APPENDIX L

Analysis of Unbonded Shear Key Systems

A model of the shear key mechanism resisting a horizontal force is shown in Fig. L.1. The angle of the shear key sides allows the two interfaces to slide across one another and hence the connecting bar starts to engage in the mechanism. While sliding occurs, the two surfaces separate from each other causing tensile and shear stresses in the steel connector. To simplify the analysis, bending stresses in the connector are neglected, as the moment arm is very small. The system of forces acting on this model are: (1) the applied force P, (2) the tensile force in the connector, P_y, (3) the shear force in the connector P_{xy}, (4) the bearing force on the side of the shear key R, and (5) the friction force on the side of the shear key δR, where δ is the "local" coefficient of friction between the two surfaces. The two surfaces are assumed to be smooth.

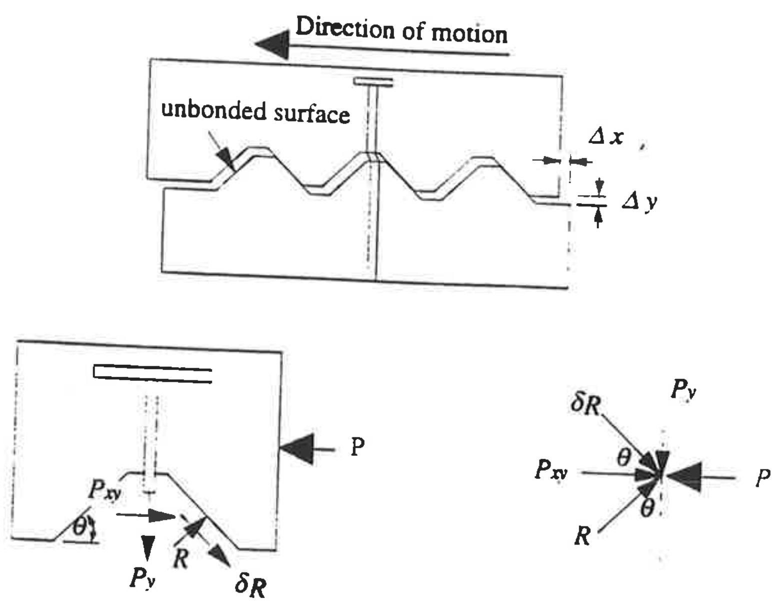


Fig. L.1. Mechanism of Proposed Composite Unbonded Shear Key Connection

The purpose of the following simplified analysis is to predict the ultimate horizontal force that can be resisted by the proposed composite connection. Once the force P is obtained, an overall coefficient of friction μ can be evaluated. The shear friction theory assumptions are valid in this analysis. For an assumed displacement Δx and Δy in the X and Y directions, the tensile and shear strains ε_y and γ_{xy} in the connecting bar can be expressed as:

$$\varepsilon_y = \frac{\Delta y}{\ell_t} \qquad \text{(L.1)}$$

where ℓ_t = effective bare length in tension.

$$\gamma_{xy} = \frac{\Delta x}{\ell_s} \qquad \text{(L.2)}$$

where ℓ_s = effective bar length in shear.

The tensile stress is $\sigma_y = \varepsilon_y E$ (L.3)

and the average shear stress is given by $\tau_{xy} = \gamma_{xy} G$ (L.4)

where E is the modulus of elasticity and G is the shear modulus of the connector

$G = \dfrac{E}{2(1+\nu)}$, where ν = the Poisson ratio of steel reinforcement , $\nu \cong 0.3$, and $G \cong E/2.6$. In Eq. L.4, the shear stress is assumed to be uniformly distributed on the connector cross section as its area is relatively small, Bakhoum (1992). For a connector with a cross sectional area A, the forces P_y and P_{xy} can be expressed as:

$$P_y = A\ \sigma_y \tag{L.5}$$

$$P_{xy} = A\ \tau_{xy} \tag{L.6}$$

From equilibrium of the forces in the Y direction as shown in Fig. L.1,

$$P_y + \delta\ R\,sin\theta = R\,cos\theta \tag{L.7}$$

Equation L.7 can be rewritten as follows:

$$R = \frac{P_y}{cos\theta - \delta\ sin\theta} \tag{L.8}$$

From equilibrium of the forces in the X direction as shown in Fig. L.1,

$$P = P_{xy} + R\,sin\theta + \delta\ R\,cos\theta = R\,(sin\theta + \delta\ cos\theta) + P_{xy} \tag{L.9}$$

Combining Equations (L.8 and L.9):

$$P = \frac{P_y\left(sin\theta + \delta\ cos\theta\right)}{\left(cos\theta - \delta\ sin\theta\right)} + P_{xy} \tag{L.10}$$

For $\theta = 45^{\circ}$, $sin\theta = cos\theta$ and also $\Delta y = \Delta x$, Equation (L.10) is simplified as:

$$P = \frac{P_y(1+\delta)}{(1-\delta)} + P_{xy} \tag{L.11}$$

For an assumed set of values of Δx, Δy, ℓ_t, ℓ_s, *and* δ, the values of P can be calculated. The ℓ_t value is taken in this analysis as the full bar length. The value of δ

is taken 0.4 as recommended by Shiakh (1978) for smooth surfaces. The displacements Δy, Δx are increased until the maximum value of the force P is obtained. The maximum value of the force P would be the force that corresponds to yield strength either due to principal shear or tensile stresses. Yield shear strength is taken as 0.6 times the tensile shear strength. This assumption is confirmed with tests results. Maximum principle tensile and shear stresses are given as follows:

$$\sigma_y\, max = \frac{\sigma_y}{2} + \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + (\tau_{xy})^2} \tag{L.12}$$

$$\tau_{xy}\, max = \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + (\tau_{xy})^2} \tag{L.13}$$

Once the value of maximum P is determined, an overall coefficient of friction μ can be evaluated as:

$$\mu = \frac{P}{A\, f_y} \tag{L.14}$$

Equation L.14 is the same linear shear friction equation. A spreadsheet is used to perform the calculation and to obtain analytical μ values for assumed values of ℓ_s of 1", 1.25", and 1.5", and assumed values of angle θ of 35, 40, 45 and 50 degrees. The assumed range of ℓ_s values are confirmed with observations of the push-off tests. However, the 45 degree angle of the shear keys is chosen to be used in the test program and is proposed for use in the new composite system to simplify forming the shear keys with a slope of 45 degrees. Fig. L.2 represents the estimated μ values. The values

range from 1.3 to 1.56 for θ of 45 degrees. This result agrees with the values found in the literature for the case of a precracked interface. In addition, similar values for coefficient of friction were obtained from test results. For $\theta = 45^{\circ}$, $\ell_s = 1.25$", the values of shear force resisted by the connector are approximately 0.35 of the total horizontal force, and the bearing force on the shear keys is approximately 0.65 of the total force.

The existence of horizontal shear stresses in a beam due to bending can be visualized using Fig. L.3. If the beam is made of a series of planks with smooth, frictionless surfaces and a transverse load is applied, the planks will bend independently and slide freely over each other as shown in Fig. L.3(B). Each plank will be in compression above its neutral axis and in tension below its neutral axis. If the planks are clamped together for the full length of the beam so that sliding is prevented, the strength of the beam will be identical to that of a monolithic beam of the same cross section. In this case, horizontal shear stress exists at the interface to prevent sliding as shown in Fig. L.3(C). In composite beams where two layers are connected together (Fig. L.3(D)), some slip must occur before full composite action takes place. Three methods of horizontal shear analysis are discussed in the following sections.

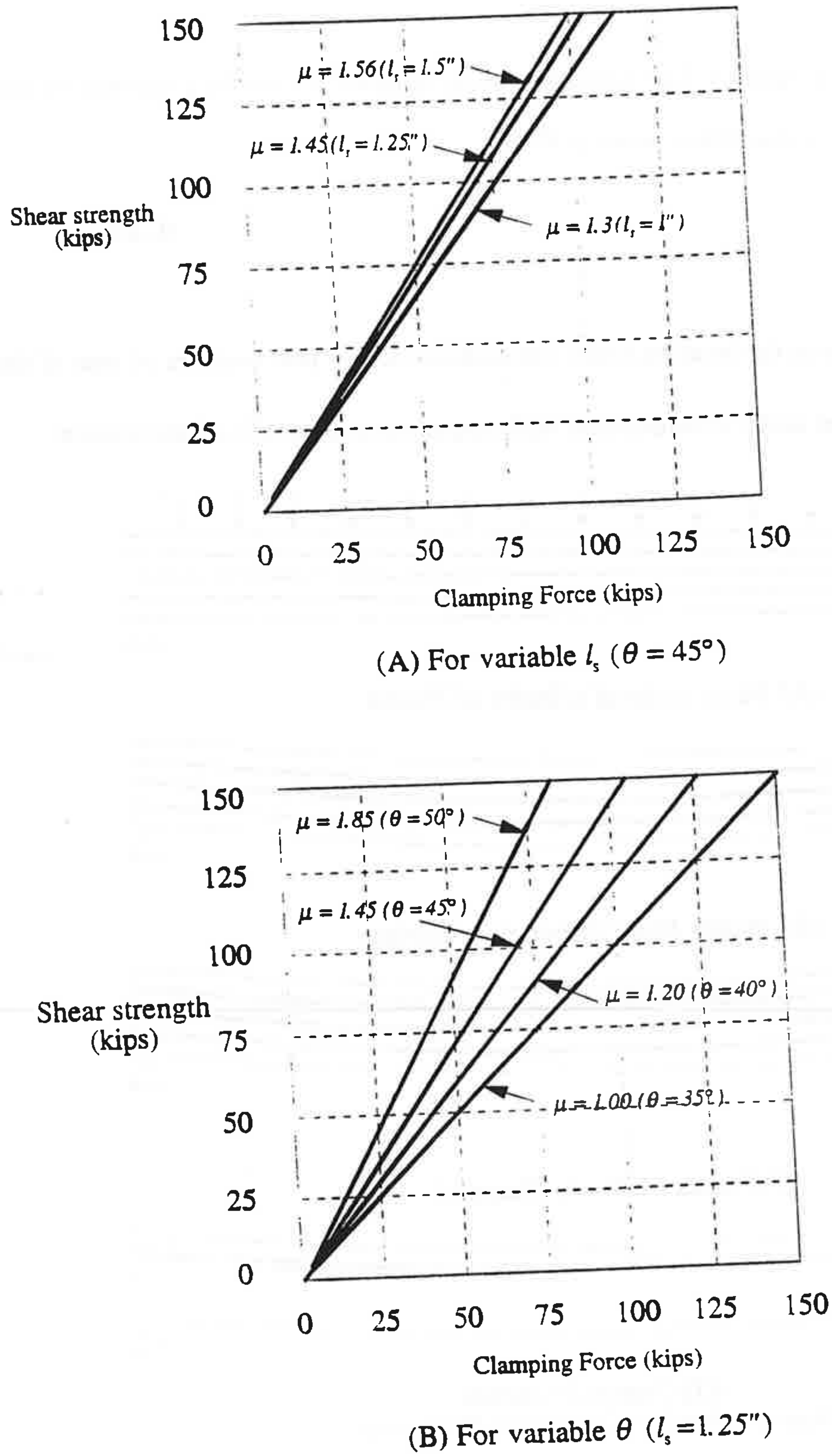
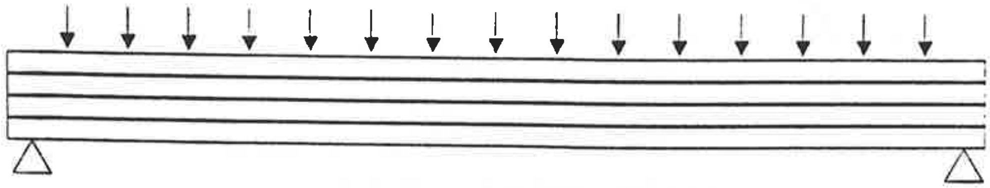


