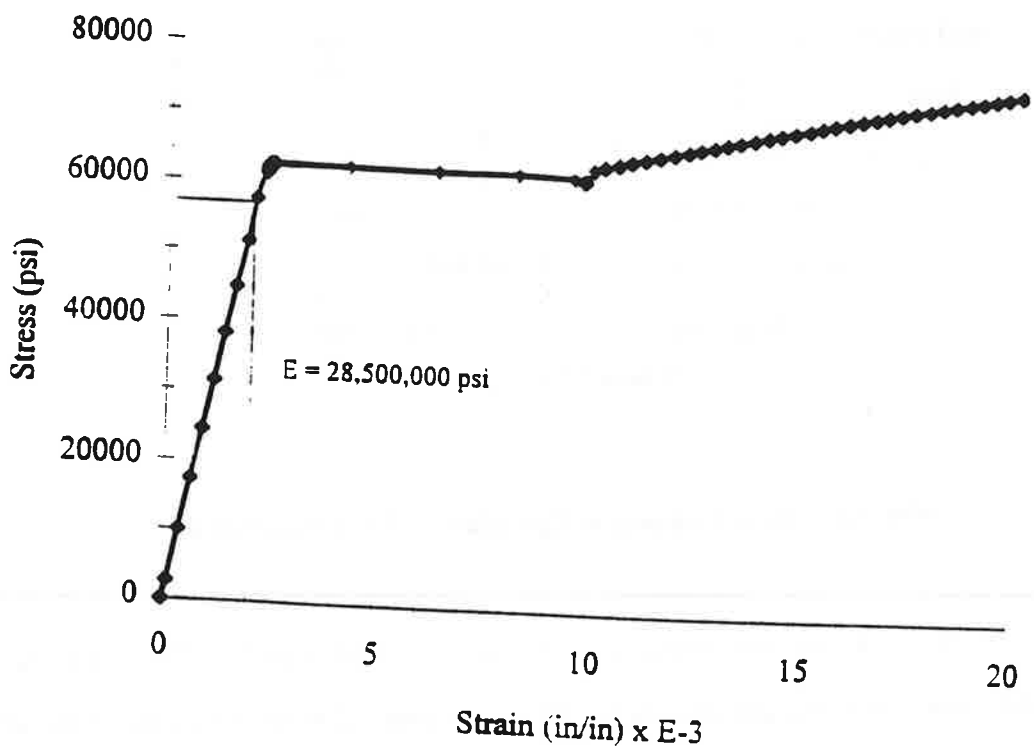


Fig. I.10 Steel Stress Strain Curve for Gr.60 Reinforcement

Direct Shear Strength of Steel Connectors: A sample of each type of steel bar used as a shear connector was tested for direct shear. A detail of the test set-up is shown in Fig. I.11. The test results are shown in Appendix I. The results show that the bar shear strength can be approximated as $0.6f_u$.

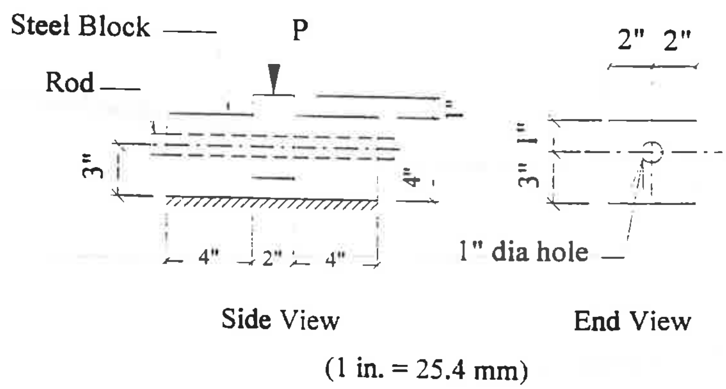


Fig. I.11 Steel Connector Bar Direct Shear Test Set-up

Concrete Compressive Strength: The concrete used was LF-5000 supplied by Ready Mix Concrete Company in Omaha, NE. Concrete cylinders were cast at the same time the push-off specimens were fabricated in order to determine the compressive strength of the concrete in both the top and bottom layers. The cylinders were cured in the same environment as the test specimens. Concrete strength for each series of specimens are given in Appendix K.

Concrete Debonding Agent: The debonding agent used on shear interfaces between the top and bottom layers of concrete was a form release product named Form-Cote by Forsco, supplied by Pro-Construction Products in Kansas City, MO. The manufacturer recommended that the first coat be applied to the concrete surface and allowed to dry for 24 hours. The first coat consisted of one part of Form-Cote and one part of Form-Cote Solvent for increased coverage. The second coat consisted of one part of Form-Coat. Each coat was allowed to dry before placing the concrete. A recommended coverage of approximately one-half pint per ft^2, per coat appeared to be sufficient. The cost for both coats is approximately $0.13 per ft^2 of coverage.

Construction of Test Specimens

Series 1 Specimens: Series 1 push-off specimens were constructed by Wilson Concrete Company of Omaha, NE. The specimens were constructed in three stages. The reinforcement and shear connectors were placed in the bottom slab prior to placing the second lift of concrete. The concrete was placed and the shear key interface was formed into the plastic concrete using the stamp method. The debonding agent was brushed on the hardened surface and allowed to dry before placing the second lift. Fig. I.12 shows the specimen after the first lift of concrete was placed.

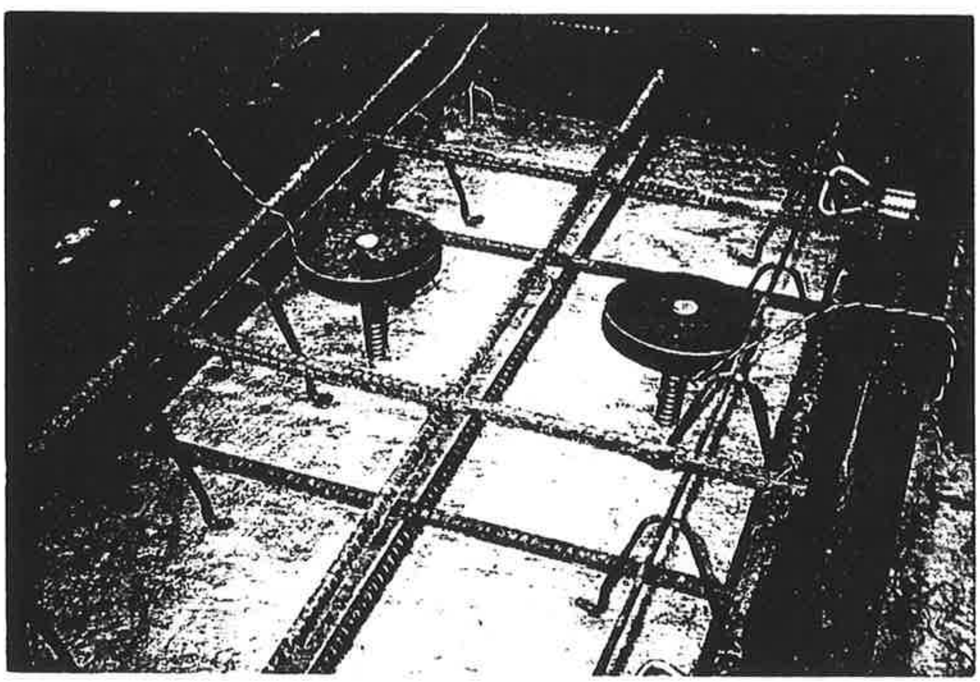


Fig. I.12 Construction of Series 1 Push-off Specimens

Series 2 Specimens: The construction sequence for Series 2 specimens was similar to that of Series 1 with the exception that the fabrication was done at the University of Nebraska Laboratory. A roller mechanism was used to form the shear keys into the plastic concrete. The debonding sealant was applied in a similar manner to Series 1.

Series 3 through 10 Specimens: The specimens were constructed in two stages. The first stage consisted of the construction of the bottom slab using plywood formwork. The steel reinforcement was tied and placed in the formwork. The steel connector bars were held in a vertical position with tie wires. Fig. I.13, I.14, and I.15 show the formwork and reinforcement for Series 7, 8, and 10 specimens, respectively. If a coupler was used with the threaded rod, it was aligned flush with the top surface of the bottom slab. Concrete was placed and consolidated using a small spud vibrator. The interface was prepared in the plastic concrete using the specified method for each series of specimens. The concrete was allowed to cure for one day. The second stage of construction consisted of cleaning the interface of laitence with muratic acid and a stiff bristled brush. The debonding sealant was then brushed on the surface with a coverage of approximately one-half pint per ft^2 and allowed to dry. These steps are shown in Fig. I.16 and I.17, respectively. The plywood formwork for the top slab was then attached to the bottom forms with wood screws and the top layers of reinforcement were tied and set in the top forms. Concrete was placed, and the top surface was troweled smooth. The concrete was allowed to cure using wet burlap. The following day, the plywood forms were removed and the specimen painted with a white paint and marked with its respective designation.

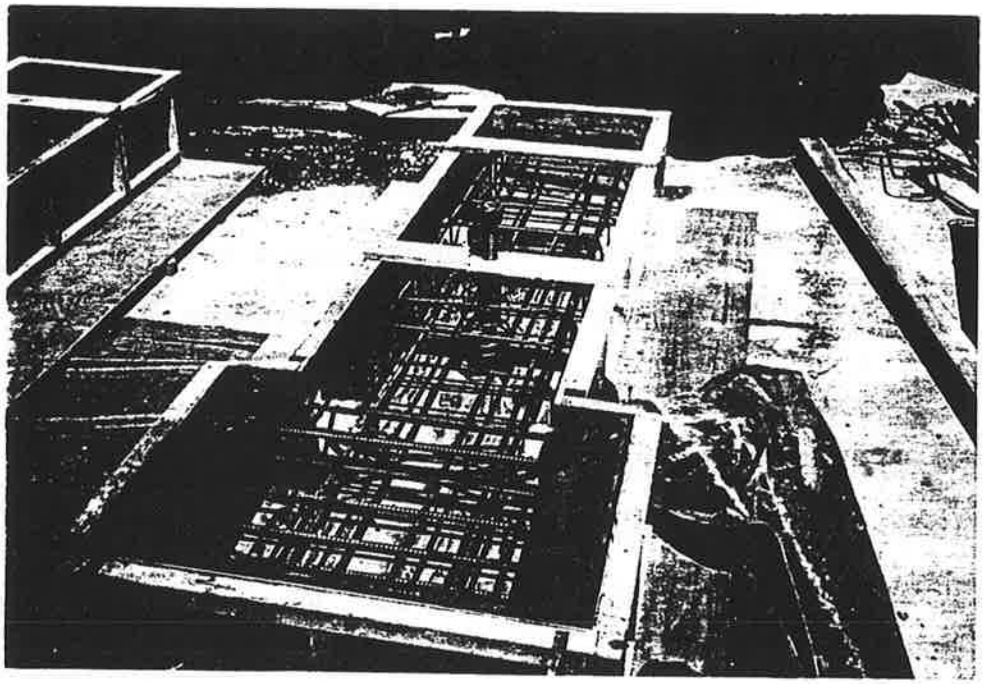


Fig. I.13 Formwork and Reinforcement for Series 7 Specimens

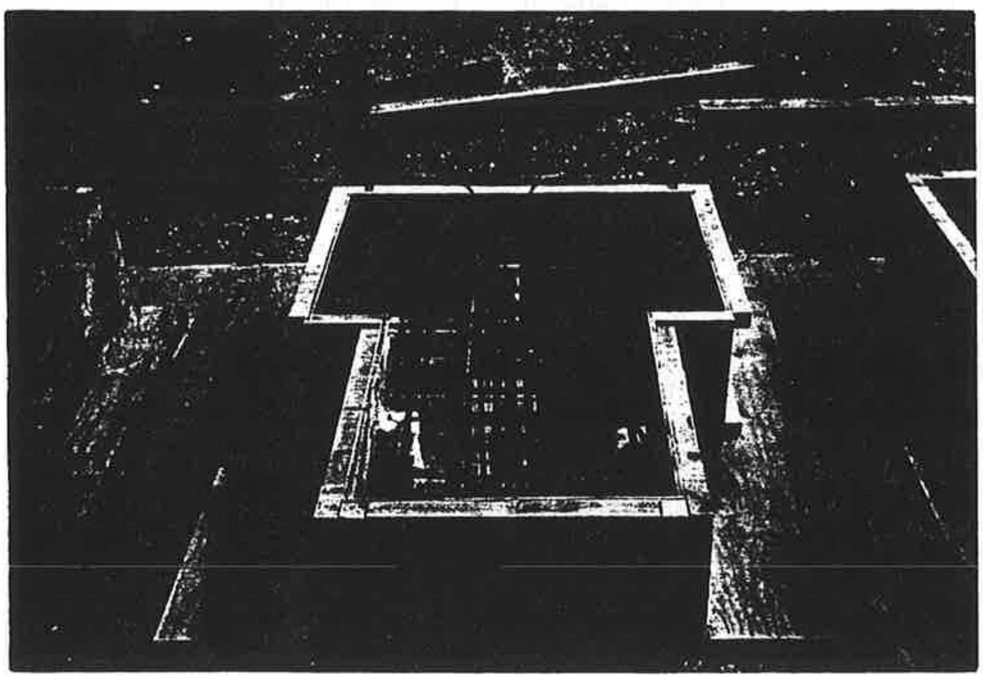


Fig. I.14 Formwork and Reinforcement for Series 8 Specimens

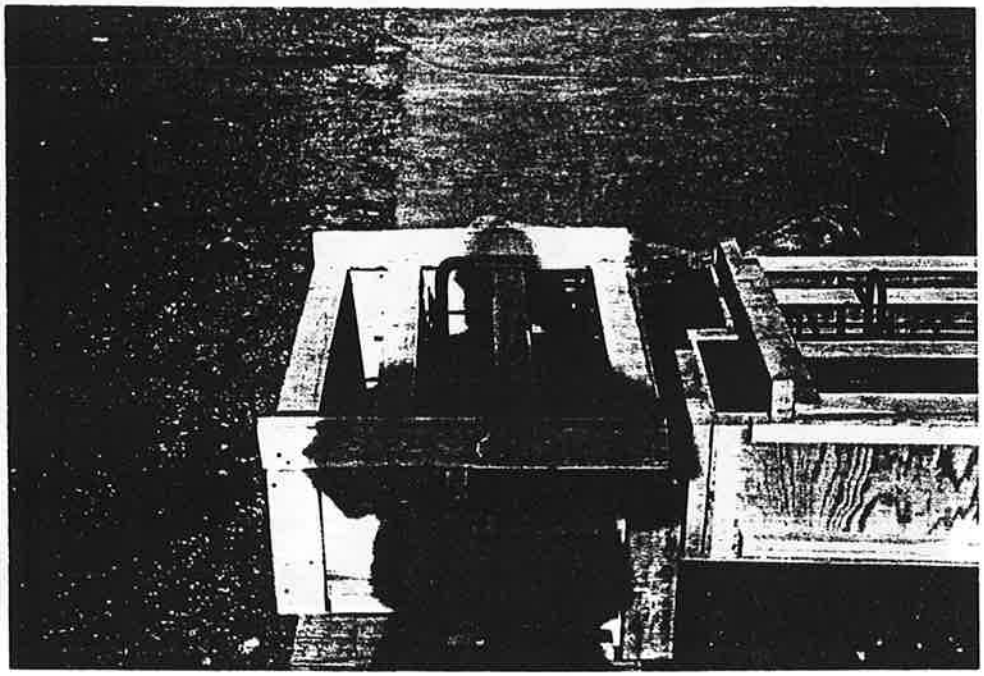


Fig. I.15 Formwork and Reinforcement for Series 10 Specimens

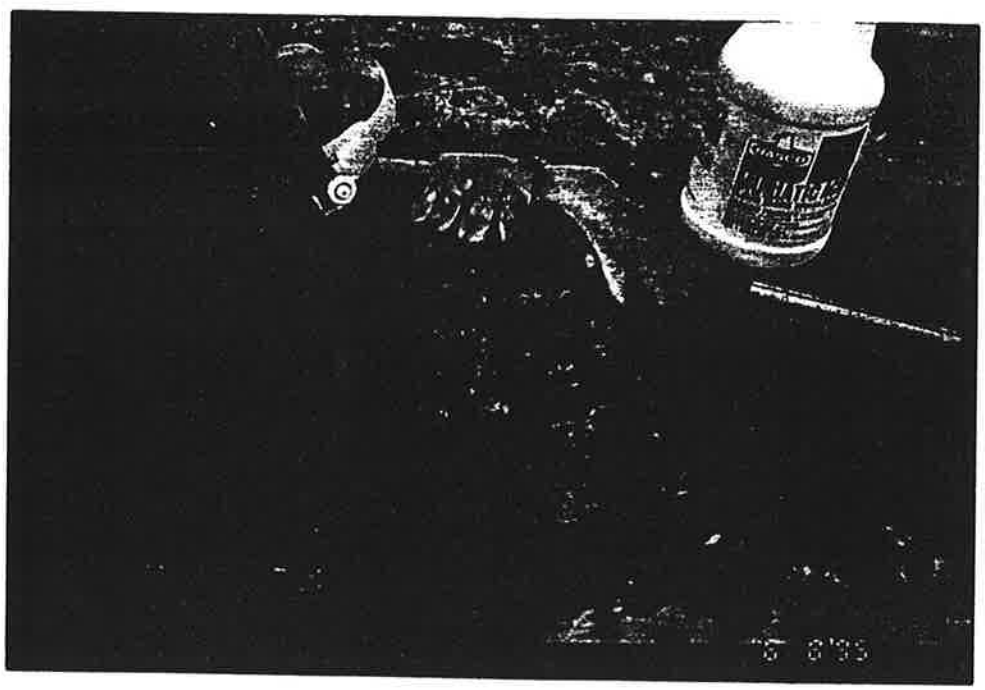


Fig. I.16 Cleaning Laitence from the Concrete Interface

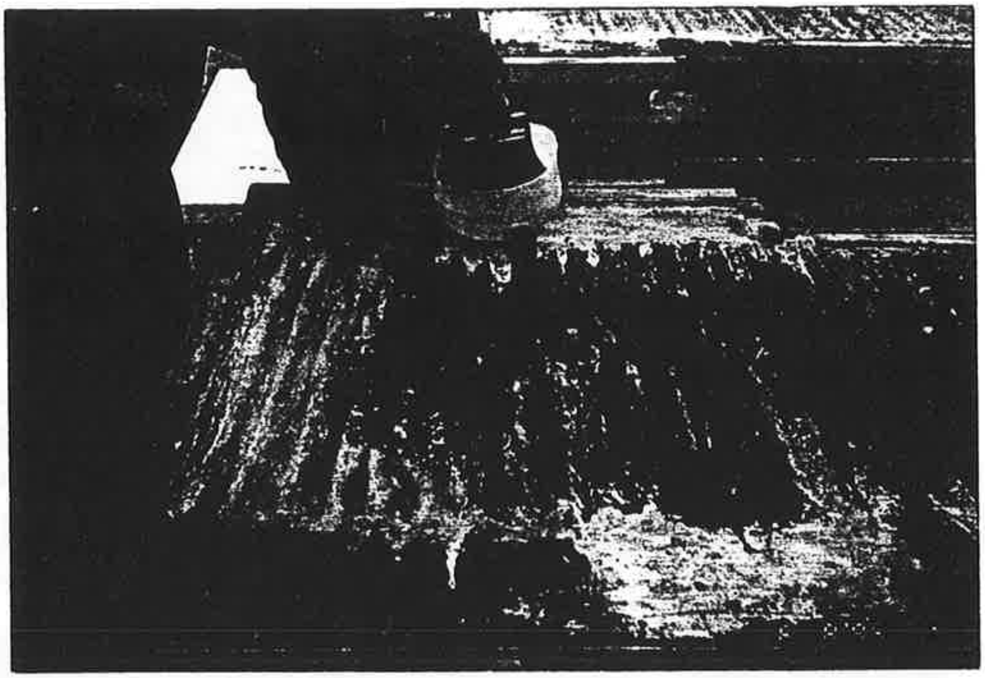


Fig. I.17 Brushing on Debonding Agent

Interface Preparation

Shear Key Method used for Series 1 Specimens: Several methods were originally investigated to determine an efficient and effective way of placing the shear keys into the surface of the girder top flange. A plywood strip with a series of shear key shapes was pressed manually into the plastic concrete at the top surface of the concrete. A detail of this stamp is shown in Fig. I.18. The stamping method was inefficient and did not form good shear keys into the concrete due to the large force required to press the stamp. If the stamping was performed when the concrete interface was still too soft, the shape of the shear keys became shallower shortly after removal of the stamp due to the slump of the concrete.

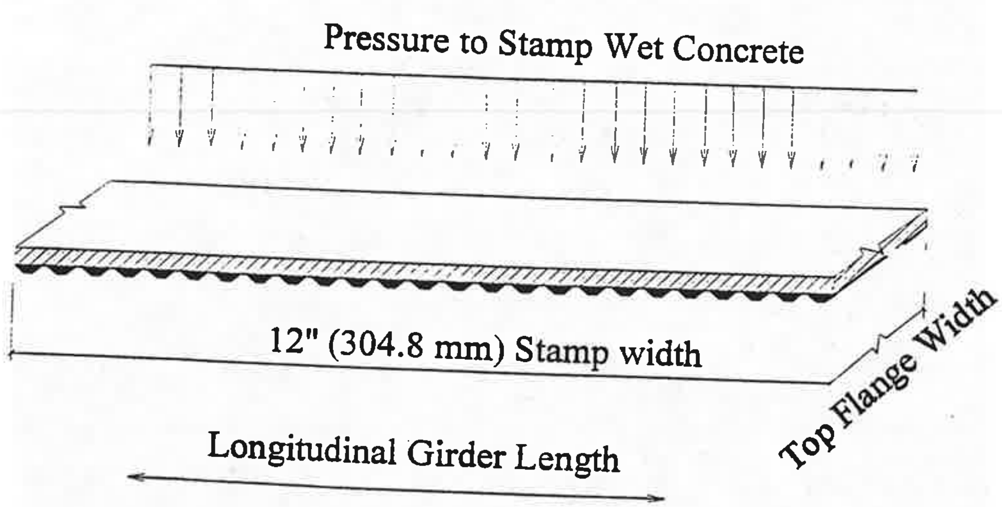


Fig. I.18 Shear Key Stamp Method

Shear Key Method used for Series 2, 4, 5, 6, 7 and 8 Specimens: A 12 in. wide roller made of hardwood with the shear keys machined into it was used to form the shear keys in the push-off specimens tested in Series 2, 4, 5, 6, 7 and 8. After the concrete lost some of its plasticity, the roller was pushed transversely across the specimen as shown in Fig. I.19. Difficulty arose in effectively maneuvering the roller around the shear connectors; however, the overall process was generally efficient. The shear key should be formed in the plastic concrete state, just prior to setting. A spray-on solution can be used to prevent accelerated moisture evaporation from the surface of the concrete.

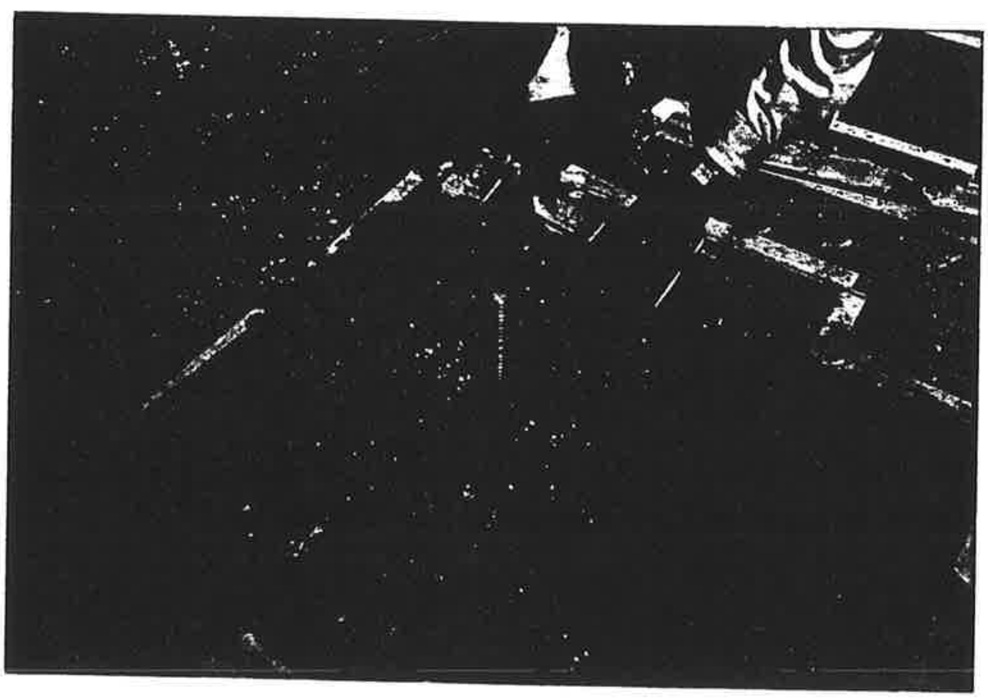


Fig. I.19 Shear Key Roller Method

Shear Key Method used for Series 10 Specimens: Steel forms were used to form the shear keys in Series 10. Details and dimensions of these shear keys are shown in Fig. I.1. This continous form was placed on the top of the girder form and the concrete was poured through the holes in the forms. Fig. I.20 shows a full scale I girder top flange formed with shear keys produced by using the steel forms.

Fig. I.20 Shear Keys on Full Scale NU Girder

Roughened Interface used for Series 3 and 9: The roughened interface used for Series 3 and 9 was applied into the top surface of the bottom slab using a stiff bristled brush. The brush was moved in a circular motion to produce a $^{1}/_{4}$ in. (6.4 mm) amplitude roughened texture in the interface. The final texture of the interface after the concrete hardened is shown in Fig. I.21.

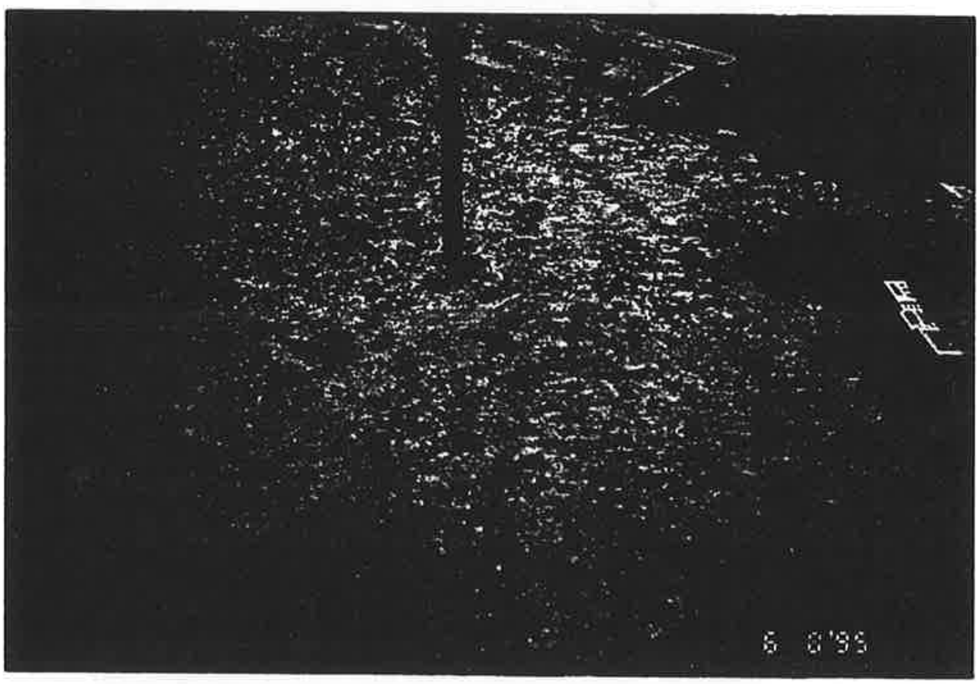


Fig. I.21 Roughened Concrete Interface

Removal and Replacement of Concrete Deck for Series 8 Specimens

The push-off specimens in Series 8 were used to evaluate the ability to remove the top concrete deck from the bottom girder using a 60 pound pneumatic jack-hammer. The concrete deck was removed from six specimens which consisted of a bonded roughened interface, debonded roughened interface, and debonded interface with shear keys. Each specimen had three #3 double leg stirrups placed at 12 in. (12.7 mm) o.c. along the length of the interface.

The removal effort was difficult for all six specimens. The debonded interface appeared to assist in the overall deck removal; however, the main effort was in removing the concrete deck around the extended stirrups. The average time for all six specimens was approximately one hour each, or $^1/_4$ hour per linear foot of girder. Concrete removal for the two specimens with the debonded shear keys, UK-T#3-a&b, was slightly easier than the debonded roughened interface of specimens UR-T#3-c&d. Damage to the shear keys was experienced from the jack-hammering process. The compressive strength of the removed concrete was approximately 7000 psi (48.3 MPa) which is not representative of the actual compressive strength of a deteriorated bridge deck. Fig. I.22 shows the interface condition of the specimens with a debonded shear key after the top concrete deck was removed.

After deck removal, plywood forms were attached to the specimen and the reinforcement was tied and set in the forms. The shear interface for specimens UK-T#3-a &b and UR-T#3-c&d, received a new application of debonding sealant. The concrete was placed and allowed to cure prior to formwork removal.

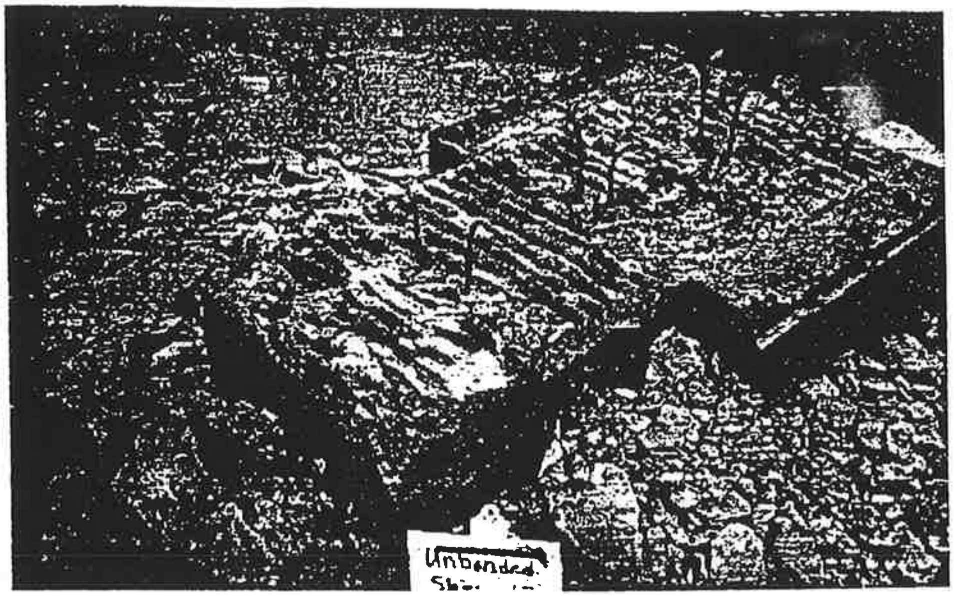


Fig. I.22 Condition of Debonded Shear Key after Deck Removal

Test Set-up and Procedures

Slip Measurements: The relative horizontal slip between the top and bottom slabs for all ultimate tests was measured using Linear Variable Differential Transducers (LVDT). The vertical slip or separation between the top and bottom slabs for all test specimens containing debonded shear key interfaces was measured with a potentiometer mounted near the center of the interface. The LVDT's and potentiometer were placed on both sides of the specimen and the slip measurements were averaged. The slip as well as the applied load were recorded at intervals of one second during the test using a multi-channel data acquisition system.

Series 1 and 2 Ultimate Test: The double shear push-off specimens were placed vertically in a steel frame and the ultimate load was applied vertically using a 400 kip

(1779 kN) hydraulic jack at the center slab. Fig. I.23 shows the test set-up. The test was stopped after the specimen failed in horizontal shear.

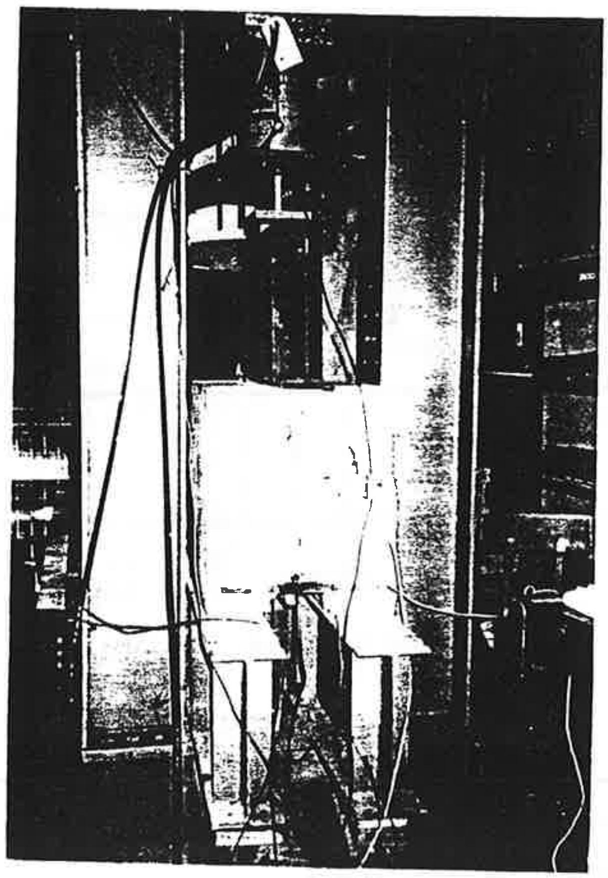
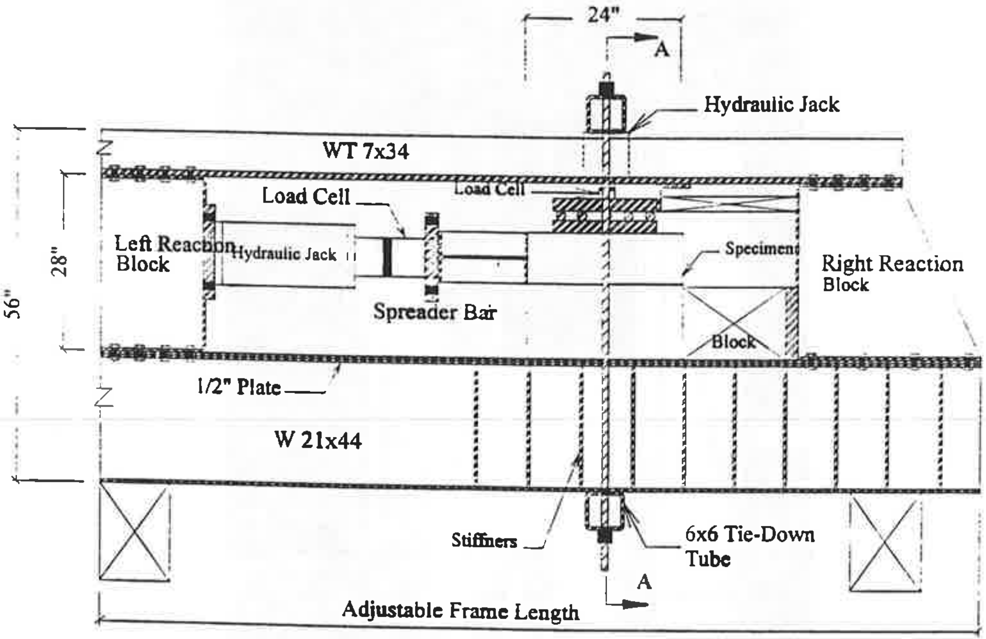
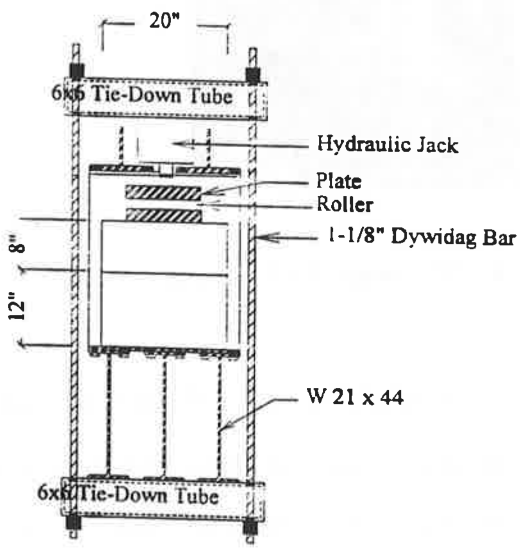


Fig. I.23 Series 1 and 2 Ultimate Test Set-up

Series 3, 4 and 9 Ultimate Test: The set-up for Series 3, 4 and 9 ultimate tests is shown in Fig. I.24. The specimens were placed in the test frame and an external force was applied both vertically and horizontally. The vertical force was held constant during the test to simulate a uniform clamping stress while the horizontal force was applied by two 400 kip (1779 kN) hydraulic jacks. The test was stopped after one inch of horizontal slip between the top and bottom slabs occurred.