Fig. L.2 *Analytical Prediction of Coefficients of Friction for Unbonded System with Shear Keys*

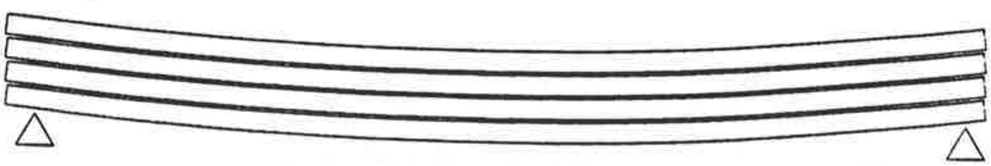
Working Stress Method: The horizontal shear stresses, v, can be evaluated by the following formula (Ger and Timoshenko (1984):

$$v = \frac{VQ}{Ib} \tag{L.15}$$

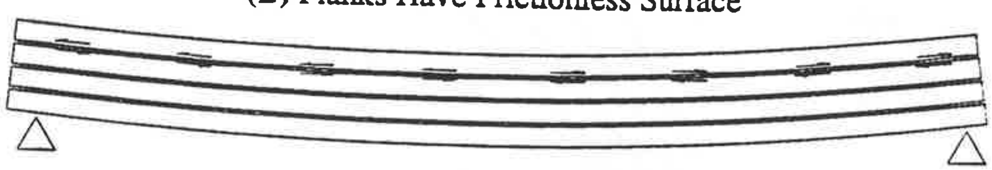
where, V = shear force at the location under consideration, Q = first moment of area of the portion above the level under consideration with respect to neutral axis of the section,



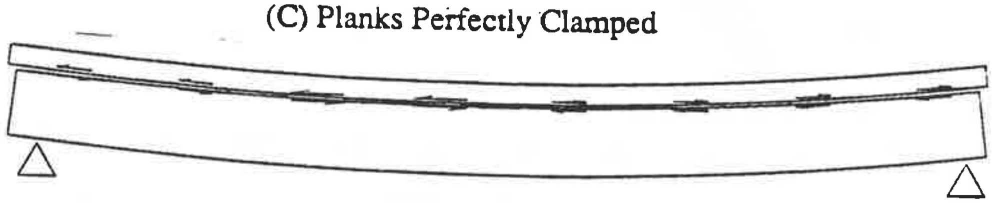
(A) Beam made of a Series of Planks



(B) Planks Have Frictionless Surface



(C) Planks Perfectly Clamped



(D) Composite Beam
(actual behavior in precast-CIP beams)

Fig. L.3. Development of Horizontal Shear Stresses in Beams

I = moment of inertia of the section, and b = width of web at the level under consideration. This expression is derived for an uncracked elastic condition. However, it was adopted by several investigators to evaluate horizontal shear stresses at ultimate loads (Hanson, (1960), Saemann & Washa, (1964), and Loove & Patnaik, (1994)). This was done using cracked section properties.

Code Method: Both the ACI Code (1995) and the AASHTO Standard Specifications (1992) related the horizontal shear stress multiplied by $(b_v d)$ to the total factored vertical shear force. This implies that the nominal horizontal shear stresses v_{nh} are calculated as:

$$v_{nh} = \frac{V_n}{d\,b_v} \tag{L.16}$$

where V_n = the nominal vertical shear force, b_v = width of interface, and d = depth of steel reinforcement.

Equilibrium Method: Horizontal shear stresses can be evaluated by satisfying equilibrium at any two adjacent sections. This is done by computing the actual compressive or tensile forces in any segment and transferring these forces as interface shear to the supporting element. This method is permitted by both the ACI Code (1995) and the AASHTO Standard Specifications (1992) as an alternative approach.

Analytical Derivations of Horizontal Shear Equations

Consider the simply supported beam shown in Fig. L.4(A) which is subjected to a random load. Isolate an infinitesimal slice of the beam of length dx bounded by the sections 1 and 2 as shown in Fig. L.4(B). The two sections are subjected to bending

moment M and $M + dM$ and shearing forces V and $V+dV$, respectively. The compressive forces on the top area, which are the resultant of the normal stresses induced by the bending moments, are indicated as the two forces C and $C + dC$.

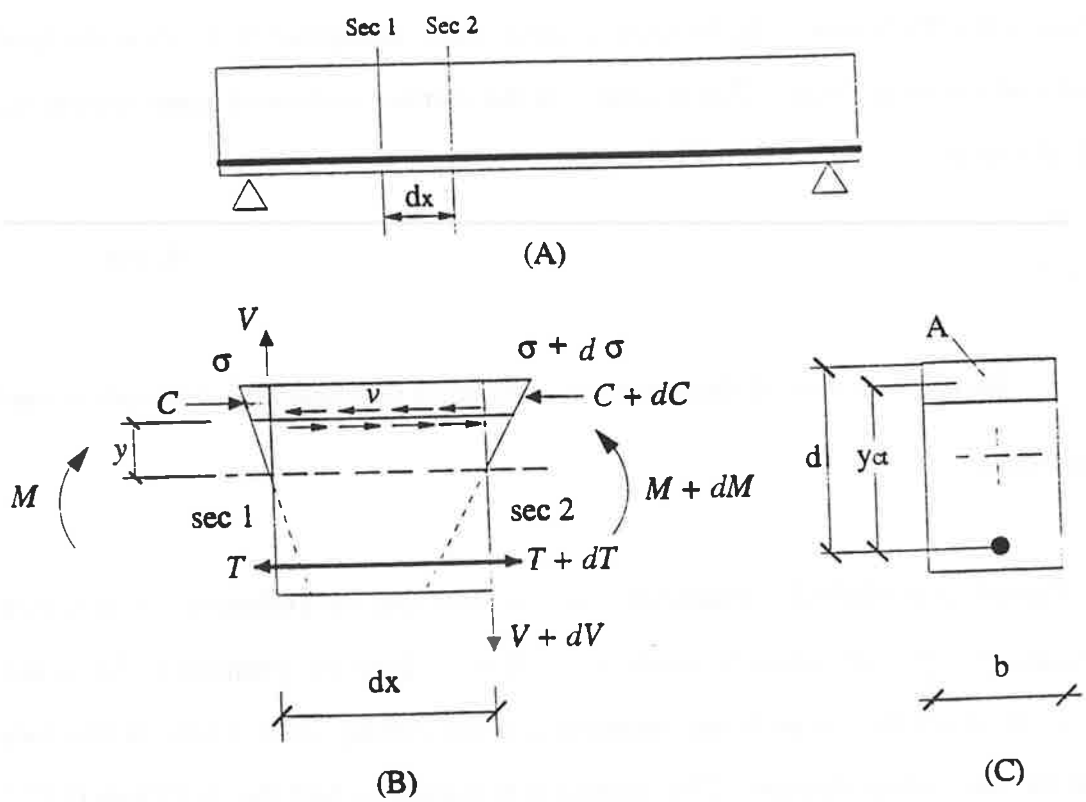


Fig. L.4. Development of Horizontal Shear Stress Expressions

Investigating the equilibrium of the upper portion of the slice at a level of y from the neutral axis shows that a horizontal shear stress has to exist in order to equilibrate the force dC. Assuming such stress v to be uniformly distributed over area bdx, the following equations can be derived:

$$dC = v \ b \ dx \tag{L.17}$$

$$C = \int_A \sigma \ dA = \int_A \frac{M}{I} y \, dA = \frac{M}{I} Q \tag{L.18}$$

where A is the area of the hatched portion of the section and Q is the first moment of this area about the neutral axis of the section. From Eq. L.18 and knowing that the shear force V is equal to the first derivative of the bending moment M, Eq. L.18 can be derived as:

$$\frac{dC}{dx} = \frac{Q}{I}\frac{dM}{dx} = \frac{VQ}{I} \tag{L.19}$$

Substituting Eq. L.17 into Eq. L.19, the traditional horizontal shear stress equation, $v = \dfrac{VQ}{Ib}$ (Eq. L.15), is obtained.

Although the previous analytical derivations were based on elastic analysis, the relations still apply to reinforced concrete in plastic states. However, a direct method of analysis may be simpler and more informative. For the same beam shown in Fig. L.4, the compressive force C can be described as:

$$C = \frac{M}{y_{ct}} \text{ and hence, } dC = \frac{dM}{y_{ct}} \tag{L.20}$$

where y_{ct} is the distance between the steel reinforcement and the centriod of the compressive force C. Substituting Eq. L.20 into Eq. L.17, the following expression is obtained:

$$v = \frac{V}{y_{ct}\, b} \tag{L.21}$$

In reinforced concrete composite beams, the neutral axis at ultimate usually lies in the depth of the top slab, and the distance between compression and tension forces = d - $a/2$ where a is the depth of the compression block in the composite slab and d is the depth of steel reinforcement. This distance can be approximated to $0.9\ d$ with no loss in accuracy. Applying this concept to Eq. L.21, the following expression is obtained:

$$v = \frac{V}{0.9 d b} \tag{L.22}$$

The previous analysis of horizontal shear stress shows that the three methods of analysis, the working stress method, the code method and the equilibrium method are basically the same. However, the code method adopted by both the current ACI Code and the AASHTO Standard Specifications are not conservative. The code equation should by divided be 0.9 to account for the distance between tensile and compressive forces. It is expected that the three methods give close results.

Composite loads

Fig. L.5(A) shows a precast beam subjected to its own weight only. When a topping slab is cast over the beam, Fig. L.6(B), the beam stresses are increased due to the dead weight of the wet concrete. However, in this case, even after the slab concrete is hardened, the stresses in the slab are theoretically zero, neglecting the minor effect of creep and shrinkage. Therefore, no horizontal shear stresses exist at the interface due to either the beam or the slab weights.

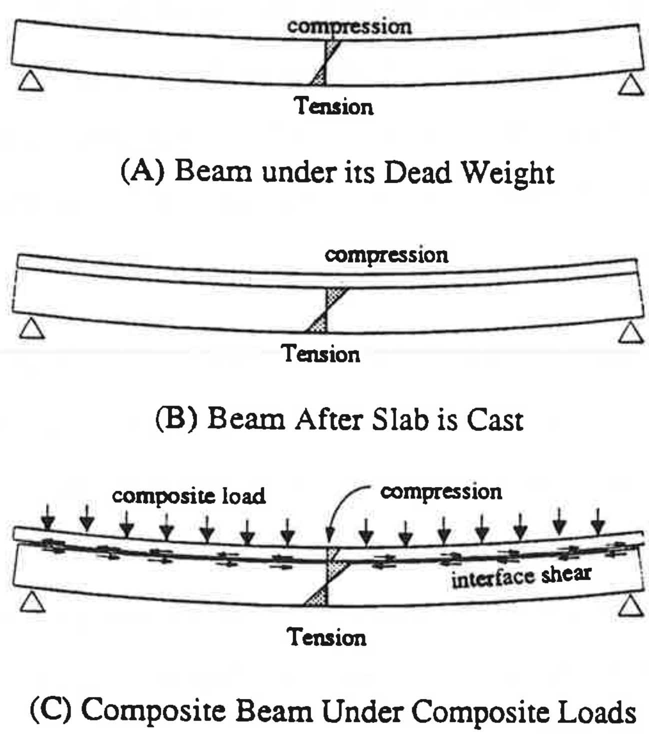


Fig. L.5. Composite Load

At this stage, the beam and the slab form a composite member which has a much higher stiffness than the summation of the stiffness of the beam and the slab separately. Any load applied on the composite section produces interface horizontal shear stresses. These loads are called composite loads. In general cases of bridge design, composite loads consist of superimposed dead loads and live loads. Superimposed dead loads include the weight of a future wearing surface and the weight of barrier rails. Live load includes truck loads, lane loads or pedestrian loads.

Both the ACI Code and the AASHTO Standard Specifications require that the horizontal shear stresses be evaluated for total factored shear force due to dead and live loads at any section along the beam length, which is considered a conservative approach. As discussed before, the two codes allow the use of the equilibrium method to evaluate the horizontal shear stresses by determining the change in compressive or tensile forces in the topping to be transferred along the interface. Any stage of loading before composite action takes place will generate no stresses in the topping, and therefore no forces need to be transferred along the interface.

If the horizontal shear stress is evaluated due to composite loads, will the member fail in flexure before it fails in horizontal shear? The answer to this question depends on the way flexural capacity is calculated and determined in the laboratory for a concrete member. According to most reinforced concrete codes, factored loads are those corresponding to each stage of loading. This means that noncomposite loads should be factored and introduced to the precast beam before composite action takes place and factored composite loads should be introduced to the composite section. This procedure would be consistent with the codes for laboratory testing of composite beams in order determine their flexural capacity. The obtained capacity would be comparable to the required ultimate moment calculated from dead and live loads, and if designed properly, flexural failure would occur before horizontal shear failure. However, this is not the way

beams are tested in structures labs. They are tested by applying the total loads on the composite section because it is more convenient. However, in this case, the obtained flexural capacity represents only the flexural capacity of the composite section. The following example compares the methods of horizontal shear analysis and demonstrate that horizontal shear stresses should be evaluated for composite loads only.

Comparison of Various Methods

In order to compare the various methods of calculating horizontal shear stress presented in the previous section, an example of a simple span beam is demonstrated. The stresses at midspan section and the distribution of the horizontal shear stresses along the span by various methods is calculated. Factored dead loads are applied to the precast beam, then factored composite loads are applied to the composite section.

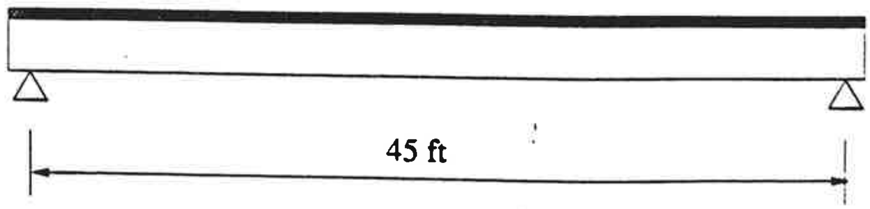
Fig. L.6 shows the beam geometry and the results of the calculations. It shows a rectangular precast reinforced concrete beam which is made composite by casting a concrete slab on its top. The horizontal shear stress of this example is calculated by the various methods discussed previously. The following comments can be made:

- Calculating horizontal shear stresses using the shear diagram divided by 0.9d (method 2) or by using the working stress equation (method 1) with cracked moment of inertia gives almost the same results.

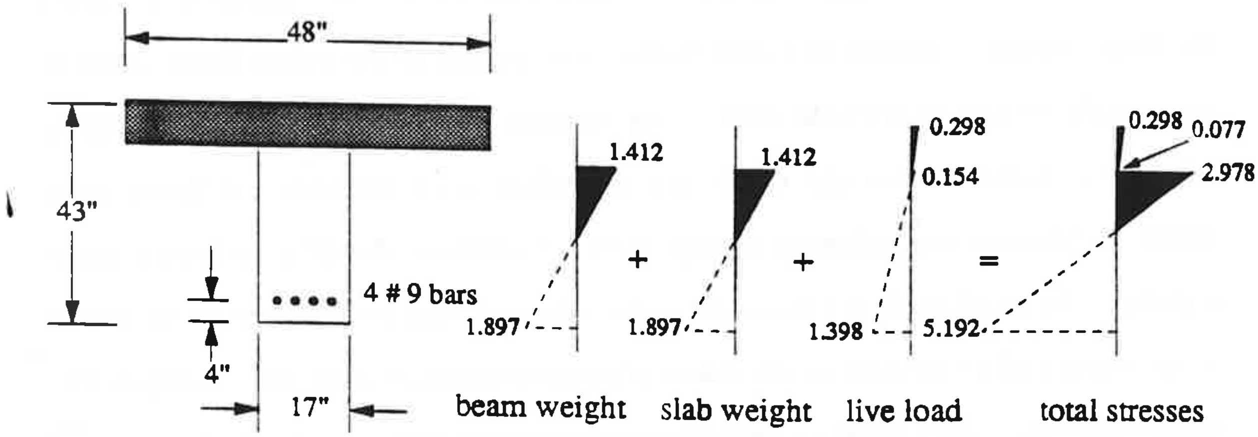
- Calculating horizontal shear stresses by estimating the compressive force at midspan from the stress diagram (method 3) or from the composite moment divided by 0.9d (method 4) and distributing the force over half the span gives almost the same results.

- The areas under the diagrams of methods 1, 2, 3, and 4 are the same. However, their shapes are different. This is because in methods 3 and 4, the midspan forces are distributed over half the span. Methods 3 and 4 would result in the same diagrams as methods 1 and 2 if the beam is divided into small segments and the forces are distributed along the length of each segment.

- Calculating horizontal shear stresses at ultimate by using the working stress equation with gross moment of inertia (method 5) is a wrong assumption and as expected would significantly underestimate the horizontal shear stresses. This method would be only applicable at working stress level where the member is still uncracked.

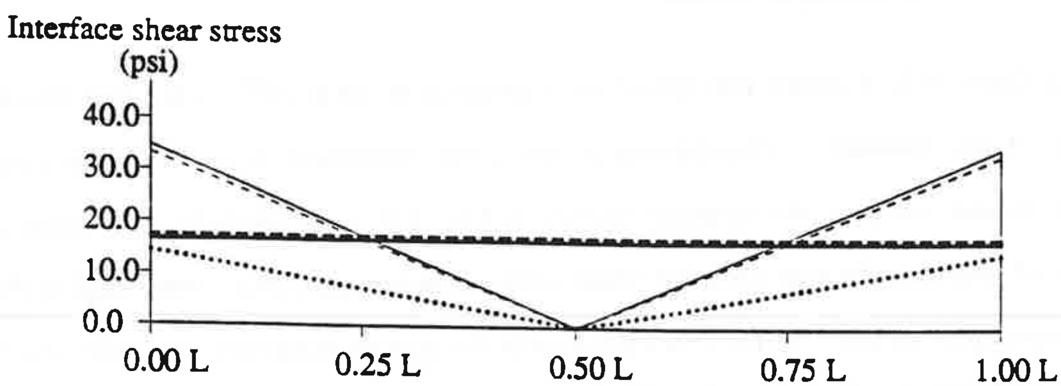


(A) Simple Span Composite Beam



(B) Beam cross section

(C) Stress diagrams at midspan (ksi)



(d) Distribution of interface shear stress along beam length

(1) ------ Elastic method = $V_{composite}\,Q/I_{cr}\,b$ (Loove& Patniak)

(2) ——— $V_{composite}/0.9d\,b$ (ACI & AASHTO Eq./0.9)

(3) - - - - - (M at midspan/0.9 d)/L/2

(4) ——— Compressive force estimated from stress diagram/L/2

(5) .... Elastic method = $V_{composite}\,Q/I_{gross}\,b$

Fig. L.6. Horizontal Shear Stress of a Simple Span Beam Example

Effect of Continuity

As discussed in the previous sections, horizontal shear stress distribution follows the shear diagram. Moment and shear forces are calculated by the elastic theory which is not exactly correct at ultimate state. As member strength is approached, inelastic behavior at some sections can result in a redistribution of moments and hence shear forces. Moment redistribution is dependent on adequate ductility in plastic hinge regions. The usual result is a reduction in the values of negative moments in the plastic hinge region and an increase in the values of positive moments from those computed by elastic analysis. Since negative moments are determined from one loading arrangement and positive moment from another, each section has a reserved capacity that is not fully utilized for any one loading condition.

The ACI Code (8.4.1) allows redistribution of moment of up to 20%, depending on the ductility of the member. Considering a two span continuous beam with the same concrete section used in the example shown in Fig. L.6, a redistribution of moment diagram of less that 6% was obtained when cracked section properties were used in the negative moment region. In prestressed concrete continuous members, redistribution of negative moments is allowed by the ACI Code if $0.85\ a / d_p$ is not greater than $0.24\ \beta_1$. In most cases of bridge design, this condition is satisfied. Examining an example of a two-span continuous bridge of NU1100 precast girders with 8 ft spacing and span length of 100 ft, a redistribution of total moment diagram of less than 10% was obtained when cracked section moment of inertia was used in the negative moment region. In conventional composite precast prestressed concrete bridges, the negative moment regions are not prestressed, and are subjected to cracking before positive moment regions at ultimate stage. This may affect the anchorage of the horizontal shear connectors and hence their efficiency to transfer forces. Evaluation of horizontal shear design after

redistribution of moments and shear forces in this case would be rational and desirable. It would result in a slight increase of the required shear connectors in the positive moment region and decrease of their values in the negative moment region.

Recommended Method of Calculating Shear Stress

Horizontal shear stresses should be evaluated due to factored composite loads only. Both ACI and AASHTO Standard methods of calculation of horizontal shear force per unit length as the vertical shear force divided by the depth of the steel reinforcement are not conservative. The equation used by both codes should be divided by 0.9 to reflect the actual couple arm between the compressive and tensile forces in the section.

It is important to notice that the approximation of the couple arm of *0.9 d* is valid only at the section that reaches ultimate flexural, for example, midspan of simple beams under uniform loads. Other sections along the span would have different arm length according to their level of stresses taking into consideration the nonlinearity of concrete stress-strain. A nonlinear analysis by Loove and Patniak (1994) shows that the approximation of *0.9d* is still valid along the rest of the span with almost no loss of accuracy.