Section A-A

Notes
1. Load applied horizontally to top slab.
2. All welds are 5/16" fillet welds.
3. External clamping stress applied by hydraulic jack.

(1 in. = 25.4 mm)

Fig. I.24 Series 3, 4 and 9 Ultimate Test Set-up

Series 5,6, 7, 8 and 10 Ultimate Test: The test set-up and test procedure for Series 5, 6, 7, 8 and 10 was similar to those of Series 3, 4 and 9. However, no external vertical force was applied at the top of the specimen. A detail of the set-up is shown in Fig. I.25. The length of the specimen varied depending on the series of specimens tested.

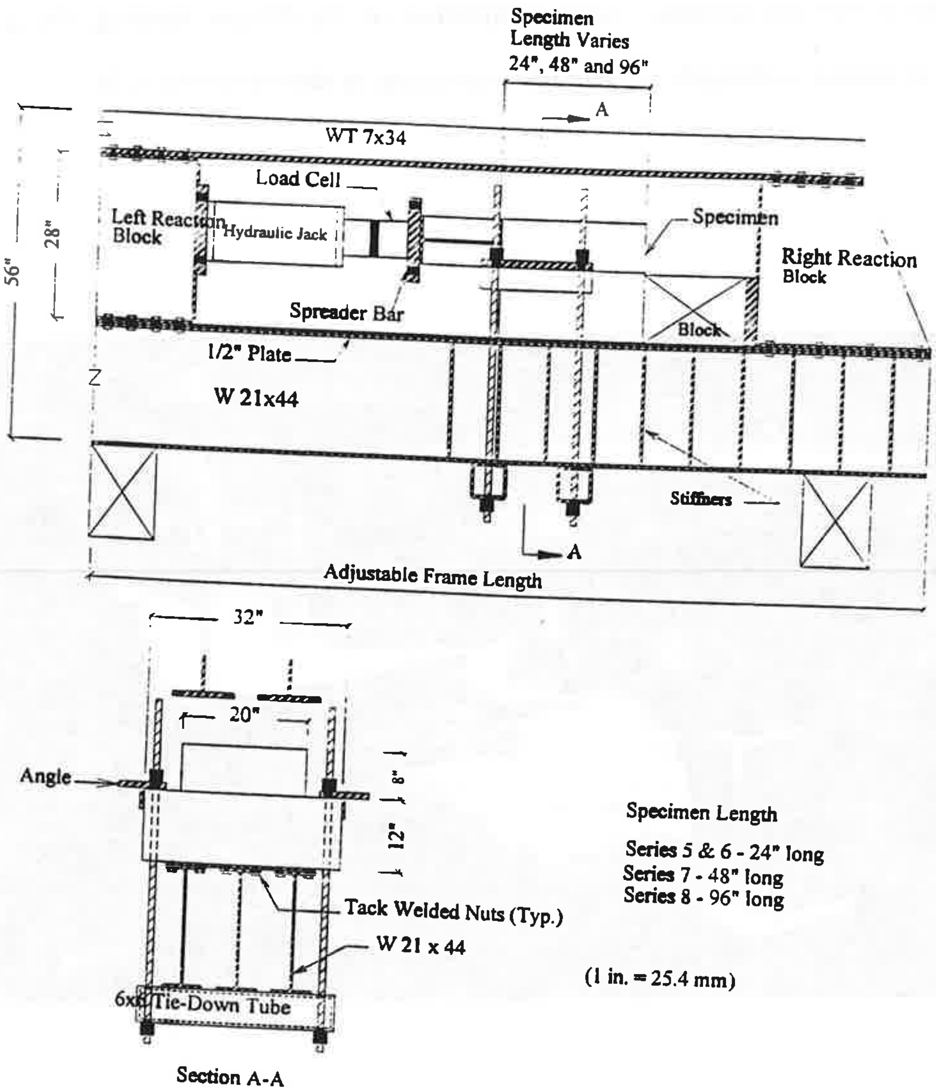


Fig. I.25 Series 5, 6, 7, 8 and 10 Ultimate Test Set-up

Fatigue Test of Specimen UK-H-1.00-a: Specimen UK-H-1.00-a was tested for fatigue strength using the unfactored service load from the example described in Appendix H. The unfactored service load was increased by 25% and applied to the specimen as a horizontal force for two million load cycles. Horizontal slip and vertical separation measurements were recorded at cycles #1 and #2,000,000. The fatigue load was applied using a closed-loop servo controlled actuator in force control at a frequency of eight cycles per second. After completion of the fatigue loading, the specimen was tested to ultimate strength. The fatigue test setup is shown in Fig. I.26.

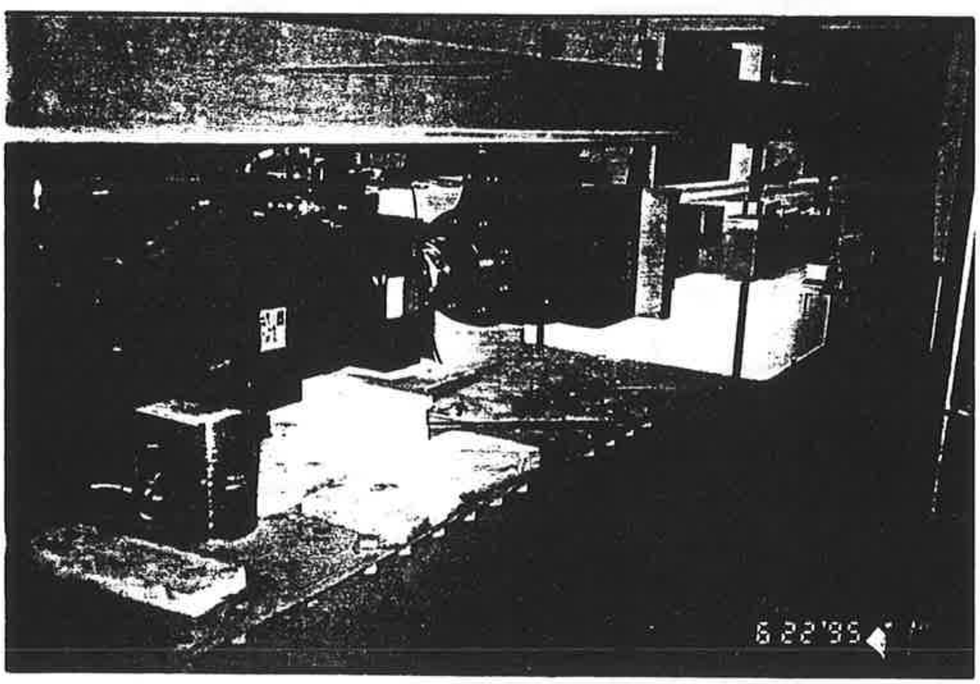


Fig. I.26 Fatigue Test of Specimen UK-H-1.00-a

Behavior of Debonded Shear Key Specimens

The push-off specimens containing a debonded shear key interface performed as expected. A definite crack occurred at the interface between the top and bottom slabs as the horizontal load increased. The level of load at which the cracking began varied depending on the type of steel connection system used. Obviously, the specimens with a low clamping stress required less force to initiate the crack and eventually cause failure. As cracking initiated, the shear key mechanism was engaged and the vertical slip was visible as the horizontal slip increased. The test results show that the horizontal shear stress increased as the relative slip between the slabs increased. Series 6 specimens, which contained reinforcing bars as shear connectors and a debonded shear key, required a larger stress to cause a crack at the interface than Series 5 specimens which had a similar clamping stress provided by a single high-strength threaded rod steel bar connector. After failure, the top slab was removed and the shear key was inspected. Fig. I.27 shows specimen UK-250, which had a clamping stress of 250 psi (1.7 MPa) , and Fig. I.28 shows a push-off specimen from Series 2. Visual inspection showed failure in about 25% of the shear keys. This was evident by the actual horizontal shearing failure of the shear keys. It was also obvious, from the condition of the interface, that the debonding agent prevented adhesion between the concrete surfaces.

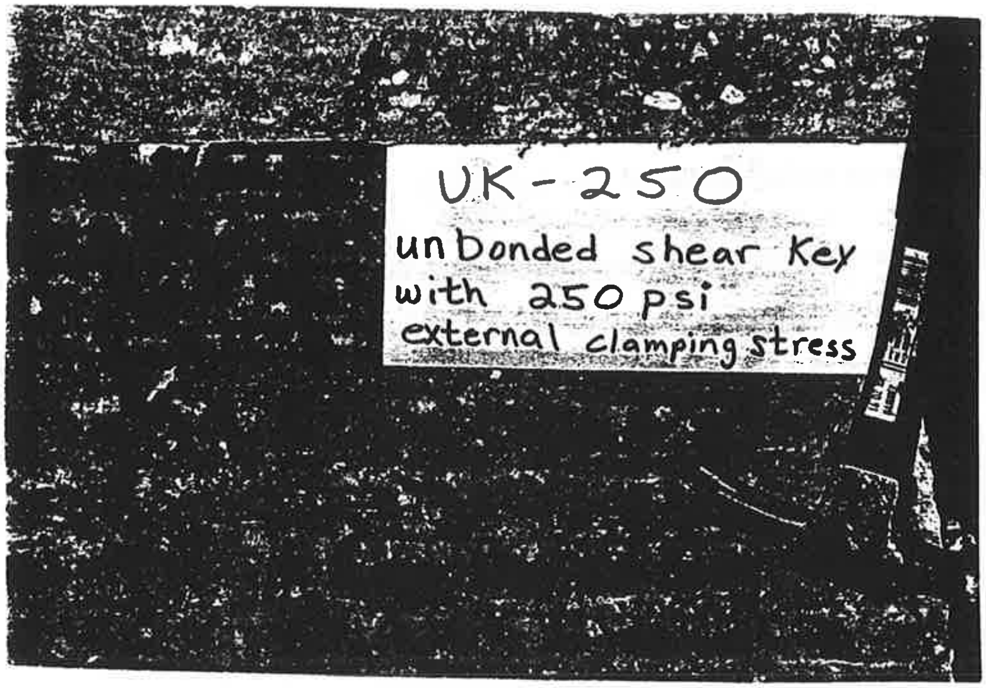


Fig. I.27 Condition of Debonded Shear Keys after Ultimate, Specimen UK-250

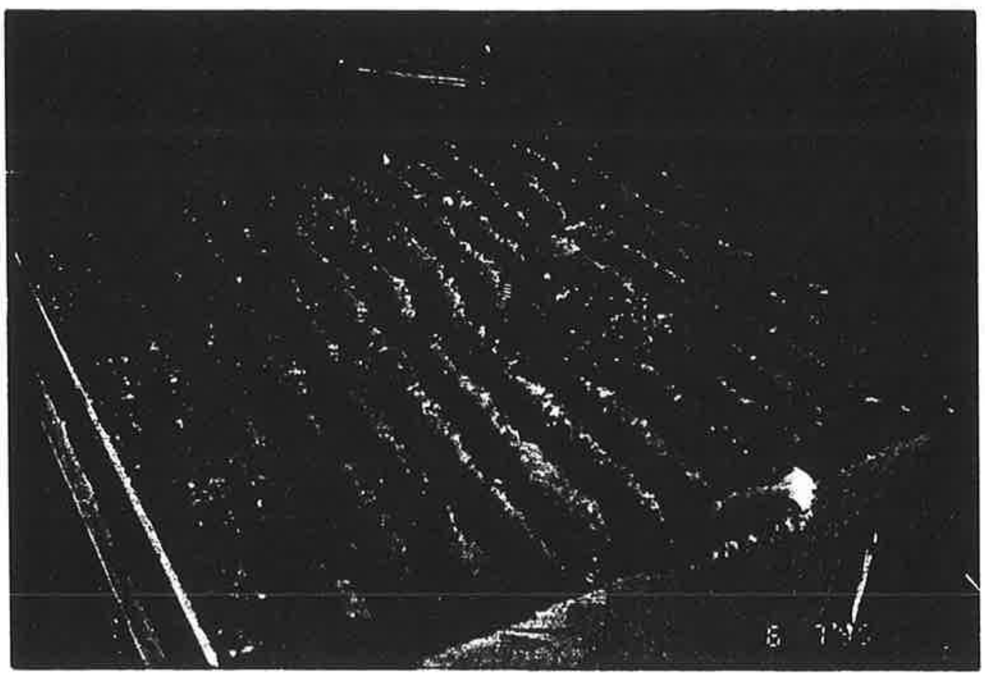


Fig. I.28 Condition of Debonded Shear Keys after Ultimate, Series 2 Specimen

Analysis of Concrete Girder-to-Deck Connection Test Results

Analysis of the results for the proposed concrete girder-to-deck connections consisted of the evaluation of several test variables. The primary investigation was the use of a debonded shear key interface formed into the concrete girder top flange. This connection, as well as several other connection schemes, were evaluated to determine the effectiveness of composite action and the ease of concrete bridge deck removal.

The test results for all specimens are given with the corresponding horizontal shear stress at certain levels of relative horizontal slip. The strength of the debonded shear key, effects of the steel connectors crossing the interface, and the fatigue strength of the shear key is discussed. The results of the proposed connection schemes which facilitate deck replacement will be discussed including a comparison to applicable design equations for horizontal shear. A recommended design equation for the debonded shear key interface will also be given.

Test Results

The results for all series of the 73 push-off specimens are shown in Table I.2. The clamping stress, $\rho_v f_y$, and concrete strengths for each specimen are given. Also included are the maximum horizontal shear stress of the interface and the shear stress at 0.005 (0.13 mm) and 0.02 in. (0.51 mm) slip. The value, μ, is the apparent coefficient of friction determined at ultimate strength. This ultimate strength could be determined at two values of relative slip: 0.005 in. (0.13 mm) as recommended by Hanson (1960), and 0.02 in. (0.51 mm) recommended by Loov et al. (1994). This coefficient is the horizontal shear stress divided by the clamping stress. Observations of the test results show that the ultimate stress is comparable to the horizontal shear stress at 0.02 in. (0.51 mm) slip.

This comparison shows that a slip value of 0.02 in. (0.51 mm), as recommended by Loov (1994), should be a slip limit to the strength of the connection.

Table I.2 Concrete Girder-to-Deck Connection Test Results

Series No.	No.	Specimen Designation	Clamping Stress (psi)	Concrete Strength (psi) top/mid/bot slab	Ultimate Stress (psi)	Ultimate Slip (inches)	Stress at slip of 0.005 in	Stress at slip of 0.02 in	μ value at 0.02 in	μ value at 0.005 in
1	1	BR-T-0.50	42	5590/7070/5590	*	-	-	-	-	-
	2	UK-H-1.00-a	134	5700/6500/4900	16	.100	4	17	0.12	0.03
	3	UK-H-1.00-b	134	7780/7920/6860	109	.052	7	68	0.81	0.05
	4	UK-H-1.00-c	134	5220/7070/5470	109	.052	7	68	0.81	0.05
	5	UK-H-1.00-d	134	6290/7020/6290	57	.020	7	57	0.43	0.05
	6	UK-H-0.75-a	77	6000/7250/6000	77	.100	5	9	1.00	0.07
	7	UK-H-0.75-b	77	5090/7070/6450	79	.100	6	13	1.03	0.08
2	8	UK-H-1.00-a	134	5000/5000/5000	255	.061	65	174	1.90	0.49
	9	UK-H-1.00-b	134	5000/5000/5000	249	.069	85	165	1.86	1.23
	10	UK-H-1.00-c	267	5000/5000/5000	347	.045	192	280	0.77	1.05
	11	UK-H-1.00-d	267	5000/5000/5000	261	.004	180	@	0.98	0.67
	12	UK-H-0.75-a	154	5000/5000/5000	306	.017	172	275	1.99	1.12
	13	UK-H-0.75-b	154	5000/5000/5000	286	.055	128	240	1.86	0.83

* unable to break specimen, no data recorded
@ specimen failed before slip value reached

Legend:

B = bonded surface
R = roughened surface
K = shear key interface
U = debonded surface
S = smooth surface

H = threaded bar
T = extended stirrups
N = new connector

a = specimen 1
b = specimen 2
c = specimen 3
d = specimen 4
e = specimen 5

(1 in. = 25.4 mm)

Table I.2 (continued)

Series No.	No.	Specimen Designation	Clamping Stress (psi)	Concrete Strength (psi) top/bottom slab	Ultimate Stress (psi)	Ultimate Slip (inches)	Stress at slip of 0.005 in	Stress at slip of 0.02 in	μ value at 0.020 in	μ value at 0.005 in
3	14	US-0	0	6000/5200	111	.041	75	104	-	-
	15	US-50	50	6000/5200	261	.006	259	261	5.22	5.18
	16	US-100	100	6000/5200	366	.008	330	335	3.66	3.30
	17	US-125	125	6000/5200	298	.010	258	282	2.38	2.06
	18	US-150	150	6000/5200	212	.022	175	210	1.41	1.17
	19	US-175	175	6000/5200	220	.005	218	191	1.26	1.25
	20	US-200	200	6000/5200	354	.013	325	338	1.77	1.63
	21	US-250	250	6000/5200	237	.124	191	220	0.95	0.76
4	22	UK-0	0	5900/5900	272	.003	261	240	-	-
	23	UK-100	100	5900/5900	491	.012	415	465	4.91	4.15
	24	UK-125	125	5900/5900	254	.009	274	253	2.03	2.19
	25	UK-150	150	5900/5900	624	.015	402	620	4.16	2.68
	26	UK-175	175	5900/5900	534	.009	452	520	3.05	2.58
	27	UK-200	200	5900/5900	650	.018	426	640	3.25	2.13
	28	UK-210	210	5900/5900	684	.021	430	684	3.26	2.05
	29	UK-250	250	5900/5900	687	.012	494	652	2.75	1.98
5	30	UK-H-0.75-a	92	5900/5900	330	.055	275	332	3.59	2.99
	31	UK-H-0.75-b	92	5900/5900	307	.048	100	247	3.34	0.92
	32	UK-H-1.00-a	160	5900/5900	287	.098	142	211	1.79	1.13
	33	UK-H-1.00-b	160	5900/5900	331	.015	259	330	2.07	0.62
	34	UK-H-1.25-a	255	5900/5900	530	.106	105	330	2.08	2.43
	35	UK-H-1.25-b	255	5900/5900	357	.037	218	314	1.40	0.86
	36	UK-H-1.50-a	368	5900/5900	544	.110	180	357	1.48	0.49
	37	UK-H-1.50-b	368	5900/5900	546	.037	229	460	1.48	0.62