APPENDIX M

Concrete Girder-to-Deck Connection

Concrete Full Scale Design Calculations

Composite I -Girder Single Span

1. DESIGN CONDITIONS

The Full Scale Girder used for testing the concrete girder-to-deck shear connection was modeled upon the following bridge. The bridge would consist of a 70 ft simple span structure. The superstructure would consist of four NU1100 girders, spaced at 12′-0″ on center. The total width of the bridge would be 44′-0″ as shown in Fig. M.1. Girders are designed to act compositely with the 8″ cast-in-place concrete deck to resist all superimposed dead loads, live load and impact. New Jersey type concrete barrier are assumed. This bridge is designed to resist a 25 psf superimposed dead load, and an AASHTO HS 25-44 truck load. Design is in accordance with AASHTO's 15th Edition including all interim up to 1995.

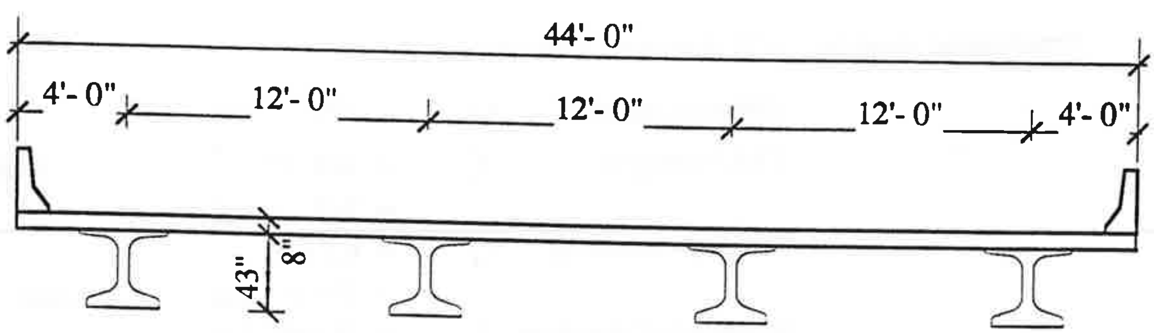


Fig. M.1 Bridge Cross-section

2. MATERIALS

Cast in place slab:	Thickness	t_s	=	8	inch
	28-Days concrete strength	f'_{cs}	=	4000	psi
Precast girder:	Nebraska University NU1100 girder (shown in Fig. 2)				
	Release strength	f'_{ci}	=	6000	psi
	28-Days concrete strength	f'_c	=	8000	psi

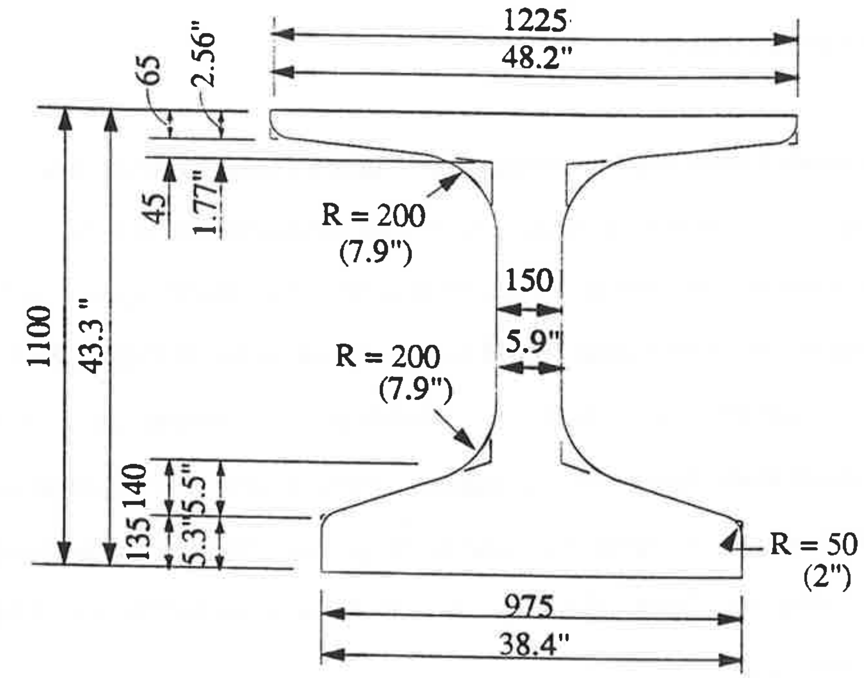


Fig. M.2 Standard NU1100 Girder

Prestressing strands: 1/2″ diameter, low relaxation

Ultimate stress	f'_s	= 270	ksi	
Yield strength	f^*_y	= 0.9 f'_s		(Art. 9.15)
		= 243	ksi	
Initial prestressing	f_{si}	= 0.75 f'_s		
		= 202.5	ksi	(Art. 9.15.1)
Modulus of elasticity	E_s	= 28,000	ksi	
Area	A_{ps}	= 0.153 in^2		

Reinforcing bars: Yield strength f_{sy} = 60,000 psi

3. CROSS-SECTIONAL PROPERTIES FOR INTERIOR GIRDER

3.1-Non-composite section

$A = 694.6 \ in^2$

$h = 43.3 \ in$

$I = 182279 \ in^4$

$y_b = 19.6 \ in$

y_t = 23.7 in

S_b = 9300 in^3

S_t = 7691 in^3

W = 724 plf

$$E_c = (w_c)^{1.5} (33) (\sqrt{f'_c}) \quad \text{(Art. 8.7.1)}$$

where E_c = modulus of elasticity of concrete (psi)

w_c = unit weight of concrete = 150 pcf (Art. 3.3.6)

for cast-in-place, $E_{cs} = (150)^{1.5}(33)\sqrt{4000} = 3.834 \times 10^6$ psi

for precast girder, $E_{ci} = (150)^{1.5}\, 33\, \sqrt{6000} = 4.696 \times 10^6$ psi

$E_c = (150)^{1.5}(33)\sqrt{8000} = 4.696 \times 10^6$ psi

3.2-Composite section

Modular ratio between girder and slab materials:

$$n = \frac{E_{cs}}{E_c}$$

$$n = \frac{3.834 \times 10^6}{5.422 \times 10^6} = 0.71$$

Compute effective flange width (Art. 9.8.1.1)

Effective girder web width shall be the least of (Art. 9.8.3)

b_E = top flange width = 48.3 in.

= 6 x 4 + 5.9 + 2 x 5 = 40 in.

use b_E = 40 in.

The effective flange width shall be the least of

1/4 span length $= \frac{70 \times 12}{4}$ = 210 in.

Distance center-to-center of girders = 12 ft = 144 in.

12 x Effective slab thickness plus effective girder web width

= 12 x 8 + 40 = 136 in. controls

Effective flange width = 136 in

Transformed flange width = (n) (effective flange width) = 0.71 x 136 = 96.56 in.

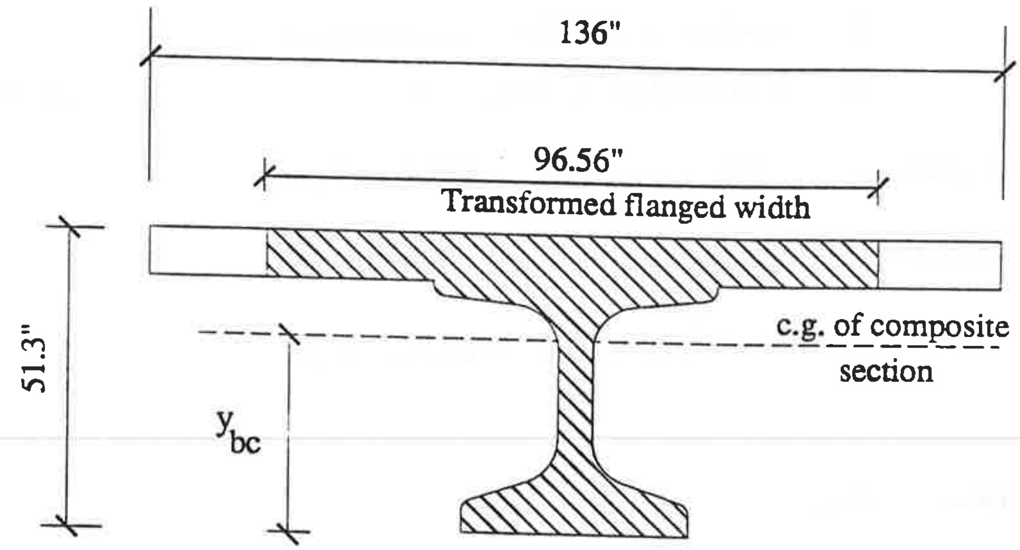


Fig. M.3 Composite Section

Table M.1 Properties of the Composite Section

	Area, in^2	y_b, in.	$A.y_b$, in^3	$(y_{bc}-y_b)$, in.	$A.(y_{bc}-y_b)^2$	I, in^4	$I + A.(y_{bc}-y_b)^2$
girder	694.6	19.6	13,614	14.7	150,096	182,279	333,375
hunch	17.11	43.55	745.14	9.25	1,464	0.0	1464
deck	772.48	47.3	36,538.3	13.0	130,549	4,120	134,669
Σ	1484.2		50,897.4		282,109		468,508.5

A_c = total area of composite section = 1,484.2 in^2

h_c = total height of composite section = 51.8 in.

I_c = moment of inertia of composite section = 468,508.5 in^4

y_{bc} = distance from the centroid of the section to extreme bottom fiber of the composite section = 34.3 in.

y_{tc} = distance from the centroid of the section to extreme top fiber of the composite section = 17.5 in.

y_{tg} = distance from the centroid of the section to top fiber of the girder = 9 in.

S_{bc} = composite section modulus for extreme bottom fiber = 13,659 in^3

S_{tc} = composite section modulus for extreme top fiber = 26,772 in^3

S_{tg} = composite section modulus for top fiber of the girder = 52,056 in^3

4. COMPUTE DESIGN LOADS

Girder self weight and deck slab weight are acting on the non-composite simple span bridge, while the barrier weight, wearing surface weight, and live loads are acting on the composite simple span bridge.

4.1-Dead loads (Art. 3.3)

4.1.1-Dead loads

Uniform non-composite:

Girder weight = = 0.724 kip/ft

$$8'' \text{ slab weight} = 0.150 \times \frac{8}{12} \times 12.0 = 1.2 \text{ kip/ft}$$

$$\text{Haunch (slab build up)} = \frac{1/2 \text{ ave.}}{12} \times 4 \text{ ft} \times 0.150 = 0.025 \text{ kip/ft}$$

Composite:

Distribute to 6 girders

Future Wearing Surface = 0.025 ksf

where, future wearing surface = 25 psf

total width of the bridge = 44 ft

$$\text{Barrier} = 2 \times \frac{330}{4 \text{ girders}} / 1000 = 0.015 \text{ ksf}$$

Total superimposed dead load 0.040 ksf

4.1.2-Dead load bending moments

For a simple span under a uniformly distributed load (W), bending moment (M) at any distance (x) from the support is given by:

$$M_x = \frac{WL^2}{2}[(\frac{x}{L}) - (\frac{x}{L})^2] = \psi\ (WL^2)$$

where the values for ψ are given in Table 2.

Table M.2. Values of ψ

x/L	0.1	0.2	0.3	0.4	0.5
ψ	0.045	0.08	0.105	0.12	0.125

Using the above equation, the bending moment values per girder due to girder weight, deck slab weight, and superimposed dead load are given in Table 3.

Table M.3. Dead load bending moment values per typical interior girder, (kip-ft)

x/L point on the span	Non-composite dead load M_g +	$M_{S+\text{ haunch}}$ =	M_D	Composite dead load M_{SDL}
Transfer	39.2	66.4	105.6	43.1
h/2	53.0	89.7	142.7	52.2
0.10	151.1	256.3	407.8	117.6
0.20	279.2	472.5	751.7	751.7
0.30	370.4	626.9	997.3	997.3
0.40	425.2	719.5	1144.7	1144.7
0.50	443.4	750.4	1193.8	1193.8

4.2-Live load and impact

4.2.1-Live load

Live load shall consist of standard truck or lane load corresponding to HS 25-44.

(Art. 3.7.1.1)

4.2.2-Live load distribution factor

Use live load distribution factor for moment for prestressed precast concrete girder. The fraction of the wheel load carried by the interior girder is:

$$DF_m = \frac{S}{5.5} \qquad \text{(Table 3.23.1)}$$

where S = average spacing between girders in feet.

$$\text{Lanes/Girder} = \frac{1}{2} \times \frac{S}{5.5} = \frac{1}{2} \times \frac{12}{5.5} = 1.09$$

4.2.3-Live load impact (Art. 3.8)

$$I_{impact} = \frac{50}{L+125} \qquad \text{(Eq. 3-1)}$$

where I = impact fraction (maximum 30%)

L = the length in feet of the span under consideration. (Art. 3.8.2.2.a)

$$I_{impact} = \frac{50}{70+125} = 0.256$$

4.2.4-Live load moment

Using CONSPAN for Windows (Version 5.0), the governing live load moment is due to truck load for this span. Design live load moment with impact =

M_{LL+I} = Distribution Factor x (1 + Impact Fraction) x Bending Moment/Lane

Table 4 shows the values of live load bending moment at different sections.

Table M.4. Values of live load bending moment and impacts (kip-ft)

x/L	Transfer	h/2	0.10	0.20	0.30	0.40	0.50
M_{LL+I}	265.9	322	717	1208	1526	1711	1741

5. COMPUTE NUMBER OF STRANDS AND STRAND PATTERN

5.1-Compute stresses due to applied loads

Compute service stress. f_t and f_b are the stresses at the top and bottom of the girder respectively.

Generally $$f = \frac{M}{S} \times 12$$

M = moment, kip-ft

S = section modulus, in^3

Girder and slab $f_b = \frac{1194 \times 12}{9300} = -1.54$ ksi

$f_t = \frac{1194 \times 12}{7691} = +1.86$ ksi

Composite D.L. $f_b = \frac{311.4 \times 12}{13659} = -0.274$ ksi

$f_t = \frac{311.4 \times 12}{26772} = +0.136$ ksi

Live load + I_{impact} $f_b = \frac{1740.7 \times 12}{13659} = -1.53$ ksi

$f_t = \frac{1740.7 \times 12}{26772} = +0.78$ ksi

Total Stress $f_b = -1.54 - 0.274 - 1.53 \quad = -3.344$ ksi (tension)

$f_t = +1.86 + 0.136 + 0.78 \quad = +2.77$ ksi (compression)

5.2-Allowable Stresses

Final at service loads:

$f_b = 6\sqrt{f'_c} = 6\sqrt{8000} \quad = -536.6$ psi (tension) (Art. 9.15.2.2)

For all load combinations:

$f_t = 0.60f''_c = 0.60 \times 8000 = +4{,}800$ psi (comp.) (Art. 9.15.2.2.a)

For prestressing force + dead loads

or for 1/2 (prestressing force + dead loads) + live load:

$f_t = 0.40f''_c = 0.40 \times 8000 = +3{,}200$ psi (comp.) (Art. 9.15.2.2.b,c)

Initial at release

$f_b = 0.60f'_{ci} = 0.6 \times 6000 \quad = +3{,}600$ psi (comp.) (Art. 9.15.2.1)

$f_t = 7.5\sqrt{f'_{ci}} = 7.5\sqrt{6000} \quad = -580$ psi (tension) (Art. 9.15.2.1)

5.3-Estimate approximate number of required strands

To estimate the required number of strands, assume the concrete tensile stresses at bottom fibers, at service load, at section of maximum positive moment govern.

Required compressive stress at bottom fiber

= Total stresses - allowable tension stresses at service

= 3.344 - 0.536 = +2.81 ksi

Assume the location of the strand c.g. at midspan is 5% of the beam depth measured from the bottom of the girder which is = 3 in.

Then, e_c = strand eccentricity at midspan

$= y_b - 4.00 = 19.6 - 3.00 = 16.6$ in.

Bottom fiber stress due to prestress: $f_b = \dfrac{P_{se}}{A_c} + \dfrac{P_{se}\, e_c}{S_b}$

Where P_{se} = effective prestress force after all losses

Then, $2.81 = \dfrac{P_{se}}{694.6} + \dfrac{16.6\ P_{se}}{9300}$

$P_{se} = 871$ kips

Assume final losses @ 25 %

losses = 0.25 x 0.75 x ultimate stress in prestressing steel

losses = 0.25 x 0.75 x 270 = 50.63 ksi.

Then, the final effective prestress force per strand is:

= cross-sectional area of one strand x [0.75 x (ultimate stress) - losses]

= 0.153 (0.75 x 270 - 50.63) = 23.24 kips

Number of strands required = $\dfrac{871}{23.24} = 37.50$

<u>Try 38 - 1/2 in. Φ 270 ksi strands</u>

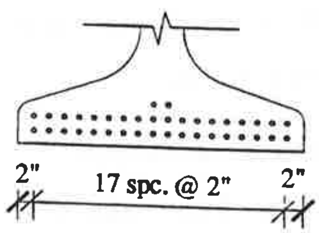
5.4-Strand pattern

For this design, Fig.M.4 shows the assumed strand pattern for the 38 strands at a midspan section.

Calculate the distance from c.g. of the strand to the bottom fiber of the girder (y_{bs}):

$$y_{bs} = \frac{18x1.97 + 18x3.94 + 2x5.9}{38} = 3.11 \text{ in.}$$

$e_c = 19.60 - 3.11 = 16.5$ in.



* Strand spacing is 2 in.

Fig. M.4 Strand Pattern @ Midspan

6. COMPUTE PRESTRESS LOSSES (Art. 9.16.2)

Total losses = SH + ES + CRc + CRs (Eq. 9-3)

where SH = loss of prestress due to concrete shrinkage.

ES = loss of prestress due to elastic shortening.

CRc = loss of prestress due to creep of concrete

CRs = loss of prestress due to relaxation of prestressing steel.

6.1-Shrinkage (Art. 9.16.2.1.1)

assume RH = 70%

SH = 17000 - 150 RH (Eq. 9-4)

= 17000 - 150 x 70 = 6500 psi

6.2-Elastic shortening (Art. 9.16.2.1.2)

$$ES = \frac{E_s}{E_{ci}} f_{cir} \qquad \text{(Eq. 9-6)}$$

Where $f_{cir} = \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{M_g e_c}{I}$ (Art. 9.16)

f_{cir}= average concrete stress at the c.g. of the prestressing steel immediately after transfer. This is due to prestress force and girder self weight.

P_{si}= prestress force after allowing the initial losses (i.e. elastic shorting and strand relaxation). The Code allows the losses due to elastic shortening and strand relaxation to be calculated using 0.69 $f_s^{'}$ tendon stresses.

= (number of strands) x (area of one strand) x (0.75) x (ultimate stresses) x (1 - losses)

= 38 x 0.153 x 0.69 x 270 = 1083 kips

M_g = self weight moment of the girder. = 443.4 kips-ft

e_c = eccentricity of the strand at the mid span = 16.5 in.

$$f_{cir} = \frac{1083}{694.6} + \frac{1083 \text{ x } (16.5)^2}{182279} - \frac{443.4 \text{ x } 12 \text{x} 16.50}{182279} = 1.56 + 1.62 - 0.4815 = 2.698 \text{ ksi}$$

$$ES = \frac{28000}{4696} \text{x} 2698 = 16087 \text{ psi}$$

6.3-Creep of concrete (Art. 9.16.2.1.3)

$CRc = 12 f_{cir} - 7 f_{cds}$ (Eq. 9-9)

Where f_{cds} = average concrete compressive stress at the c.g. of prestressing steel due to all dead loads except the dead loads present at the time that the prestressing force is applied.

$$= \frac{M_S e_c}{I} + \frac{M_{SDL} (y_{bc} - y_{bs})}{I_c}$$

M_S = slab dead load moment = 750.4 kip-ft

M_{SDL}= super-imposed dead load moment = 311.4 kip-ft

y_{bc} = 34.3 in.

y_{bs} = the distance from c.g. of the strand at midspan to the bottom of the girder = 3.11 in.

I = moment of inertia for the non-composite section = 182,279 in^4

I_c = moment of inertia for the composite section = 468,508.5 in^4

$$f_{cds} = \frac{750.4 \times 12 \times 16.5}{182279} + \frac{311.4 \times 12 \times (34.3 - 3.11)}{468508.5} = 1.047 \text{ ksi}$$

CRc = 12 x 2600 - 7 x 1047 = 24,591 psi

6.4-Relaxation of prestressing steel (Art. 9.16.2.1.4)

CRs = 5000 - 0.10 ES - 0.05(SH + CRc) = (Eq. 9-10A)

= 5000 - 0.10 x 16087 - 0.05(6500 + 24591) = 1837 psi

6.5-Total initial losses of prestress

ES = 15773.4 psi

Total initial losses = 16087 psi

f_{si} = effective initial prestress = 0.75x270 - 16.087 = 186.4 ksi

P_{si} = 38 x 0.153 x 1864 = 1084 kips

6.6-Total final loss of prestress

SH = 6500 psi

ES = 15773.4 psi

CRc = 24591 psi

CRs = 1837 psi

Total finale losses = 6500 + 15773.4+ 24591+ 1837 = 48701 psi

or $\frac{44.654}{0.75 \times 270}$ x 100 = 24.1% losses

f_{se} = effective final prestress = 0.75x270 - 48.7 = 153.8 ksi

P_{se} = 38 x 0.153 x 153.8 = 894 kips

7. COMPUTE INITIAL STRESSES AT PRESTRESS TRANSFER

7.1-Allowable stresses: (Art. 9.15.2.1)

Compression	$= 0.6\, f'_{ci}$	$= +\, 3.600$ ksi (comp.)
Tension	= the maximum tensile stress shall not exceed	
	$= 7.5\sqrt{f'_{ci}} = 7.5\sqrt{6000} = -\, 0.581$ ksi (tension)	
	= with no bonded reinforcement	
	$= 200$ psi or $3\sqrt{f'_{ci}}$	$= -\, 0.232$ ksi (tension)

If the calculated tensile stress exceeds whichever is smaller, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

7.2-At midspan section

$$f_t = +\frac{P_{si}}{A} - \frac{P_{si}\, e_c}{S_t} + \frac{M_g}{S_t}$$

$$f_b = +\frac{P_{si}}{A} + \frac{P_{si}\, e_c}{S_b} - \frac{M_g}{S_b}$$

Girder weight moment, $M_g = 443.4$ kips-ft

$$f_t = +\frac{1084}{694.6} - \frac{1084 \times 16.5}{7691} + \frac{443.4 \times 12}{7691}$$

$= +\, 1.56 - 2.32 + 0.69$ $= -\, 0.03$ ksi $< -\, 0.58$ ksi O.K.

$$f_b = +\frac{1084}{694.6} + \frac{1084 \times 16.5}{9300} - \frac{443.4 \times 12}{9300}$$

$= +\, 1.56 + 1.92 - 0.572$ $= +\, 2.86$ ksi $< +\, 3.600$ ksi O.K.