Table I.2 (continued)

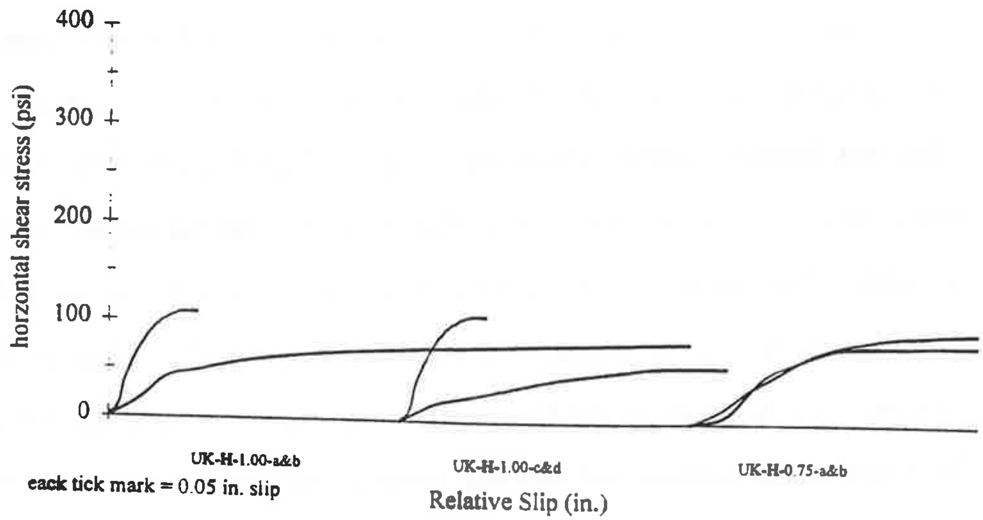
Series No.	No.	Specimen Designation	Clamping Stress psi)	Concrete Strength (psi) top/bottom slab	Ultimate Stress (psi)	Ultimate Slip (inches)	Stress at slip of 0.005 in	Stress at slip of 0.02 in	μ value at 0.02 in.	μ value at 0.005 in
6	38	UK-T#4-a	100	6000/5200	295	.026	170	286	2.95	1.70
	39	UK-T#4-b	100	6000/5200	253	.217	120	180	2.53	1.20
	40	UK-T#4-c	150	6000/5200	471	.017	309	471	3.14	2.06
	41	UK-T#4-d	150	6000/5200	487	.025	309	482	3.25	2.06
	42	UK-T#5-a	230	6000/5200	571	.035	360	547	2.48	1.57
	43	UK-T#5-b	230	6000/5200	652	.044	388	580	2.84	1.69
	44	UK-T#6-a	330	6000/5200	588	.046	301	510	1.78	0.91
	45	UK-T#6-b	330	6000/5200	726	.070	385	526	2.20	1.17
7	46	UK-H1.00-a	16	7400/7900	98	.004	93	90	6.13	0.17
	47	UK-H1.00-b	16	7400/7900	119	.028	79	85	7.44	0.20
	48	BR-H1.00	40	7400/7900	237	.003	234	220	5.93	5.85
	49	BR-H-0.50	20	7400/7900	232	.015	75	231	11.60	3.75
	50	UK-H-1.50-a	92	7400/7900	125	.002	120	113	1.36	1.30
	51	UK-H-1.50-b	92	7400/7900	test failed	-	-	-	-	-
	52	UR-H-1.50-a	92	7400/7900	test failed	-	-	-	-	-
	53	UR-H-1.50-b	92	7400/7900	data lost	-	-	-	-	-
8	54	BR-T#3-a	41	4500/7000	355	.002	350	355	8.66	8.54
	55	BR-T#3-b	41	4500/7000	281	.025	192	269	6.85	4.68
	56	BR-T#3-c	41	4500/7000	368	.0125	277	365	8.98	6.76
	57	BR-T#3-d	41	4500/7000	440	.005	440	425	10.73	10.73
	58	UK-T#3-a	41	4500/7000	173	.004	173	162	4.22	4.22
	59	UK-T#3-b	41	4500/7000	data lost	-	-	-	-	-
	60	UR-T#3-a	41	4500/7000	172	.017	129	170	4.20	3.15
	61	UR-T#3-b	41	4500/7000	124	.013	112	124	3.02	2.73
9	62	BS-0	0	5900/5900	328	.002	320	308	-	-
	63	BS-100	100	5900/5900	650	.004	645	610	6.50	6.45

Table I.2 (continued)

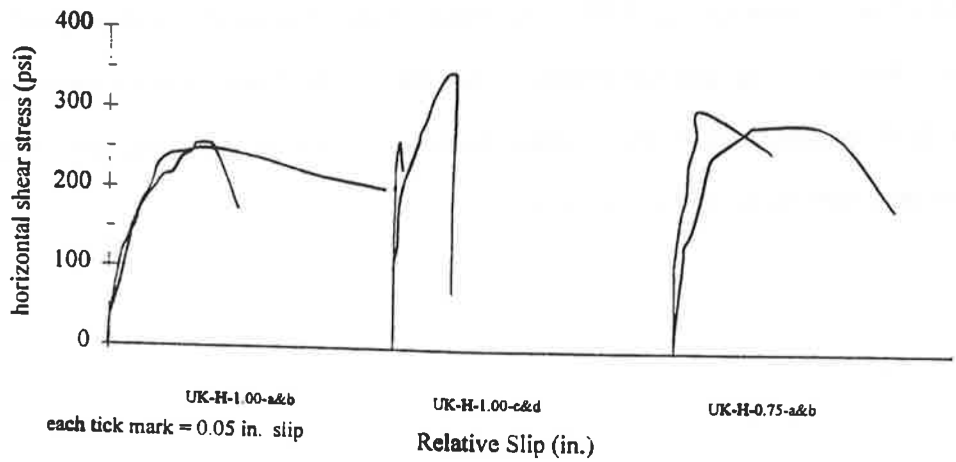
Series No.	No.	Specimen Designation	Clamping Stress (psi)	Concrete Strength (psi) top/bottom slab	Ultimate Stress (psi)	Ultimate Slip (inches)	Stress at slip of 0.005 in	Stress at slip of 0.02 in	μ value at Slip ≤ 0.02 in
10	64	UK-N-Gr.60-a	97	4210/6200	132	.021	99	132	1.36
	65	UK-N-Gr.60-b	97	4210/6200	116	.002	broke	broke	1.20
	66	UK-N-Gr.60-c	97	4210/6200	110	.003	broke	broke	1.13
	67	UK-N-Gr.60-d	97	4210/6200	104	.008	98	broke	1.00
	68	UK-N-Gr.60-e	97	4210/6200	-	-	-	-	-
	69	UK-N-Gr.100-a	194	4210/6200	48	.009	46	broke	-
	70	UK-N-Gr.100-b	194	4210/6200	72	.0.16	61	broke	-
	71	UK-N-Gr.100-c	194	4210/6200	-	-	-	-	-
	72	UK-N-Gr.100-d	194	4210/6200	78	.036	62	67	-
	73	UK-N-Gr.100-e	194	4210/6200	65	.024	49	56	-

Strength of the Debonded Shear Key Interface

Evaluation of Methods to Form the Shear Key Interface System: The three methods evaluated for forming the shear key interface in the top flange were a stamp, roller, and forming method. The stamp required a high force to form the shear key in the freshly placed concrete. This force also needed to be maintained until the concrete hardened. The quality of the shear key was not as consistent as the roller or forming methods because of the inability to sufficiently press the stamp manually into the concrete. A comparison of the three series of specimens constructed, Series 1, 2, and 10 show that stamp method and forming method had lower strength as compared to the strength of the rolled shear key interface. The results of the push-off tests for Series 1, 2 and 10 are shown in Fig. I.29. The roller method is an efficient method of placing the shear key in the girder top flange, however it was found to be a very time consuming method as compared to the forming method. Forming the shear keys proved to be the best way for placing the shear keys.

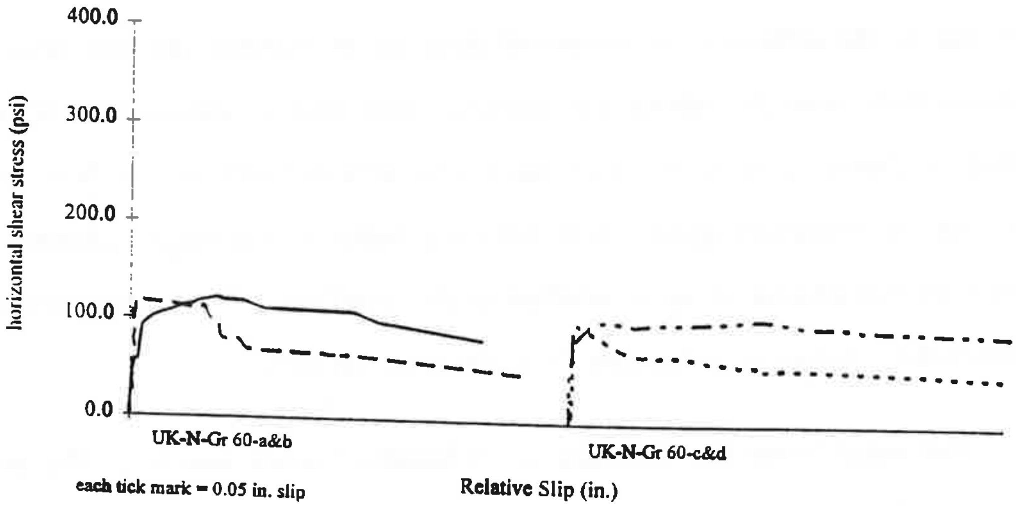


Series 1 Stamped Shear Key Surface



Series 2 Shear Key Rolled into Interface

Fig. I.29 Comparison of Methods to Form Shear Key



Series 10 Formed Shear Key

Fig. I.29(Continued) Comparison of Methods to Form Shear Key

Effect of Debonding Agent: The effectiveness of the debonding agent applied to the interface is evident from the results shown in Fig. I.30. The push-off specimen results for Series 3 and 9, which had similar clamping stresses, show the effect of the debonding agent due to the difference in horizontal shear stress between the two series. Both interfaces were smooth without any crossing steel and an external clamping stress applied. It should be noted that a certain degree of initial adhesion or bond is present even with the debonding agent. A contributing factor for the initial adhesion may be affected by the amount of agent applied to the interface. Greater amounts of agent applied to the interface may decrease the initial adhesion effect.

Debonded Shear Key Interface vs. Debonded Smooth Interface: The horizontal shear strength for the shear key interface formed into the girder by the roller method is shown in Fig. I.31. The clamping stress was applied by an external force at the top of the specimen. This may be considered an hypothetical connection. A best fit line through the data points for both series shows the effectiveness of the shear key. The difference between the data points is the shear key effect. The horizontal shear stress for the debonded smooth interface is generally constant for all levels of clamping stress, while the shear stress for the debonded shear key interface increases as the clamping stress increases. This indicates that the shear key mechanism is resisting horizontal shear force generated by composite action.

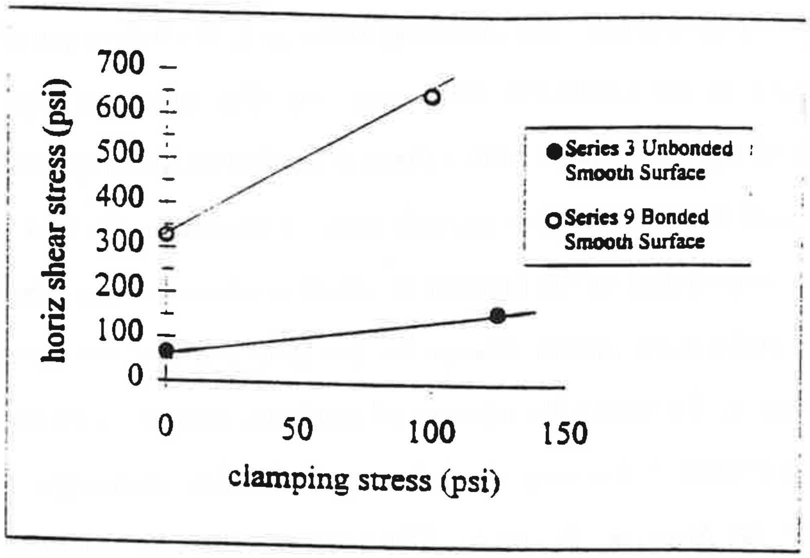


Fig. I.30 Effect of Debonding Sealant

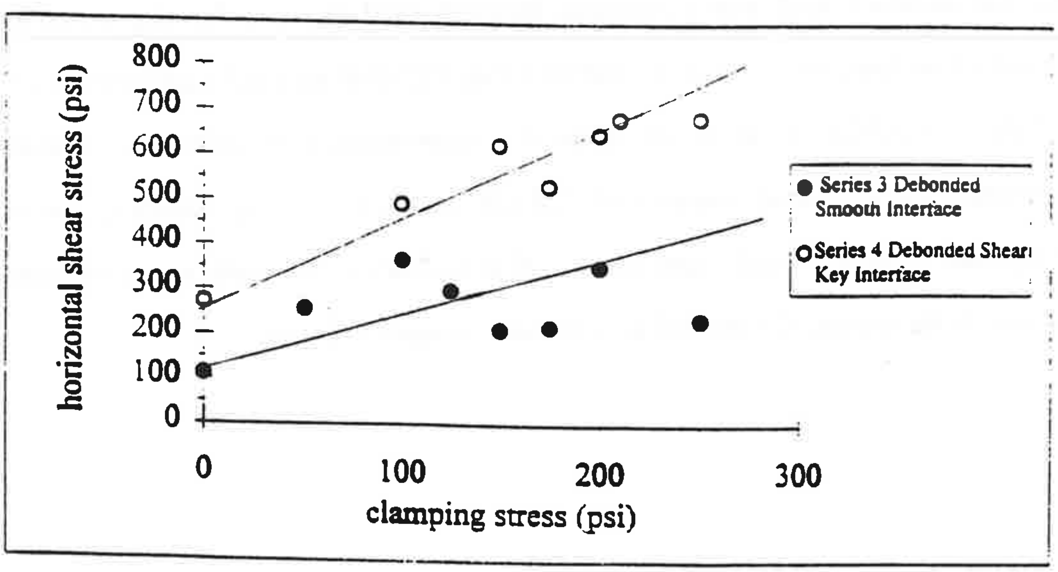
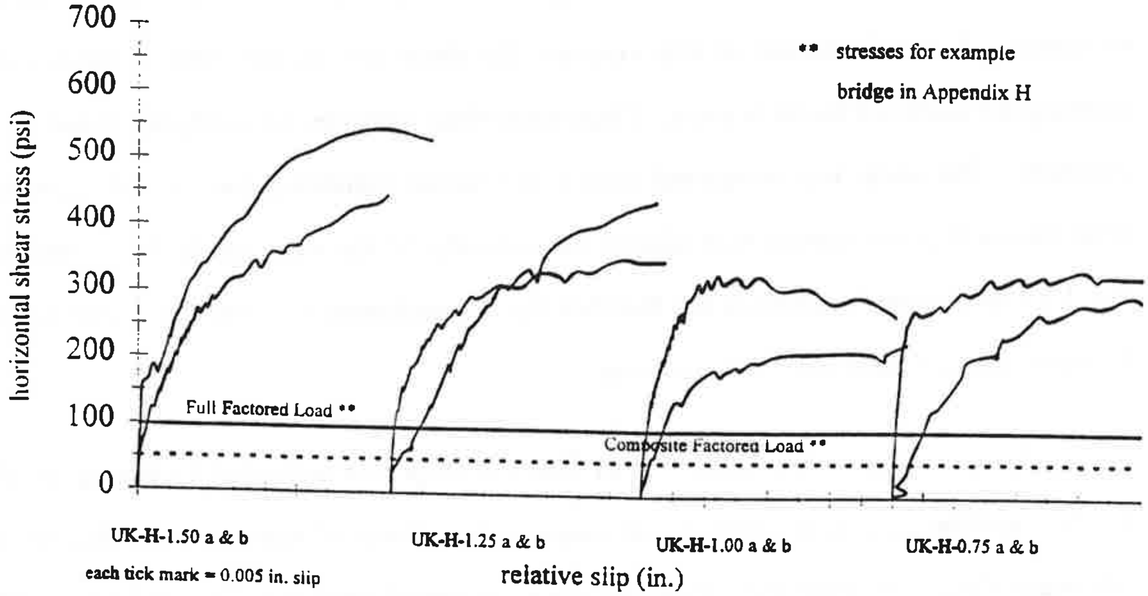


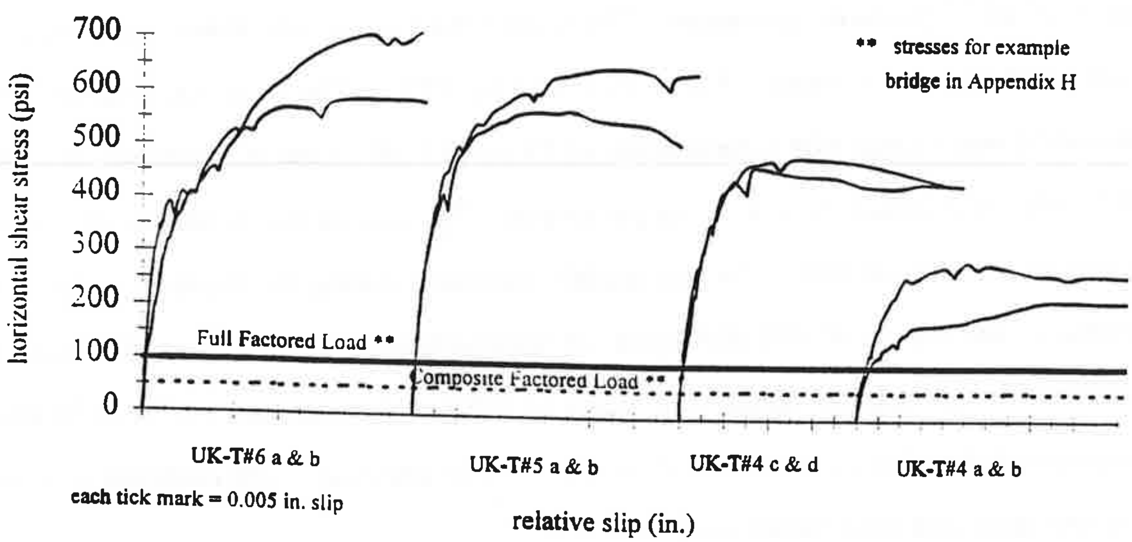
Fig. I.31 Strength of Debonded Shear Key vs. Debonded Smooth Interface

Steel Connectors Crossing the Interface: Two types of steel connectors crossing the shear key interface were used: high strength threaded rods (f_y = 100 ksi), and Gr. 60 extended reinforcement. The clamping stress, $\rho_v f_y$, for both types of bars was similar. A comparison of the horizontal shear stress vs. slip, shown in Fig. I.42, shows that the strength of the connections with extended reinforcement is approximately 25% to 30% greater than that of the high-strength bars. Obviously, the area of steel crossing the interface contributed to an increase in shear resistance. The clamping stress for both Series 5 and 6 were similar, except for the yield stress of the steel ties. The lower the yield stress is, the higher the amount of steel area required. The bar deformations on the extended reinforcement may cause better mechanical anchorage in the concrete which develops the force in the bars. The relatively smaller amounts of shear resistance developed by the high-strength threaded rods may be attributed to local concrete crushing around the bar due to fewer bars crossing the interface.