7.3-At transfer length section

Transfer length	= 50 (strand diameter)	(Art. 9.20.2.4)
	= 50 x 0.50	= 25 inches = 2.1 ft
	@ point 0.03	

$$f_t = +\frac{P_{si}}{A} - \frac{P_{si}\, e_e}{S_t} + \frac{M_g}{S_t}$$

$$f_b = +\frac{P_{si}}{A} + \frac{P_{si}\, e_e}{S_b} - \frac{M_g}{S_b}$$

Bending moment at transfer length section:

M_g = 39.2 kips-ft (@ 0.03 L from Table 3)

$$f_t = +\frac{1084}{694.6} - \frac{1084 \times 16.5}{7691} + \frac{39.2 \times 12}{7691}$$

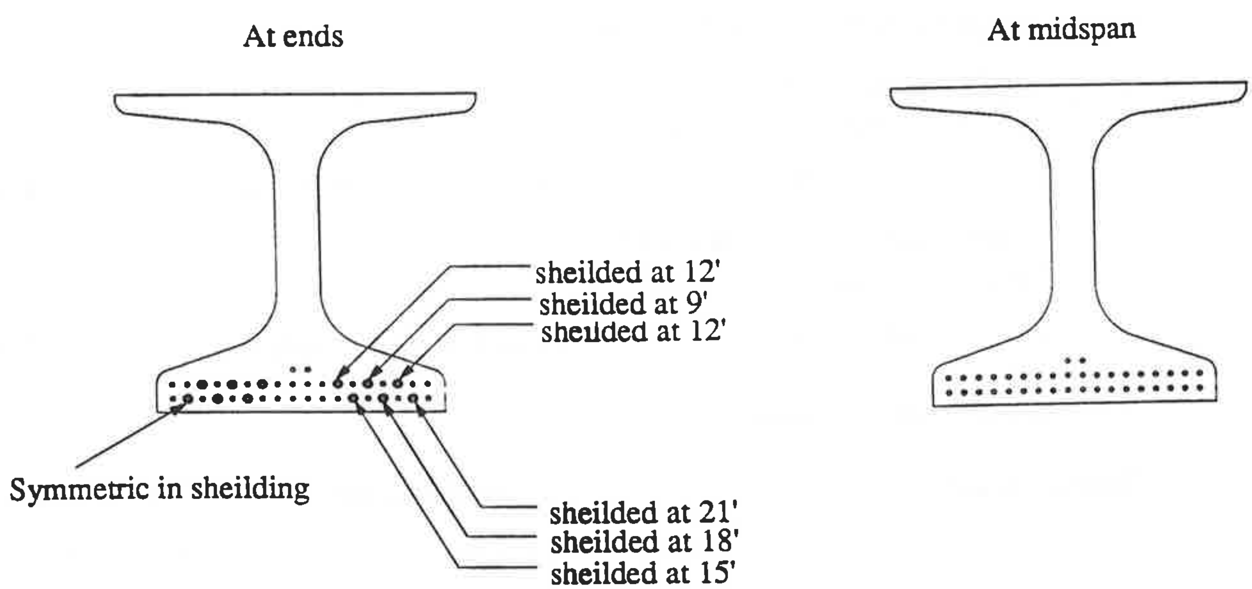
$= + 1.56 - 2.32 + 0.06$ $\quad = - 0.71$ ksi $\quad > - 0.581$ ksi $\quad$ N.G.

$$f_b = +\frac{1084}{694.6} + \frac{1084 \times 16.5}{9300} - \frac{39.2 \times 12}{9300}$$

$= + 1.56 + 1.92 - 0.05$ $\quad = + 3.42$ ksi $\quad < + 3.60$ ksi $\quad$ O.K.

Since the top fiber stresses exceed the allowable, draped and/or debonded strands should be used. Debond 12 Strands.

Assume the following strand pattern.



* Strand spacing is 2 in.

Fig. M.5 Strand Pattern

Compute c.g. of strand pattern at ends:

$$c.g. = \frac{12x1.97 + 12x3.94 + 2x5.9}{38} = 3.2 \text{ in.}$$

eccentricity of the strands at end of the span:

$e_e = y_b - c.g. = 19.6 - 3.2 = 16.4$ in.

$$f_t = +\frac{1084 \text{ x } 26}{694.6 \text{ x } 38} - \frac{1084 \text{ x } 16.4 \text{ x } 26}{7691 \text{ x } 38} + \frac{39.2 \text{ x } 12}{7691}$$

$= + 1.07 - 1.583 + 0.05$ $= - 0.453$ ksi $< - 0.581$ ksi O.K.

$$f_b = +\frac{1084 \text{ x } 26}{694.6 \text{ x } 38} - \frac{1084 \text{ x } 16.4 \text{ x } 26}{9300 \text{ x } 38} - \frac{39.2 \text{ x } 12}{9300}$$

$= + 1.123 + 1.31 - 0.05$ $= + 2.38$ ksi $< + 3.600$ ksi O.K.

7.4-At debonded section (0.3 L)

Bending moment due to girder weight at draped section:

M_g = 997.3 kips-ft (@ 0.3 L from Table 3)

$$f_t = +\frac{1084}{694.6} - \frac{1084 \text{ x } 16.5}{7691} + \frac{997.3 \text{ x } 12}{7691}$$

$= + 1.56 - 2.32 + 1.56$ $= + 0.8$ ksi $< + 3.600$ ksi O.K.

$$f_b = +\frac{1084}{694.6} + \frac{1084 \text{ x } 16.5}{9300} - \frac{997.3 \text{x} 12}{9300}$$

$= + 1.56 + 1.92 - 1.29$ $= + 2.19$ ksi $< + 3.60$ ksi O.K.

8. SERVICE LOAD STRESSES AT MIDSPAN

Effective prestress force after all losses:

$P_{se} = 38 \text{ x } 0.153 \text{ x } 153.8 = 894$ kips

8.1-Allowable stresses (Art. 9.15.2.2)

Compression

for all load combinations:

$$= 0.60f''_c = 0.60 \times 8000 \qquad = +\ 4{,}800 \text{ psi (comp.)}$$

for prestressing force + dead loads

or for 1/2 (prestressing force + dead loads) + live load

$$= 0.40\ f''_c = 0.40 \times 8000 \qquad = +\ 3{,}200 \text{ psi (comp.)}$$

$$\text{Tension} \qquad = 6\sqrt{f'_c} \qquad = -\ 536.7 \text{ psi (tension)}$$

8.2-At 1.4L section & midspan section

8.2.1- At 1.4L

From Table 3 and 4, the values of loads bending moments at this section are:

M_D = 1144.7 kips-ft

M_{SDL} = 299.2 kips-ft

M_{LL+I} = 1711 kips-ft

$$f_t = +\frac{P_{se}}{A} - \frac{P_{se} \times e_c}{S_t} + \frac{M_D}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tc}}$$

$$f_b = +\frac{P_{se}}{A} + \frac{P_{se} \times e_c}{S_b} - \frac{M_D}{S_b} - \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

$$f_t = +\frac{894}{694.6} - \frac{894 \times 16.5}{7691} + \frac{1144.7 \text{x} 12}{7691} + \frac{(299.2 + 1711.1) \text{x} 12}{52056}$$

For all loads combinations

$= +\ 1.287 - 1.92 + 1.786 + 0.069 + 0.3944$

$= +\ 1.62$ ksi $< +\ 4.800$ ksi O.K.

For prestress force + dead loads

$= +\ 1.287 - 1.92 + 1.786 + 0.069 = +\ 1.22$ ksi $< +\ 3.200$ ksi O.K.

For 1/2 (pretress force + dead loads)

$= 1/2 \times (+\ 1.287 - 1.92 + 1.786 + 0.069) + 0.3944 = +\ 1.007$ ksi $< +\ 3.200$ ksi O.K.

$$f_b = +\frac{894}{694.6} - \frac{894 \times 16.5}{9300} + \frac{1144.7 \text{x} 12}{9300} + \frac{(299.2 + 1711.1) \text{x} 12}{13659}$$

$= +\ 1.287 + 1.586 - 1.477 - 1.766$

$= -\ 0.361$ ksi $< -\ 0.536$ ksi O.K.

8.2.2- At midspan

From Table M.3 and M.4, the values of bending moments at this section are:

M_D = 1193.8 kips-ft

M_{SDL} = 311.4 kips-ft

M_{LL+I} = 1740.7 kips-ft

$$f_t = +\frac{894}{694.6} - \frac{894 \times 16.5}{7691} + \frac{1144.7x12}{7691} + \frac{(311.4 + 1740.7)x12}{52056}$$

For all loads combinations:

= + 1.287 - 1.92 + 1.786 + 0.072 +0.40

= + 1.625 ksi < + 4.800 ksi O.K.

For prestress force + dead loads:

= + 1.287 - 1.92 + 1.786 + 0.072 = + 1.225 ksi < + 3.200 ksi O.K.

For 1/2 (pretress force + dead loads):

= 1/2 x (+ 1.287 - 1.92 + 1.786 + 0.072) + 0.4 = + 1.0125 ksi < + 3.200 ksi O.K.

$$f_b = +\frac{894}{694.6} - \frac{894 \times 16.5}{9300} + \frac{1144.7x12}{9300} + \frac{(311.4 + 1740.7)x12}{13659}$$

= + 1.287 + 1.586 - 1.477 - 1.80

= - 0.40 ksi < - 0.536 ksi O.K.

8.3-At transfer length section

From Table M.3 and M.4, the values of bending moments at this section are:

M_D = 105.6 kips-ft

M_{SDL} = 43.1 kips-ft

M_{LL+I} = 265.9 kips-ft

$$f_t = +\frac{P_{se}}{A} - \frac{P_{se}\ e_e}{S_t} + \frac{M_D}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tc}}$$

$$f_b = +\frac{P_{se}}{A} + \frac{P_{se}\, x e_e}{S_b} - \frac{M_D}{S_b} - \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

$$f_t = +\frac{894}{694.6} x 0.68 - \frac{894 \times 16.5}{7691} x 0.68 + \frac{105.6x12}{7691} + \frac{(43.1 + 265.9)x12}{52056}$$

For all loads combinations

= + 0.88 - 1.30 + 0.164 + 0.010 +0.0613

= - 0.19 ksi < - 0.581 ksi O.K.

For prestress force + dead loads

$= + 0.88 - 1.30 + 0.164 + 0.010 = - 0.1287$ ksi $< - 0.581$ ksi O.K.

For 1/2 (pretress force + dead loads)

$= 1/2 \times (+ 0.88 - 1.30 + 0.164 + 0.010) + 0.0613 = - 0.003$ ksi $< - 0.581$ ksi O.K.

$$f_b = +\frac{894}{694.6} x\,0.68 + \frac{894\,x\,16.5}{9300} x\,0.68 - \frac{105.6x12}{9300} - \frac{(43.1 + 265.9)x12}{13659}$$

$= + 0.88 + 1.07 - 0.136 - 0.272$

$= + 1.542$ ksi $< +4.800$ ksi O.K.

8.4-At debonded section (0.2 L)

From Table M.3 and M.4, the values of bending moments at this section are:

$M_D = 752$ kips-ft

$M_{SDL} = 202.4$ kips-ft

$M_{LL+I} = 1208$ kips-ft

$$f_t = +\frac{P_{se}}{A} - \frac{P_{se}\,x\,e_c}{S_t} + \frac{M_D}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tc}}$$

$$f_b = +\frac{P_{se}}{A} + \frac{P_{se}\,x\,e_c}{S_b} - \frac{M_D}{S_b} - \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

$$f_t = +\frac{894}{694.6} - \frac{894\,x\,16.4}{7691} + \frac{752x12}{7691} + \frac{(202.4 + 1208)x12}{52056}$$

For all loads combinations:

$= + 1.29 - 1.90 + 1.173 + 0.047 + 0.278$

$= +0.88$ ksi $< + 4.800$ ksi O.K.

For prestress force + dead loads:

$= + 1.29 - 1.90 + 1.173 + 0.047 = + 0.61$ ksi $< + 4.800$ ksi O.K.

For 1/2 (pretress force + dead loads):

$= 1/2 \times (1.29 - 1.90 + 1.173 + 0.047) + 0.278 = +0.583$ ksi $< +4.800$ ksi O.K.

$$f_b = +\frac{894}{694.6} + \frac{894\,x\,16.4}{9300} - \frac{752x12}{9300} - \frac{(202.4 + 1208)x12}{13659}$$

$= + 1.29 + 1.58 - 0.97 - 1.24$

$= +0.66$ ksi $< +4.800$ ksi O.K.

8.5-Summary of final stresses

At transfer section: f_t = -0.19 ksi

f_b = + 1.542 ksi

At debonded section: f_t= + 0.88 ksi

f_b = + 0.66 ksi

At midspan section: f_t = + 1.625 ksi

f_b = - 0.40 ksi

9. CHECK GIRDER FOR THE ULTIMATE POSITIVE MOMENT (Art. 3.22)

Using Group I load factor design loading combination:

$$M_u = 1.3\ [M_D + M_{SDL} + \frac{5}{3}(M_{LL+I})] \qquad \text{(Table 3.22.1.A)}$$

$$= 1.3\ (1193.8 + 311.4 + \frac{5}{3} \times 1740.7) = 5728 \text{ kip-ft}$$

Locate the depth of compression block assuming a rectangular section:

$$a = \frac{A_s * f_{su} *}{0.85 \times f'_c\ b} \qquad \text{(Art. 9.17.2)}$$

$$f_{su}* = f'_s(1 - \frac{\gamma *}{\beta_1}\rho * \frac{f'_s}{f'_{cs}}) \qquad \text{(Art. 9.17.4.1)}$$

$f_{su}*$ = average stress in prestressing reinforcement at ultimate load.

f'_s = ultimate stress of prestressing steel = 270 ksi

$\gamma*$ = 0.28 for low relaxation strand. (Art. 9.17)

β_1 = 0.85 - 0.05 (f'c - 4000) when f'c ≥ 4000 psi (Art. 8.16.2.7)

= 0.85 for cast-in -place concrete strength.

f'_{cs} = concrete strength of compression fiber. = 4.0 ksi

$$\rho* = \frac{A_s *}{bd}$$

A_s*= area of prestressed reinforcement. = 38 x 0.153 = 5.814 in^2

b = effective flange width = 136 in.

d = distance from top of slab to centroid of prestressing strands

= girder depth (h) + slab thickness - y_{bs} = 43.3 + 8 - 3.32 = 48.0 in.

y_{bs} = distance from c.g. of the strand to the bottom fiber of the girder

$$\rho^* = \frac{5.84}{136 \times 48.0} = 0.00089$$

$$f_{su}^* = 270\left(1 - \frac{0.28}{0.85} \text{x} 0.00089 \text{x} \frac{270}{4}\right) = 264.65 \text{ ksi}$$

$$a = \frac{5.814 \times 264.65}{0.85 \times 4 \times 136} = 3.32 \text{ in} \quad < 8.0 \text{ in}$$

Compression block is within the deck slab thickness, therefore, this section is considered a rectangular section.

Ultimate moment capacity of rectangular section:

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

Where ϕ = strength reduction factor

M_n = nominal moment strength of a section.

$$= 1.0 \text{ x } 5.814 \text{ x } 264.65 \text{ x } 48 \left(1 - 0.6 \text{x} \frac{0.00089 \times 264.65}{4}\right) / 12$$

$= 5937$ kip-ft $> M_u = 5728$ kip-ft

10. DUCTILITY LIMITS

10.1-Maximum steel (Art. 9.18.1)

Prestressed concrete members shall be designed so that the steel is yielding as ultimate capacity is approached.

Reinforcement index for rectangular section

$$= \rho^* \frac{f_{su}^*}{f'_{cs}} < 0.36\beta_1$$

$$= 0.00089 \text{ x} \frac{264.65}{4} = 0.059 < 0.36 \text{ x } 0.85 = 0.306 \qquad \text{O.K.}$$

10.2-Minimum steel (Art. 9.18.2)

The total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment.

$\phi M_n \geq 1.2 M_{cr}^*$

Compute cracking moment:

$$M_{cr}^* = (f_{cr} + f_{pe})S_{cb} - M_D(\frac{S_{cb}}{S_b} - 1)$$

Where M_{cr}^* = moment causing flexural cracking at section due to externally applied loads (Art. 9.20)

f_{cr} = modulus of rupture (Art. 9.15.2.3)

$= 7.5\sqrt{f_c'} = 7.5\sqrt{8000} = 671$ psi

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads.

$$= \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

P_{se} = effective prestress force after losses

= 894 kips

e_c = 16.5 in.

S_{bc} = section modulus of composite section = 13,659 in^3

S_b = section modulus of non-composite section = 9300 in^3

$$f_{pe} = \frac{894}{694.6} + \frac{894 \text{ x } 16.50}{9300} = 1.29 + 1.585 = 2.875 \text{ ksi}$$

M_D = moment due to self weight and slab = 1193.8 kip-ft

$$M_{cr}^* = (0.671 + 2.875)\ \frac{13659}{12} - 1193.8 \text{ x } (\frac{13{,}659}{9300} - 1) = 3453.9 \text{ kip-ft}$$

$1.2\ M_{cr}^* = 4144.7$ kip-ft $< \phi M_n = 5937$ kip-ft

11. VERTICAL SHEAR

(Art. 9.20)

The vertical shear reinforcement has been designed in unique way to serve other research purposes. However, this design has been checked and found to meet all the design requirements required by the AASHTO Standard Specifications.

12. HORIZONTAL SHEAR

The critical section is at a distance of h/2 from the support.

$V_u = 325.6$ kips

$V_{u\,(Composite)} = 242$ Kips

V_{nh} is nominal horizontal shear strength

$$V_{nh} \geq \frac{V_u}{\phi} = \frac{242}{0.9} = 268.9 \text{ kips}$$

$\upsilon_{nh} = \mu f_{cl}$ (psi) , μ=1

$$\upsilon_{nh} \geq \frac{Vnh}{0.9\, d\, b_v} = \frac{268.9 \, x \, 1000}{0.9 \, x \, 48 \, x \, 40} = 156 \text{ psi}$$

$$\upsilon_{nh} = f_{cl} = \frac{A_v f_y}{S\, b_v}$$

$A_v = 1.24$ in^2 / one connector

$$S = \frac{A_v f_y}{\upsilon_{nh}\, b_v}$$

$$S = \frac{1.24 \, x \, 60{,}000}{40 \, x \, 156} = 12 \text{ in.}$$

At a distance = 0.2L

$V_{u\,(Composite)} = 194$ Kips

$V_{nh} = 215$ Kips

$\upsilon_{nh} = 124$ psi

S = 15 in.

At a distance = 0.3L

$V_{u\,(Composite)} = 164$ Kips

$V_{nh} = 182$ Kips

$\upsilon_{nh} = 105$ psi

S = 18 in.

At a distance = 0.4L

$V_{u\,(Composite)} = 124$ Kips

$V_{nh} = 138$ Kips

$\upsilon_{nh} = 79$ psi

$S = 24$ in.

At a distance = 0.5L

$V_{u\ (Composite)} = 92$ Kips

$V_{nh} = 102$ Kips

$\upsilon_{nh} = 59$ psi

$S = 32$ in.

To maintain uniform spacing of 2 feet between the connectors, type II connectors were used where necessary. Type II connectors provide double the amount of steel available therefore they will double the spacing needed, S' = 2S. Also, redistribution of horizontal shear stresses was assumed as shown in Figure M.6.

Use Type II Connectors in the first and the last 14 ft of the girder. Use Type I Connector in between.