The horizontal shear strength or resistance provided by both connections exceeds the full factored load and composite factored load stresses for the example bridge described in Appendix J. The AASHTO (15th Ed.) full factored load consists of 1.3[DL + SDL + 1.67(LL + I)] or all loads the superstructure experiences. However, the composite factored load consists of 1.3[SDL + 1.67(LL + I)], which is the force the composite section would experience. The resistance for both types of connections exceeds both stresses for the full factored and composite loads.



Series 5 High Strength Bolts



Series 6 Extended Reinforcement

Fig. I.42 Comparison of Steel Connectors Crossing the Debonded Shear Key Interface

Horizontal Slip vs. Vertical Separation: The mechanism of the shear key forces vertical separation between the deck and girder to occur as the relative horizontal slip increases. A small amount of slip engages the shear key as the steel connector bars crossing the interface begin to yield. This mechanism provides the clamping stress on the interface. The shear key is formed with a 45° slope; therefore, the vertical separation between the top and bottom slab should theoretically be the same as the horizontal slip. Fig. I.43 shows confirmation of the fact that the slip and separation vary at approximately the same rate for two push-off specimens.

Fatigue Effects: Specimen #46 or UK-H-1.00-a was subjected to fatigue loading for two million cycles in order to investigate the effects of repetitive loading on the debonded shear key interface. This specimen contained three headless and one headed high-strength threaded rods crossing the interface. This resulted in the lowest clamping stress of all 73 push-off specimens. The service load stress was determined from the example bridge in Appendix J and increased by 25%. The load was applied in a sinusoidal wave shape with a mean stress of 15 psi (0.1 MPa) and an amplitude of 30 psi (0.2 MPa), at a frequency of 8 cycles per second. Fig. I.44 shows the horizontal slip for cycles #1 and #2,000,000. The slip slightly increased during the duration of the test; however, the slip at #2,000,000 cycle of approximately 0.00015 in. (.004 mm) is considerably less than the critical slip valves of 0.005 (0.13 mm) and 0.02 in. (0.51 mm) recommended by Hanson (1960) and Loov (1994), respectively. The clamping stress of this specimen was considerably low.

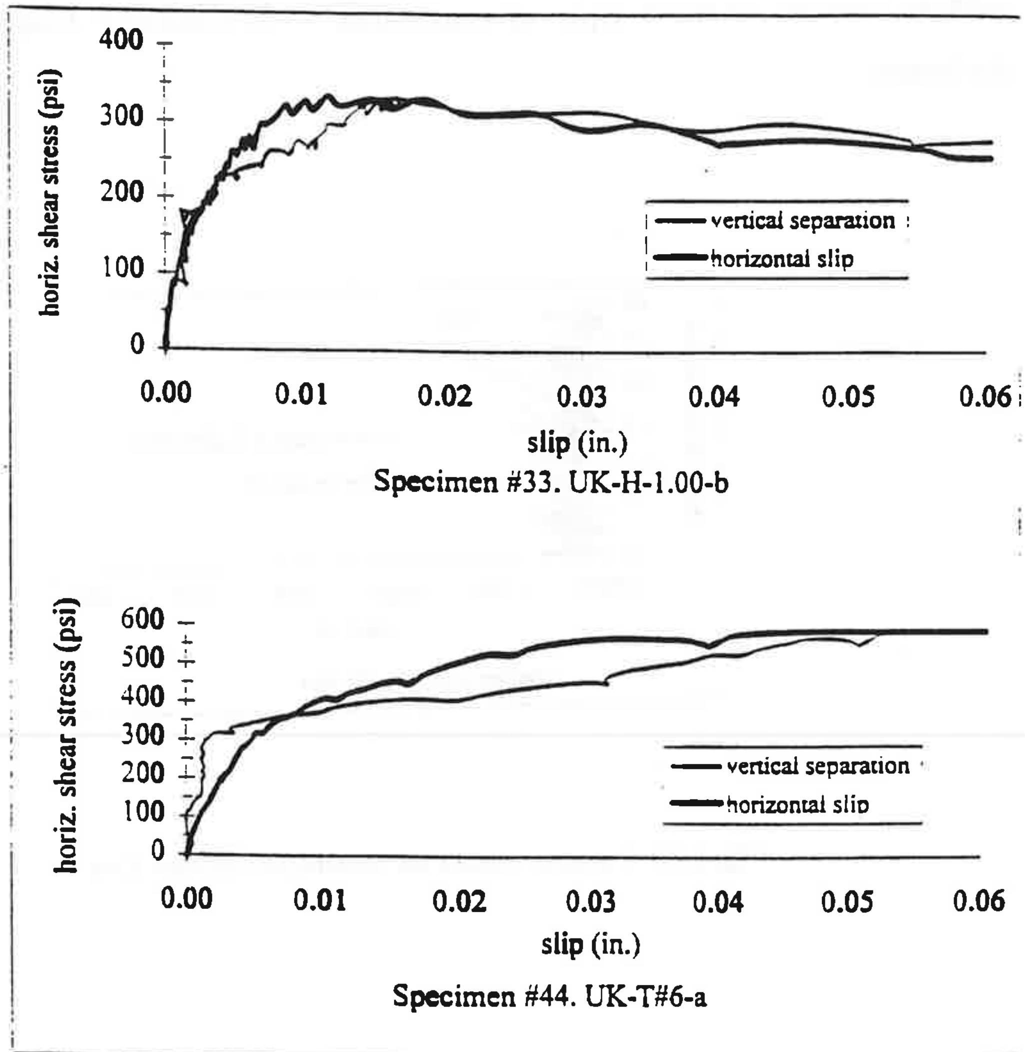


Fig. I.44 Horizontal Slip vs. Vertical Separation for Debonded Shear Key

Even after the fatigue test was completed, this specimen produced similar results under ultimate strength as an identical specimen (UK-H-1.00-b) that was not subjected to fatigue. Therefore, it could be concluded that fatigue will have no effect on service or ultimate capacity of these types of connections. No additional fatigue tests were performed.

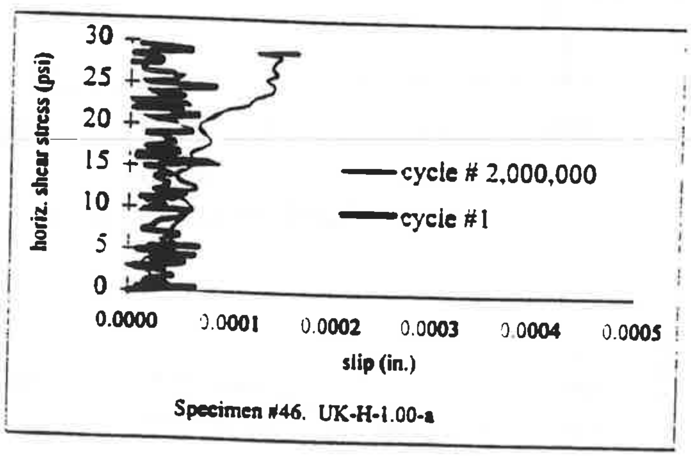


Fig. 1.44 Fatigue Effect on Debonded Shear Key

Effect of Wide Spacing Between Connectors: Series 7 push-off specimens consisted of connection schemes which utilized a wide spacing between connectors and also a combination of headed and headless bolts combined with either a bonded roughened, debonded roughened, or debonded shear key interface. By increasing the distance between connectors, these proposed connection systems may facilitate deck replacement. The results of the Series 7 ultimate push-off tests are shown in Fig. I.45. The full and composite factored load stresses for the example bridge of Appendix J compares actual stresses to the resistance of the composite connection. The capacity of specimens with the bonded roughened interface obviously exceeded the factored stresses; however, specimens UK-H-1.50 and UK-H-1.00 did not necessarily satisfy the full factored stresses. This could be due to the small amount of clamping stress and the wide spacing between the connectors, coupled with the use of a debonded interface. It is obvious that the concrete adhesion between the two surfaces tremendously affects the horizontal shear resistance of the connection even though a wide spacing between the connectors was provided. AASHTO limits the spacing between connectors to 24 in. (609.6 mm); however, the steel connectors do not necessarily contribute to the shear resistance in bonded systems until the bond is broken. Connectors placed at a spacing wider than 24 in. (609.6 mm), with the bonded roughened interface may be applicable to design however, may not gain acceptance by designers and bridge owners. Based on these observations, it is recommended to standardize the spacing to a maximum of 24 in. (609.6 mm) throughout the bridge span in order to facilitate deck replacement.

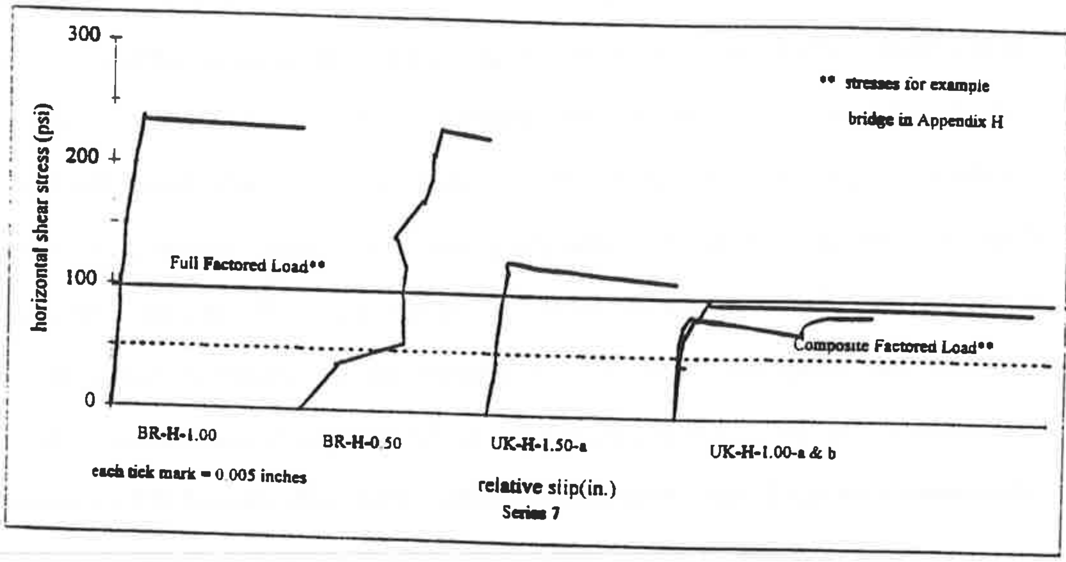


Fig. I.45 Rapid Deck Replacement Connection Schemes

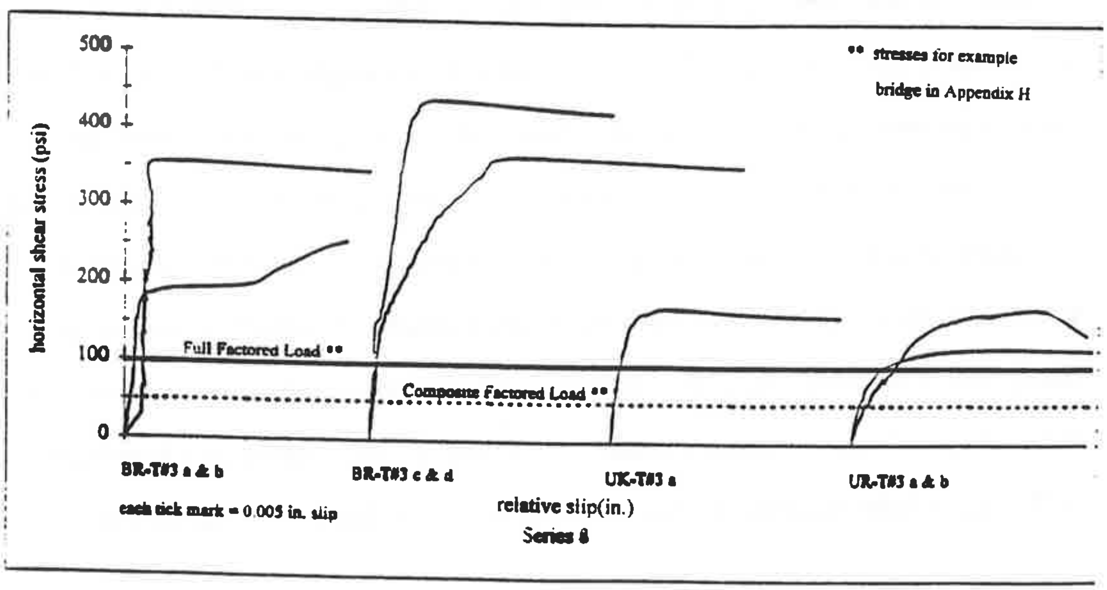


Fig. I.46 Effect of Deck Replacement on Connection Schemes

Effect of Deck Replacement on Connection Interface: The results of Series 8 push-off specimens in Fig. I.46, show the effect of replacing the concrete deck on the connection strength. Specimens BR-T#3-a&b are for baseline comparison including the full and composite factored load stress for the example bridge of Appendix J. Comparison of those specimens with a bonded roughened interface with the baseline specimens show that removal and replacement had minimal effect on the behavior of the connection. The specimens with a debonded interface, UK-T#3-a and UR-T-#3-a&b, both exceeded the full and composite factored stresses. Deck removal methods by jack-hammering did cause some damage to the debonded shear key interface.

Comparison to Design Equations

AASHTO LRFD Design Equation: AASHTO LRFD (1995) design equation (I.1) is applicable to the debonded shear key interface since it is based on the shear friction theory.

$$V_n = cA_{cv} + \mu[A_{vf} f_y + P_c] \quad \text{(I.1)}$$

$$V_n \leq 0.2 f'_c A_{cv}$$

$$V_n \leq 0.8 A_{cv}$$

Where	c	=	cohesion coefficient
	A_{cv}	=	contact area, in^2
	μ	=	friction coefficient
	A_{vf}	=	area of vertical shear reinforcement, in^2
	f_y	=	steel yield stress

P_c = net compressive force normal to shear interface, lbs

The cohesion term c, is theoretically equal to zero since the interface is debonded. The test results are shown in Fig. I.47 with the LRFD equation applied with a value of μ = 1.4, c = 0, and P_c = 0. This shows that LRFD equation is relatively conservative for a debonded interface.

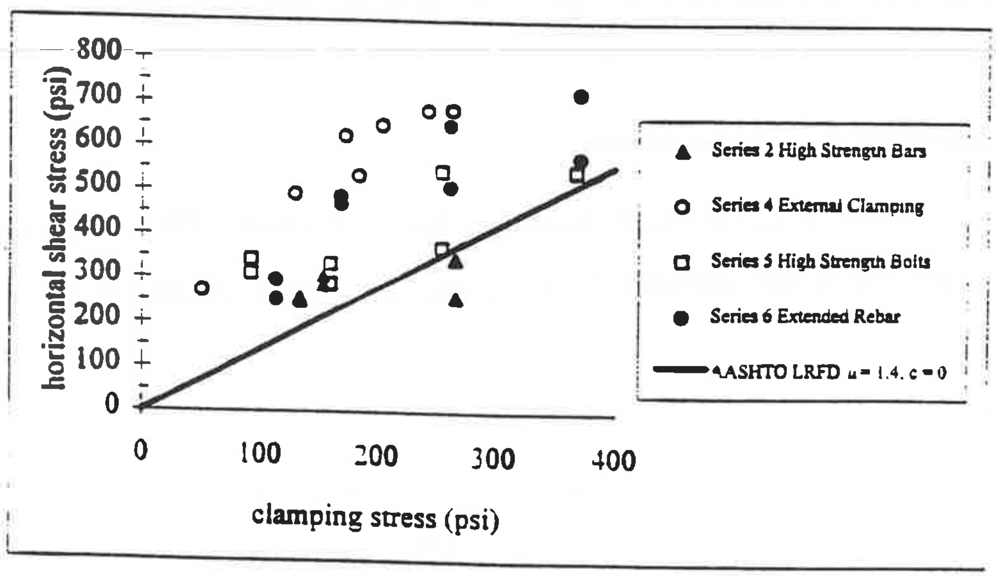


Fig. I.47 Comparison of Debonded Shear Key with AASHTO LRFD Equation

Debonded Shear Key Interface Compared to Loov's Equation (1994): Although concrete strength was not included as a main test parameter, Loov's equation (I.2) could be compared to the results of Series 2, 5 and 6 push-off tests which had a debonded shear key interface.

$$v_u = k\sqrt{15 + \rho_v f_y (f'_c)} \quad (psi) \qquad \text{(I.2)}$$

The constant k, could be taken as 0.5 for composite construction and the value of 15 under the radical can be ignored if a debonded interface is used. Therefore, the equation could be rewritten as (I.3):

$$v_u = 0.5\sqrt{\rho_v f_y (f'_c)} \quad (psi) \qquad \text{(I.3)}$$

Test results including Loov's equation (I.2) are shown in Fig. I.48. The results show that Loov's equation is generally unconservative for a debonded shear key interface with a low clamping stress. However, it has been shown from the literature review that this equation is applicable for both bonded and debonded interfaces with high clamping stresses.

Recommended Design Equation for Debonded Shear Key

The test results and literature review have confirmed that the shear friction theory is the recommended method for determining the resistance of the debonded shear key. It neglects the effect of bond and provides the clamping stress from the steel reinforcement. The shear friction equation is conservative for low clamping stresses which would be applicable to horizontal shear design for concrete girders.

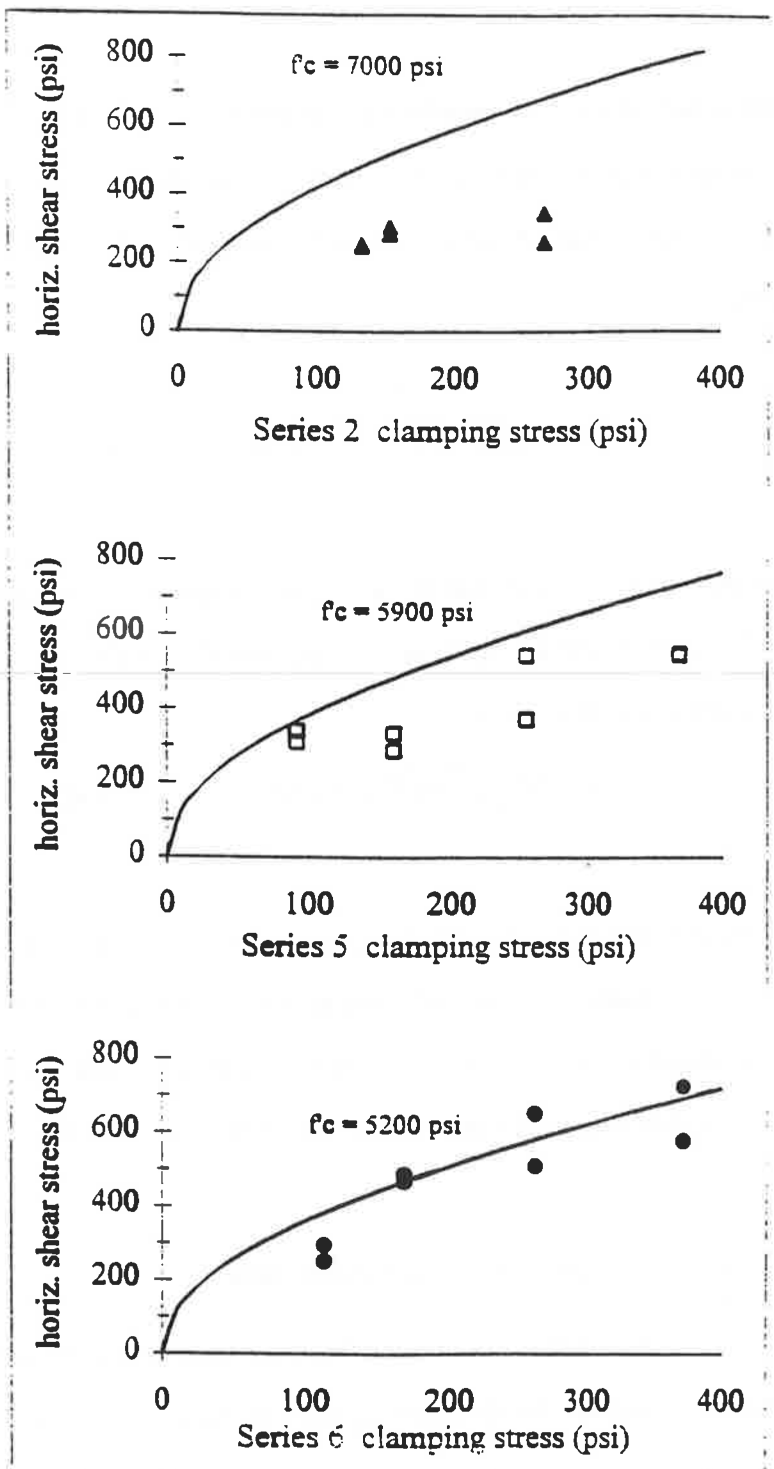


Fig. I.48 Debonded Shear Key Comparison with Loov's Equation (1994)

Friction Coefficient for Smooth Surfaces: Series 3 push-off specimens were used to determine a coefficient of friction between the smooth surfaces on the 45^0 slope of the debonded shear keys. The results shown in Fig. I.49 are inconclusive with results from previous research. Shiakh (1978) recommended a friction coefficient, $\mu = 0.4$ for smooth concrete surfaces. A possible contributing factor to the inconsistent results could be variations in the smooth surface of the test specimens or errors in the test set-up. The value of horizontal shear stress was determined when the slabs began sliding across one another. This value was not determined at the peak stress, but rather the stress needed for the top slab to slide across the bottom slab under a predetermined external clamping stress.

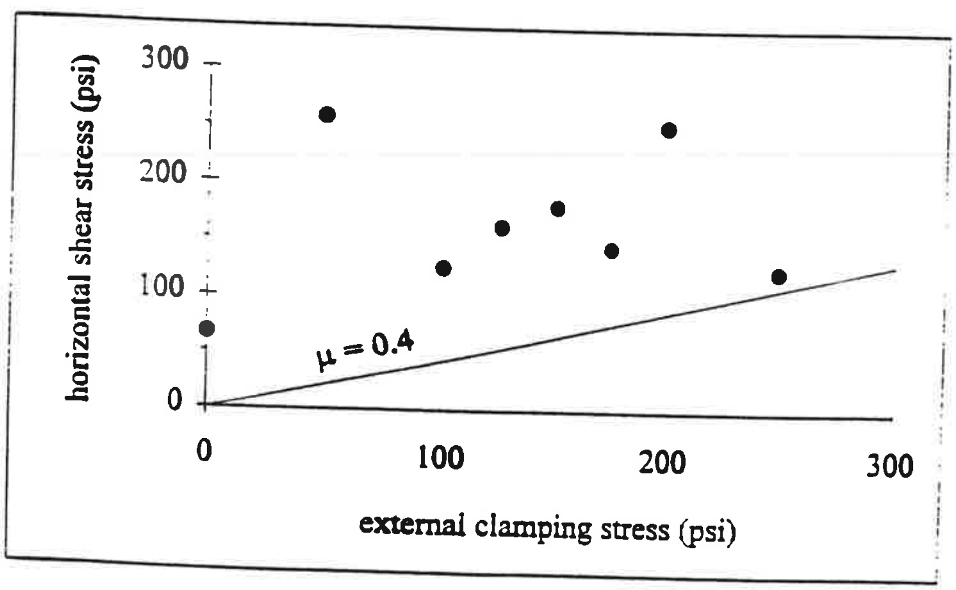


Fig. I.49 Friction Coefficient for Smooth Surfaces

Friction Coefficient of Debonded Shear Key Interface: In order to limit the slip in the design of the debonded shear key interface, a conservative approach could be used. A value of $\mu = 1.0$ from the best fit line corresponding to a slip of 0.005 in. (0.13 mm) as shown in Fig. I.50 may be appropriate. For ultimate design, $\mu = 1.4$ represents the lower bound of the test results for slip at ultimate and 0.02 in. (0.51 mm). The values of μ are very similar between ultimate and 0.02 in. (0.51 mm) slip as recommended by Loov et al. (1994). The shear friction theory is recommended as a design equation for the debonded shear key with $\mu = 1.0$, which limits the design to a slip of 0.005 in. (0.13 mm).

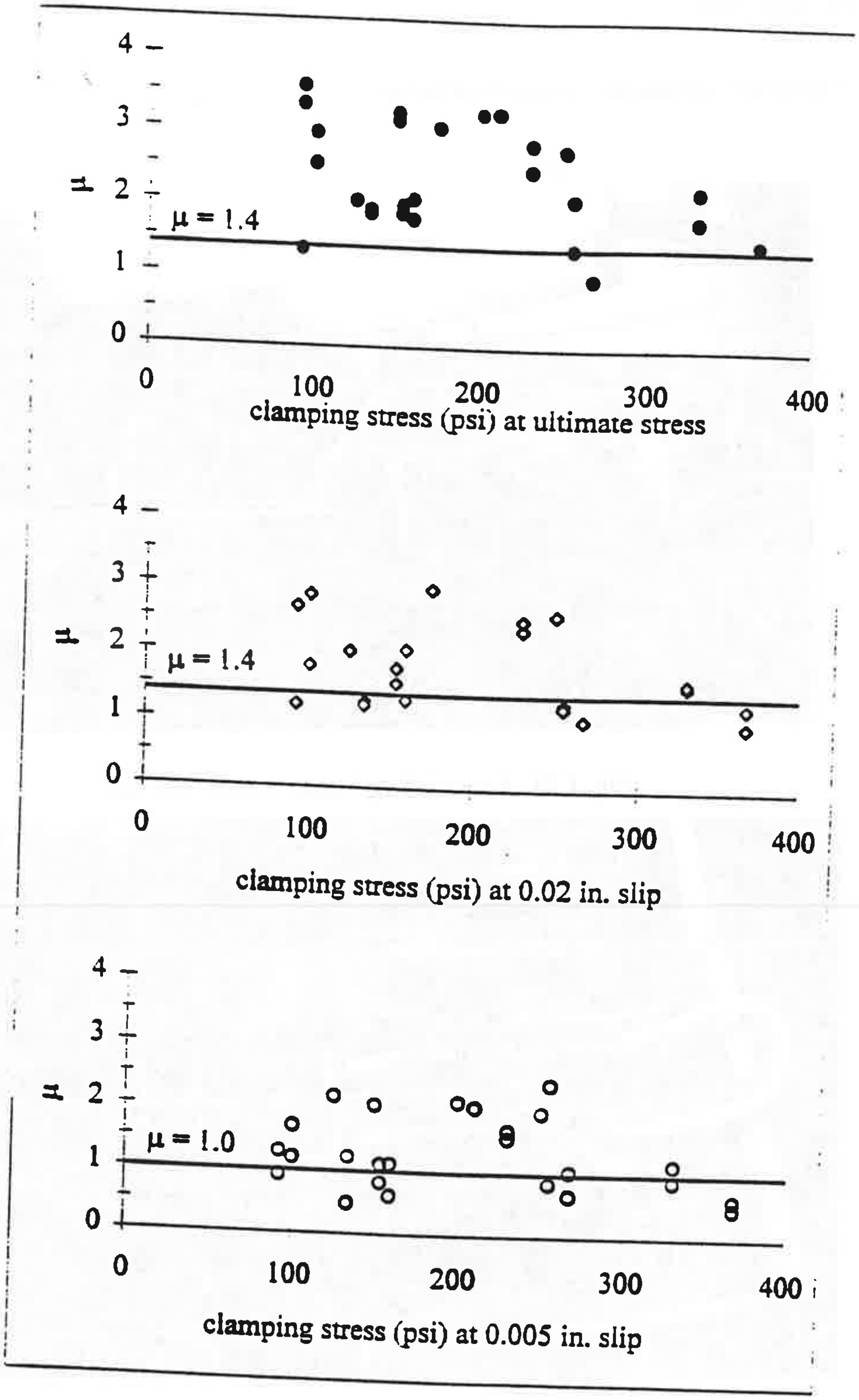


Fig. I.50 Friction Coefficient for Debonded Shear Key Interface

Full Scale Test

Fabrication of Test Specimen and Test Setup

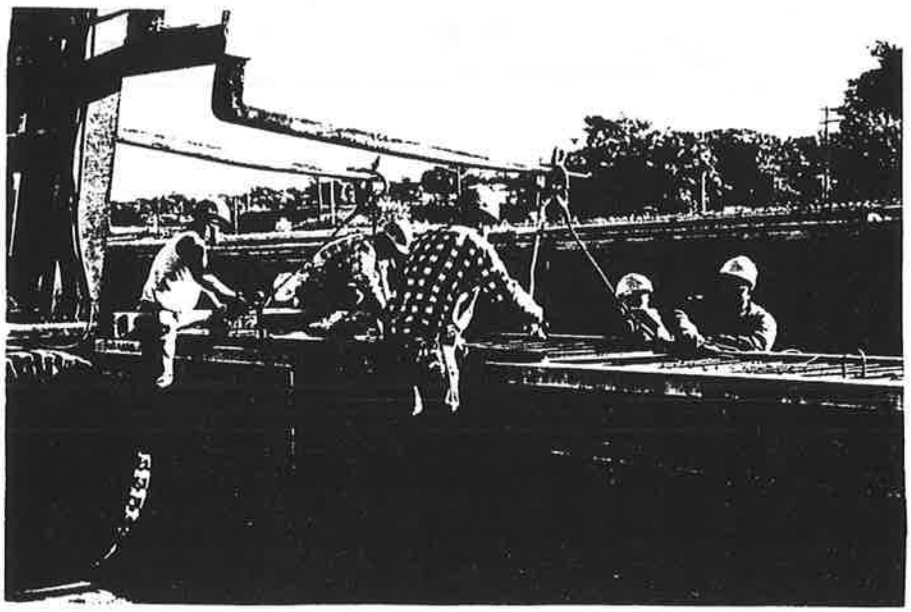


Fig. I.51 Assembling Shear Key's Forms

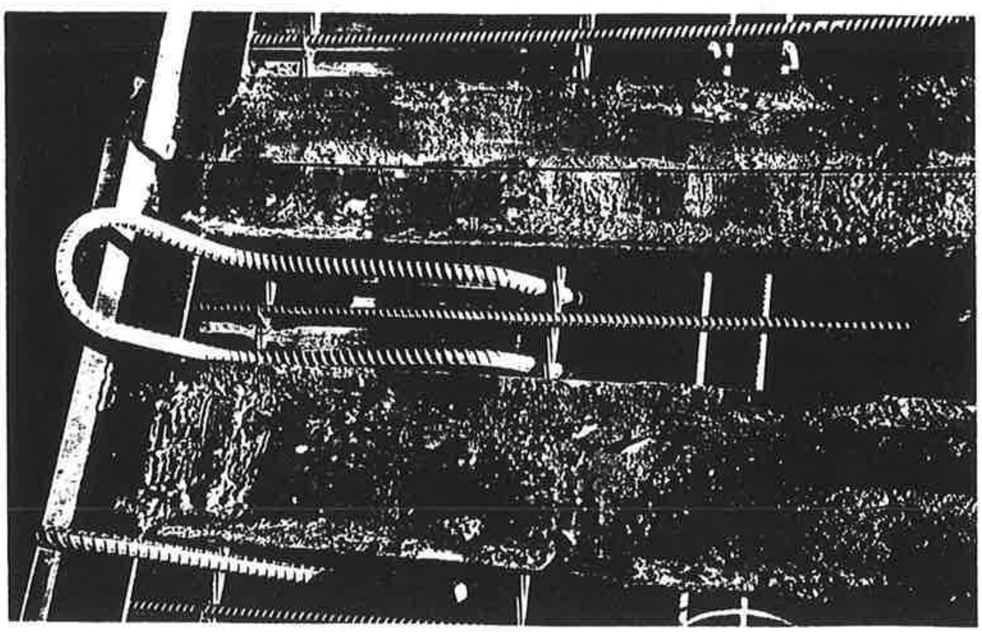


Fig. I.52 Assembling Steel Connectors

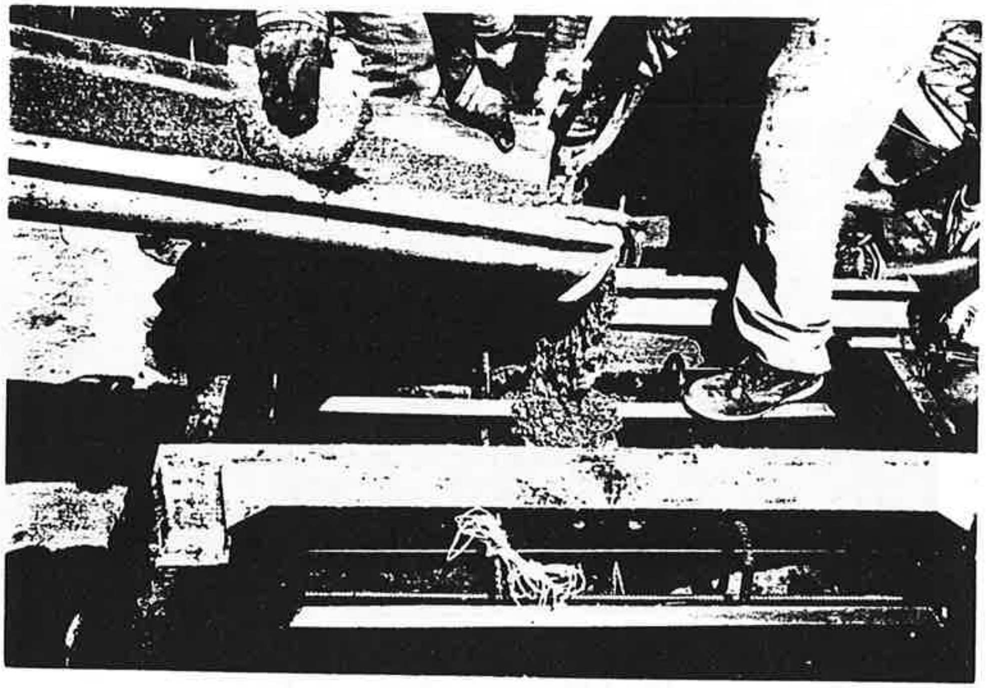


Fig. I.53 Pouring Concrete Girder

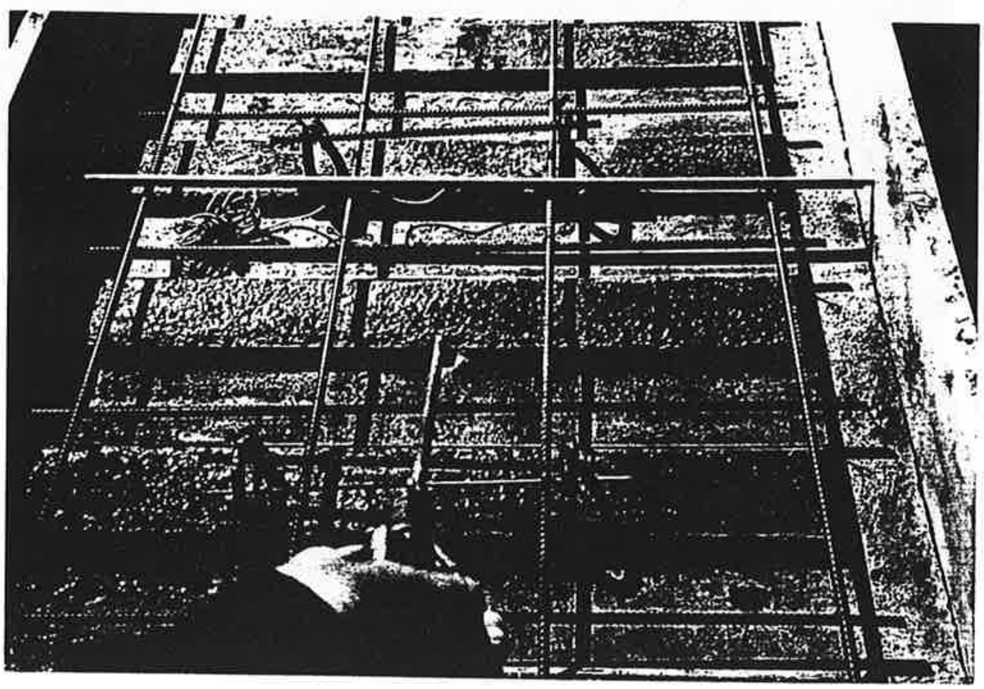


Fig. I.54 Applying Debonding Agent

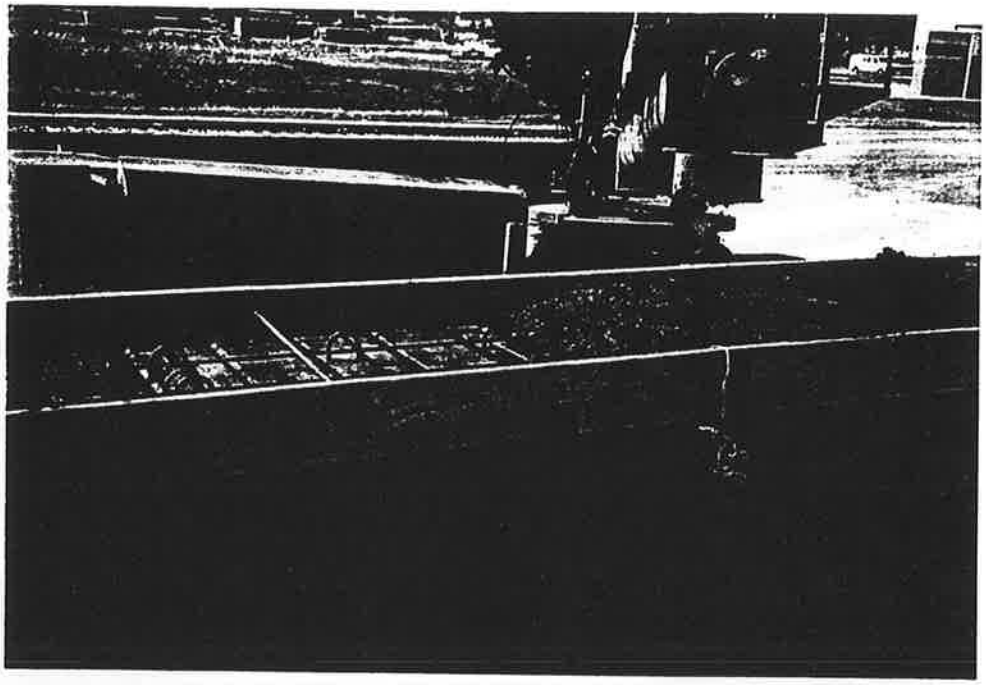


Fig. I.55 Pouring Concrete Deck

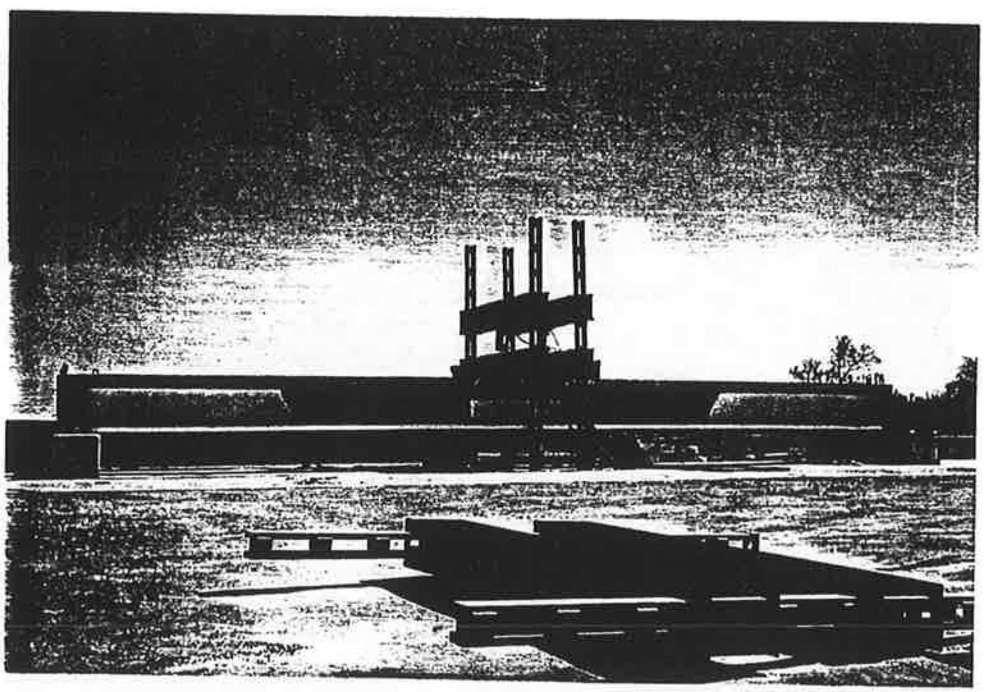


Fig. I.56 Set Up for Flexural Test

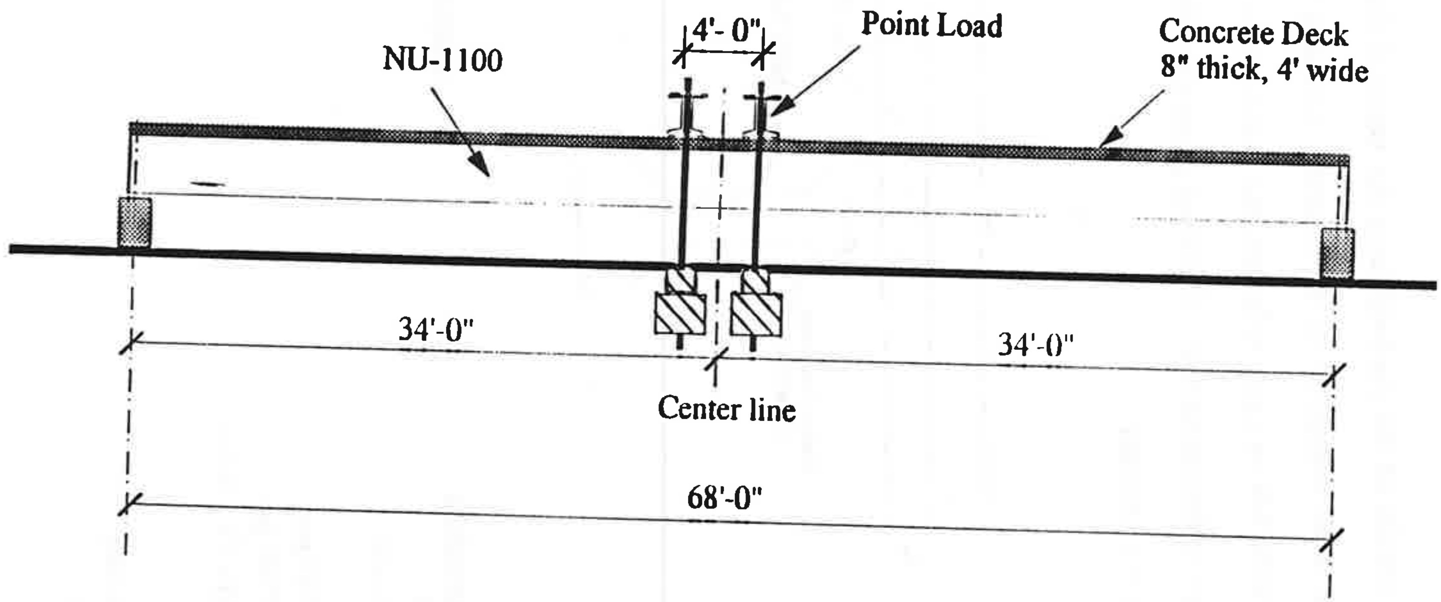
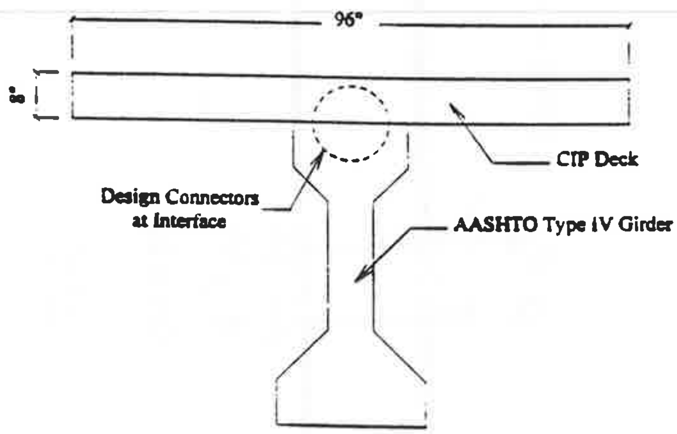


Fig. I.57 Set Up for Flexural Test

APPENDIX J

Concrete Bridge Example for Horizontal Shear Calculations

The results of the push-off tests for series 5 through 8 concrete girder-to-deck connections, were compared to an example concrete bridge design. The computer program CONSPAN by Leap Software, Inc. was used to generate the output results used for the design of the connectors. The input parameters for the example bridge are shown below. (1 in. = 25.4 mm)



<u>Design Assumptions</u>
span = 100 ft
HS-25 Live Load
SIDL = 0
Deck f'_c = 4000 psi
Girder f'_c = 7500 psi
AASHTO Type IV Girder
b_v = 20 in.
8 ft o.c.
24 ft bridge width
4 girder lines

<u>Output Results from CONSPAN</u>
at midspan
$M_{(LL+I)} = 1266.7$ kip ft
$M_u = 5385.5$ kip ft
$\phi M_n = 6289.2$ kip ft
$d = 57.0$ in.
$a = 4.21$ in.

- The calculations for determining the horizontal shear stress and size of connector bars for a 24 in. spacing with a debonded shear key interface are calculated as follows:
- Using the balanced section to determine the horizontal shear stress

$$M_u = (C\,or\,T)\left(d - \tfrac{a}{2}\right)$$

$$(C\,or\,T) = \frac{M_u}{d - \frac{a}{2}}$$

$$V = \frac{C}{span/2}$$

$\phi M_n = 6289.2$ kip ft   from CONSPAN

$$C = \frac{6289.2\,kip\,ft}{57 - \frac{4.21}{2}} = 1374.8\,kips$$

$$\phi V = \frac{1374.8\,kips}{100\,ft/2} = 27.5\,kips/ft$$

- Design shear connector for 24 in. spacing using AASHTO LRFD (1st edition)

  $\phi = 0.85$ for shear design per AASHTO

$$V_n = \frac{27.5\,kips/ft}{0.85} = 32.4\,kips/ft$$

$$V_n = c\,A_{cv} + \mu[A_{vf}\,f_y + P_c] \le 0.2\,f'_c\,A_{cv}\ or\ 0.8\,A_{cv} \quad \text{(LRFD Eq. 5.8.4.1-1)}$$

$c = 0$ (debonded shear key)

$A_{cv} = 20$ in. x 12 in. $= 240$ in^2/ft

$\mu = 1.0\ \lambda$, with $\lambda = 1.0$ for normal weight concrete

$f_y = 100$ ksi for high-strength bars

$P_c = 0$ (neglect)

$$A_{vf} = \left[\frac{V_n - c A_{cv}}{\mu} - P_c\right]\frac{1}{f_y} = \left[\frac{32.4k/ft - 0}{1.0} - 0\right]\frac{1}{100ksi} = 0.324\,\tfrac{in^2}{ft}$$

$$0.324\ \tfrac{in^2}{ft}\ \text{x 2 ft spacing} = 0.648\ \tfrac{in^2}{ft}$$

use 1 in. diameter bar ($f_y = 100$ ksi) at 24 in. o.c. with a debonded shear key interface

This is specimen UK-H-1-a&b for Series 7

- Determine the Full Factored and Composite Factored Load Stress using the balanced section at midspan.

1) Full Factored Load Stress = [1.3 x (1.0 x DL + 1.67 x (LL + I)] = 98 psi

$$V = \frac{C}{(span/2) x b_v}$$

$$C = \frac{M_u}{d - \frac{a}{2}}$$

$$C = \frac{5385.5 \times 12}{57 - \frac{4.21}{2}} = 1177 kips$$

$$v_n = \frac{1177 kips x 1000}{\left(\frac{100 ft}{2}\right) x\, 12" x\, 20"} = 98 psi$$

2) Composite Factored Load Stress = 1.3[1.67(LL + I)] = 50 psi

$$M_{comp} = 1.3[1.67(1266.7 \text{ ft kips})] = 2750 \text{ ft kips}$$

$$C=\frac{2750.0 \; x \; 12}{57-\frac{4.21}{2}}=601.0\,kips$$

$$v_n=\frac{601\,kips\,x\,1000}{\left(\frac{100\,ft}{2}\right)x\;12"\;x\;20"}=50\,psi$$

APPENDIX K

Material Testing

Concrete Cylinder Strength

4 in. x 8 in. cylinders
900 lbs/sec rate of loading

Series 1 Push-off Specimen Bottom Slab

see Table 6.1 for 28 day strengths

Series 2 Push-off Specimen Middle Slab Only

Age (days)	Cylinder 1 (psi)
7	5400
28	7000

Series 3 and 6 Push-off Specimen Bottom Slab

Age (days)	Cylinder 1 (psi)	Cylinder 2 (psi)	Average Stress (psi)
7	4961	5445	5203
25	5887	6083	5985
28	5997	5953	5975
33	6271	5836	6054
39	5679	5679	5679

Series 3 and 6 Push-off Specimen Top Slab

Age (days)	Cylinder 1 (psi)	Cylinder 2 (psi)	Average Stress (psi)
7	4420	4401	4411
20	4611	4479	4545
23	5244	4873	5059
28	5163	5071	5117

Series 4, 5 and 9 Push-off Specimen Bottom Slab

cylinders not constructed

Series 4, 5 and 9 Push-off Specimen Top Slab

Age (days)	Cylinder 1 (psi)	Cylinder 2 (psi)	Average Stress (psi)
14	5580	5583	5582
17	5693	5263	5478
22	5601	5410	5506
28	5949	5949	5949

Series 7 Push-off Specimen Bottom Slab

Age (days)	Cylinder 1 (psi)
5	5338
7	5652
12	6209
16	6728
23	7095
30	7974
51	7512
58	7804
72	7953

Series 7 Push-off Specimen Top Slab

Age (days)	Cylinder 1 (psi)
5	5844
10	6290
14	7100
21	7737
28	8050
49	7157
56	7815
70	7496
75	7240

Series 8 Push-off Specimen Bottom Slab

Age (days)	Cylinder 1 (psi)
8	4006
19	4924
22	4491

Series 8 Push-off Specimen Top Slab

Age (days)	Cylinder 1 (psi)	Cylinder 2 (psi)	Average Stress (psi)
2	4431	4261	4346
13	6255	6320	6288
16	6456	6395	6426
21	6583	6543	6563
34	6529	6349	6439
37	6526	-	6526

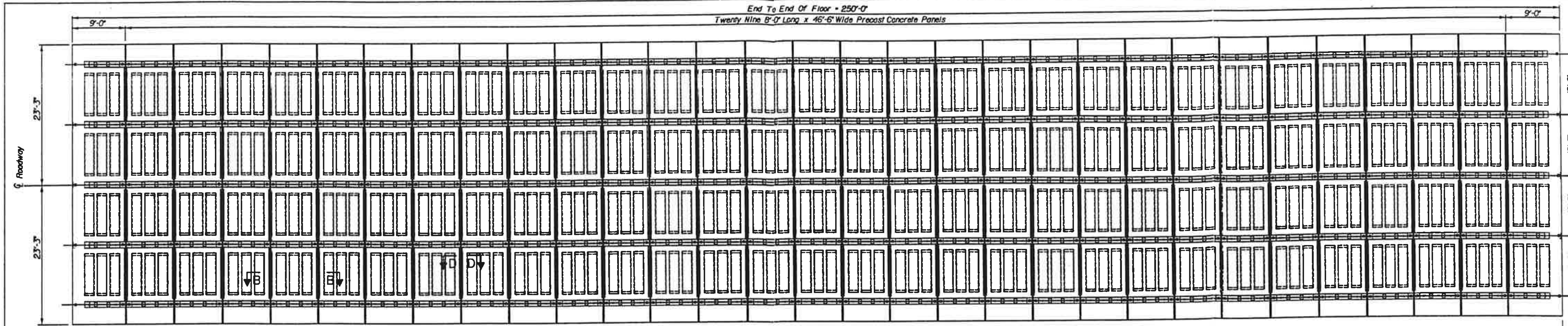
Steel Push-off Specimen

Age (days)	Cylinder 1 (psi)	Cylinder 2 (psi)	Average Stress (psi)
7	3696	3913	3755
18	4592	5459	5026
21	5271	5480	5376
28	5623	4710	5167
42	4939	5025	4982
47	5360	-	5360

Direct Shear Tests of Steel Threaded Rods

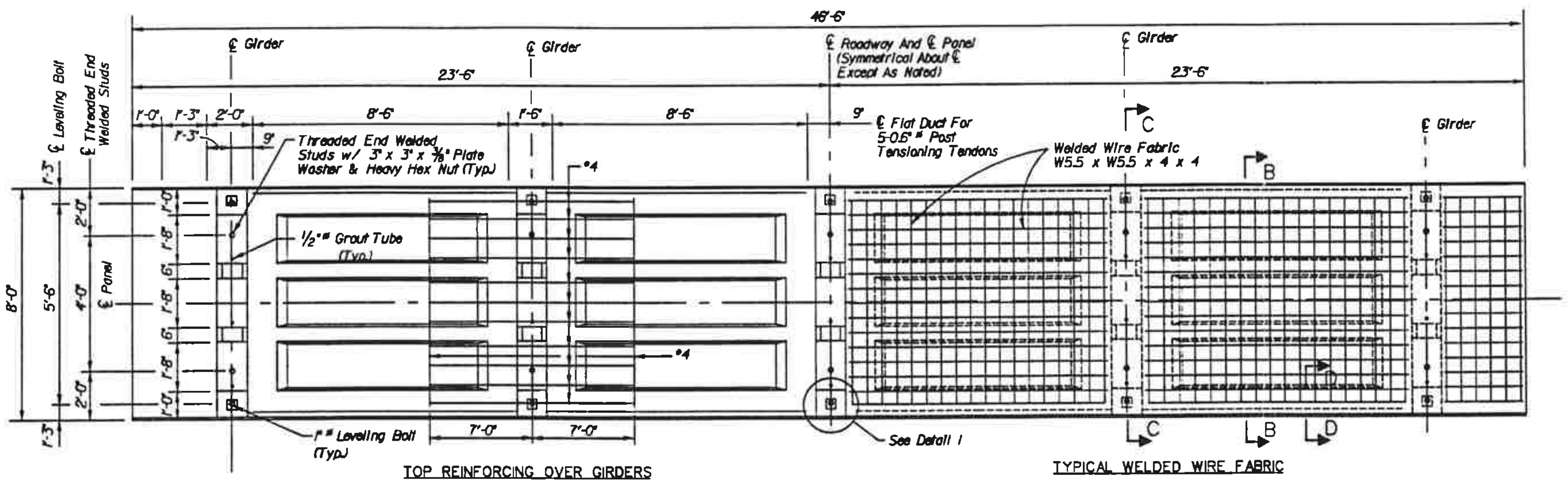
High-Stength Threaded Rod (f_y = 100 ksi)

Bar Size	Bar 1 Ultimate Shear Stress (ksi)	Bar 1 Ultimate Shear Stress (ksi)	Average Ultimate Shear Stress (ksi)
1 in. Dia.	61.1	60.5	60.8
$^3/_4$ in. Dia.	55.4	66.7	61.1
$^1/_2$ in. Dia.	43.4	58.7	51.1



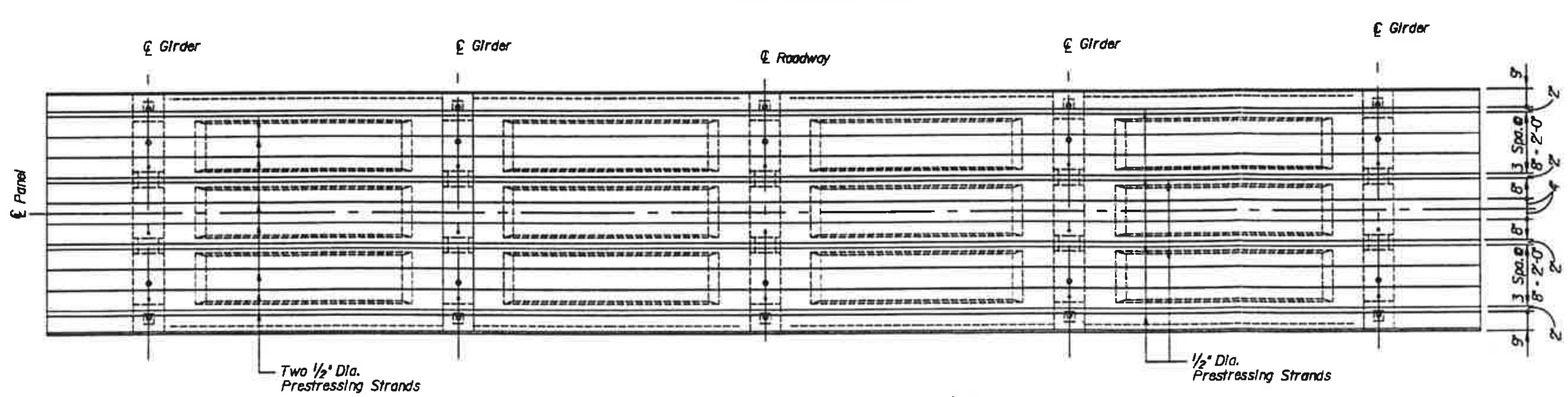
PLAN VIEW

PRECAST PANEL LAYOUT FOR A 250' X 46'-6" BRIDGE



PLAN VIEW

TYPICAL INTERIOR PANEL



PRESTRESSING STRAND LAYOUT

CONSTRUCTION DETAILS
TASK 7.9 AS IT RELATES
TO 7.1 NEW CONSTRUCTION
OR DECK REPLACEMENT
UNDER TOTAL CLOSURE

Date: December 1995

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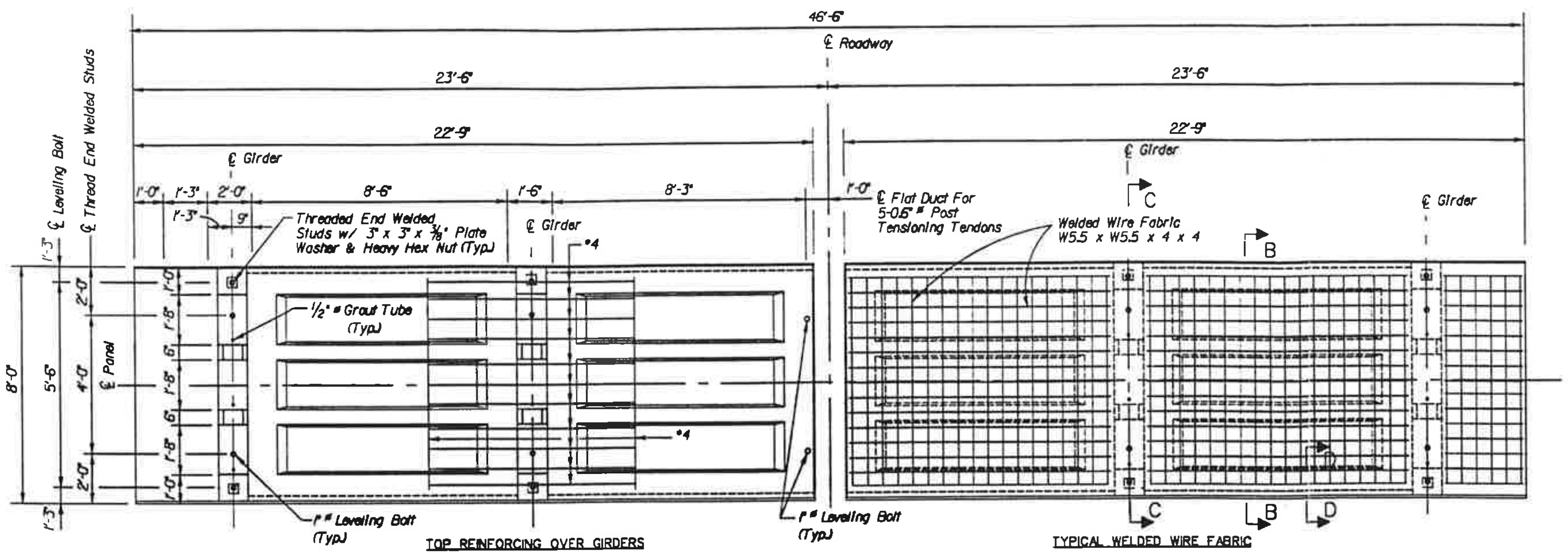
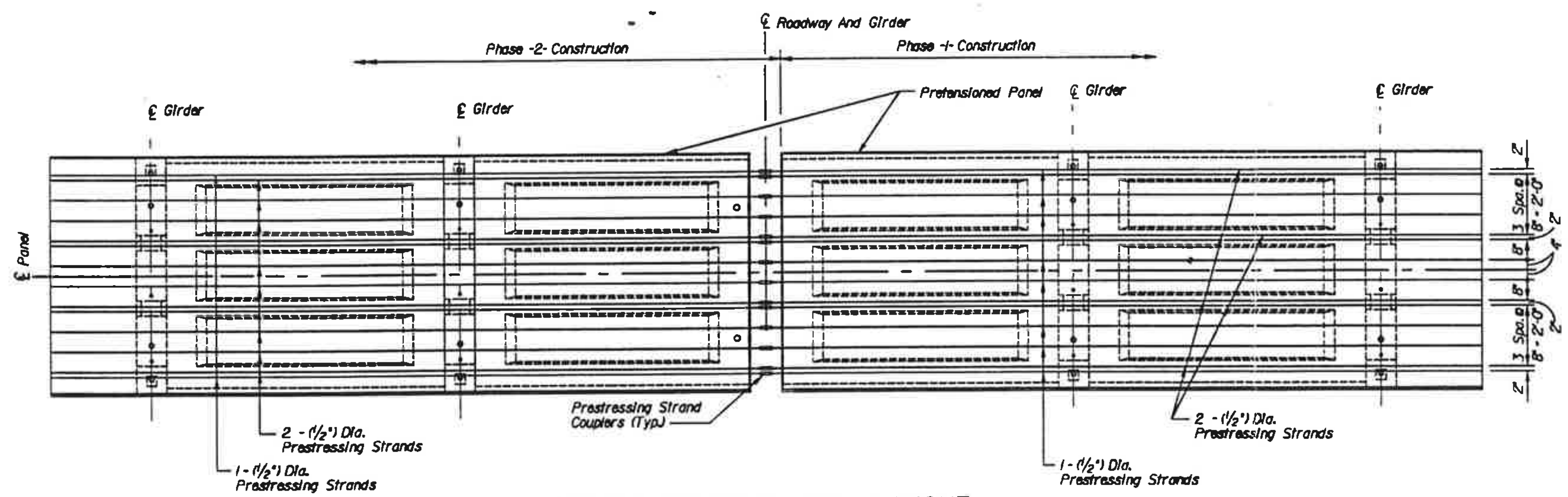
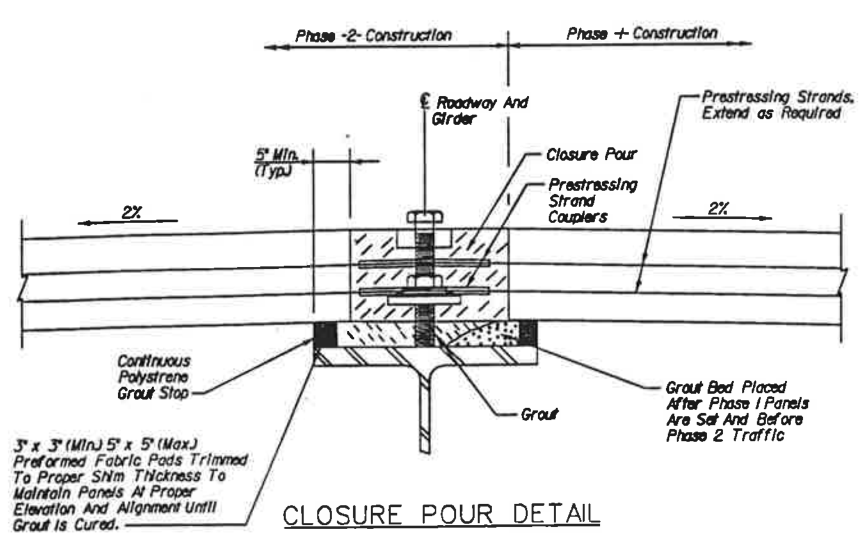


PLAN VIEW

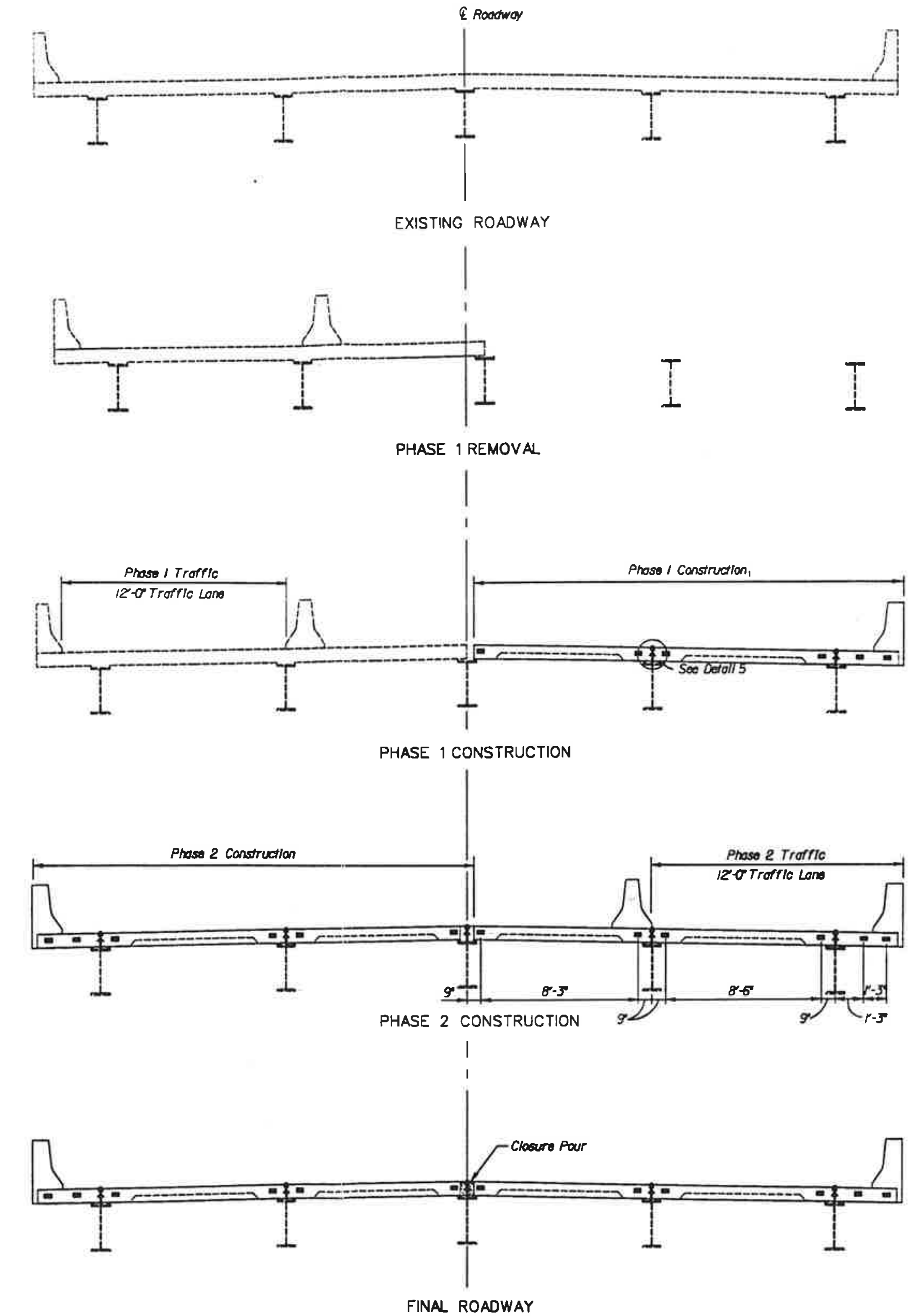
TYPICAL INTERIOR PANEL



PRESTRESSING STRAND LAYOUT



CLOSURE POUR DETAIL



PHASING PLAN

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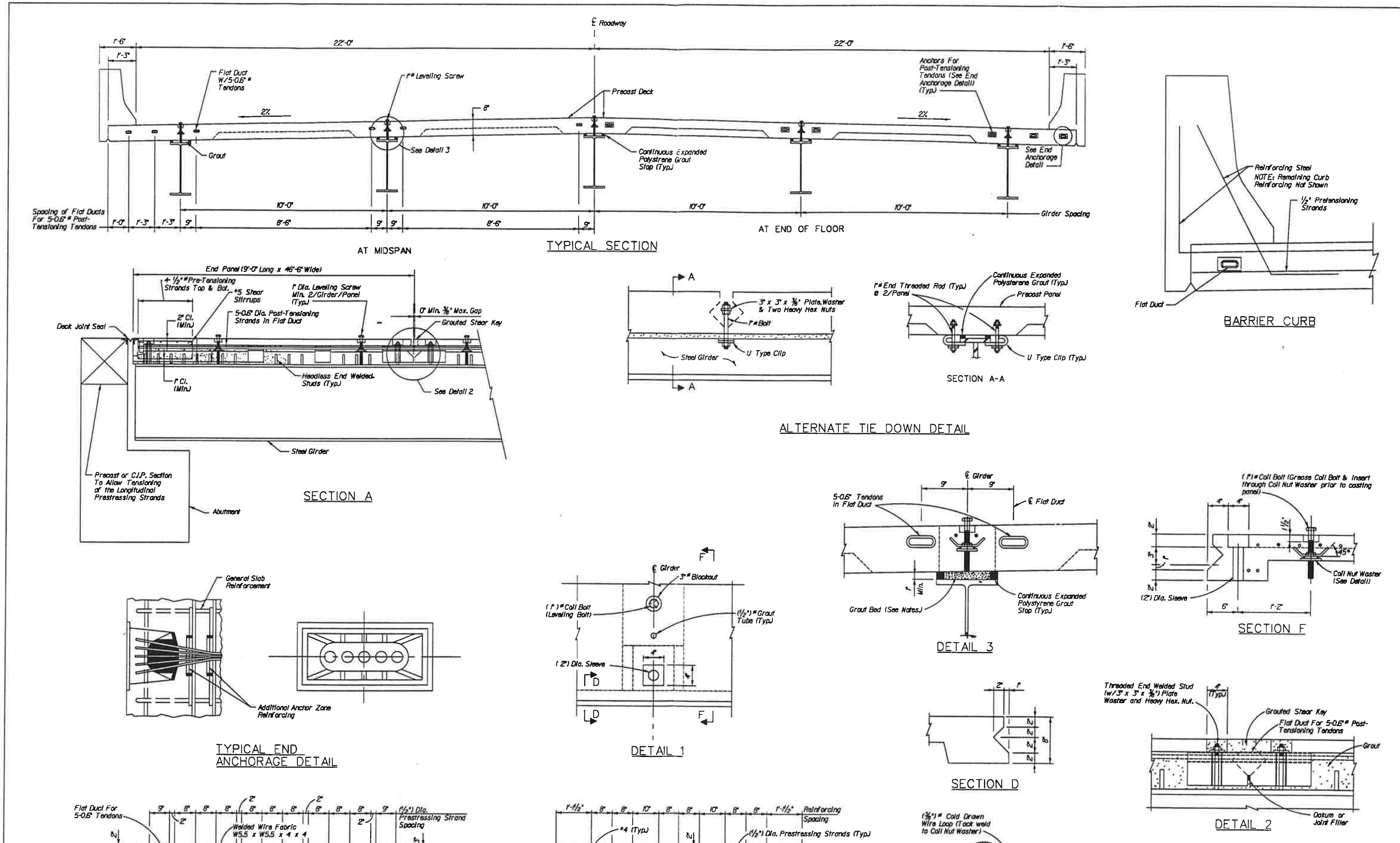
CONSTRUCTION DETAILS

TASK 7.9 AS IT RELATES TO TASK 7.1 DECK REPLACEMENT, PHASED CONSTRUCTION

Date: December 1995



CONSTRUCTION DETAILS TASK 7.9 AS IT RELATES TO TASK 7.1

Date: December 1995

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Lewis Demitroff
FILE: d:\slides\ochrp2.dgn
DATE: 28-Jul-95 14:41