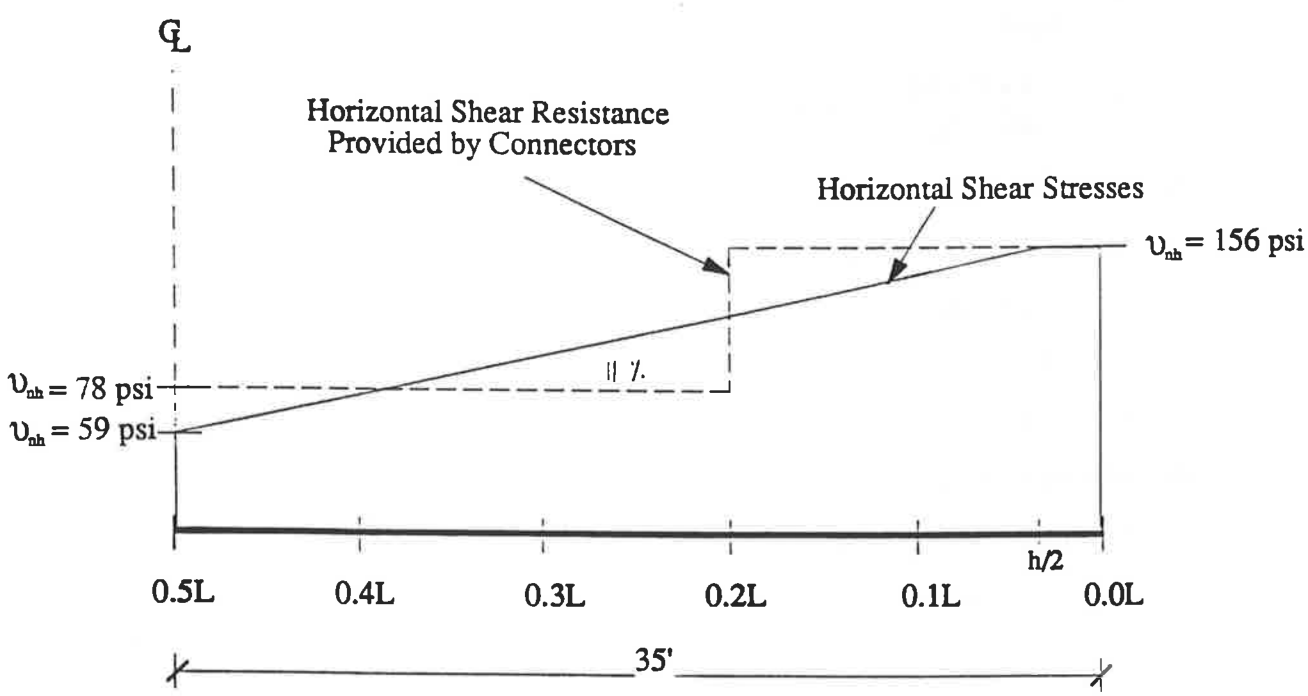


Fig. M.6 Redistribution of Horizontal Shear Stresses

APPENDIX N

Test Program of

Steel Girder-to-Deck Connection

Overview of Experimental Program

The test program for the steel girder-to-deck connection consisted of a series of push-off specimens for a total of 34 specimens. The tested connection schemes were 7/8 in. (22.22 mm) and 1-$^1/_4$ in. (31.8 mm) diameter welded steel studs. The first series consisted of eight push-off specimens with 1-$^1/_4$ in. (31.8 mm) diameter welded headed studs. All the specimens in this group were tested for fatigue until failure. The second series consisted of eight push-off specimens with 1-$^1/_4$ in. (31.8 mm) diameter welded headed and headless studs. Four specimens were tested for ultimate strength and four tested for fatigue strength, which then were tested for ultimate strength to study the effect of the fatigue on the ultimate strength. The third series consisted of four push-off specimens with 7/8 in. (22.22 mm) diameter welded studs tested for ultimate strength only. The fourth series consisted of four push-off specimens with 1-$^1/_4$ in. (31.8 mm) diameter welded headed studs. The 1-$^1/_4$ in. (31.8 mm) studs in the fourth series replaced existing 7/8 in. (22.22 mm) diameter welded studs in the second group after they had been tested. Two specimens were tested for ultimate strength and two tested for fatigue strength, which then were tested for ultimate strength. The fifth series consisted of four push-off specimens with 1-$^1/_4$ in. (31.8 mm) diameter welded studs designed to be tested in way that would minimize the eccentricity of the applied load. Fully anchored reinforcing bars consisting of #5 bars at 6" spacing were used in the transverse direction in two specimens. The transverse reinforcement in the other two specimens was the same as the specimens in the other groups. A tensile stress of 10 ksi was applied to the reinforcing bars when they were anchored at their ends to provide the anchorge length needed for these reinforcing bars.

The effects on ultimate strength of using a combination of 1-$^1/_4$ in. (31.8 mm) diameter welded headed (T) and headless (I) studs and the effect of replacing existing 7/8

in. (22.22 mm) studs with new 1-$^{1}/_{4}$ in. (31.8 mm) diameter welded headed studs are discussed in the accompanying text.

Designations, descriptions and test types for each series are listed in Table N.1. Details of the specimens, test setups and procedures, and test results are shown in Figs. N.1 through N.7. and Tables N.2 and N.3.

Table N.1 Designation and Description of Push-off Specimens

Series	Designation	Description	Test Type
I	TT1.... ....TT8	Two 1-$^{1}/_{4}$ in. headed studs	Fatigue until failure
II	TTTT-1	Four 1-$^{1}/_{4}$ in. headed studs	Ultimate Strength
	TTTT-2	Four 1-$^{1}/_{4}$ in. headed studs	Ultimate Strength
	TITI-1	Pattern of four alternating headed and headless 1-$^{1}/_{4}$ in. studs	Ultimate Strength
	TIIT-1	One headed 1-$^{1}/_{4}$ in. studs two headless 1-$^{1}/_{4}$ in. studs one headed 1-$^{1}/_{4}$ in. studs	Ultimate Strength
	TTTT-3	Four 1-$^{1}/_{4}$ in. headed studs	Fatigue then Ultimate
	TTTT-4	Four 1-$^{1}/_{4}$ in. headed studs	Fatigue then Ultimate
	TITI-2	Pattern of four alternating headed and headless 1-$^{1}/_{4}$ in. studs	Fatigue then Ultimate
	TIIT-2	One headed 1-$^{1}/_{4}$ in. studs two headless 1-$^{1}/_{4}$ in. studs one headed 1-$^{1}/_{4}$ in. studs	Fatigue then Ultimate
III	tttt-1	2 rows of 7/8 in. shear studs placed at 6 in. spacing	Ultimate Strength

Series	Designation	Description	Test
	tttt-2	2 rows of 7/8 in. shear studs placed at 6 in. spacing	Ultimate Strength
	tttt-3	2 rows of 7/8 in. shear studs placed at 6 in. spacing	Ultimate Strength
	tttt-4	2 rows of 7/8 in. shear studs placed at 6 in. spacing	Ultimate Strength
IV	RTTTT-1	Four 1-$^{1}/_{4}$ in. studs replacing existing 7/8 in. studs	Ultimate Strength
	RTTTT-2	Four 1-$^{1}/_{4}$ in. studs replacing existing 7/8 in. studs	Ultimate Strength
	RTTTT-3	Four 1-$^{1}/_{4}$ in. studs replacing existing 7/8 in. studs	Fatigue then Ultimate
	RTTTT-4	Four 1-$^{1}/_{4}$ in. studs replacing existing 7/8 in. studs	Fatigue then Ultimate
V	ATITI-1	Pattern of alternating headed and headless 1-$^{1}/_{4}$ in. studs with fully anchored reinforcing bars, reduced eccentricity.	Ultimate Strength
	ATITI-2	Pattern of alternating headed and headless 1-$^{1}/_{4}$ in. studs with fully anchored reinforcing bars, reduced eccentricity.	Ultimate Strength
	STITI-1	Pattern of alternating headed and headless 1-$^{1}/_{4}$ in. studs, reduced eccentricity.	Ultimate Strength
	STITI-2	Pattern of alternating headed and headless 1-$^{1}/_{4}$ in. studs, reduced eccentricity.	Ultimate Strength

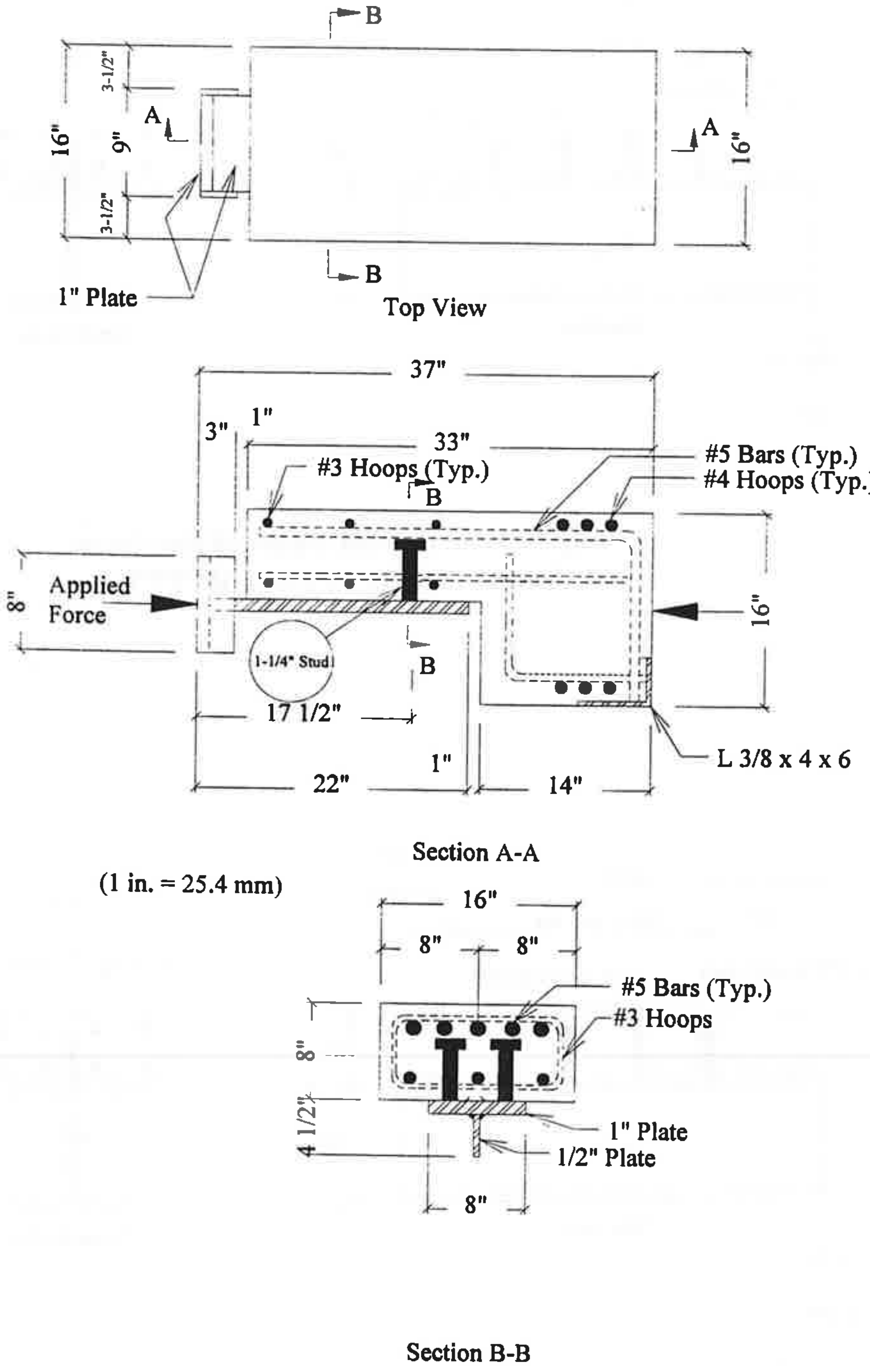


Fig. N.1 Steel Stud Push-off Specimen
Series I, (TT)

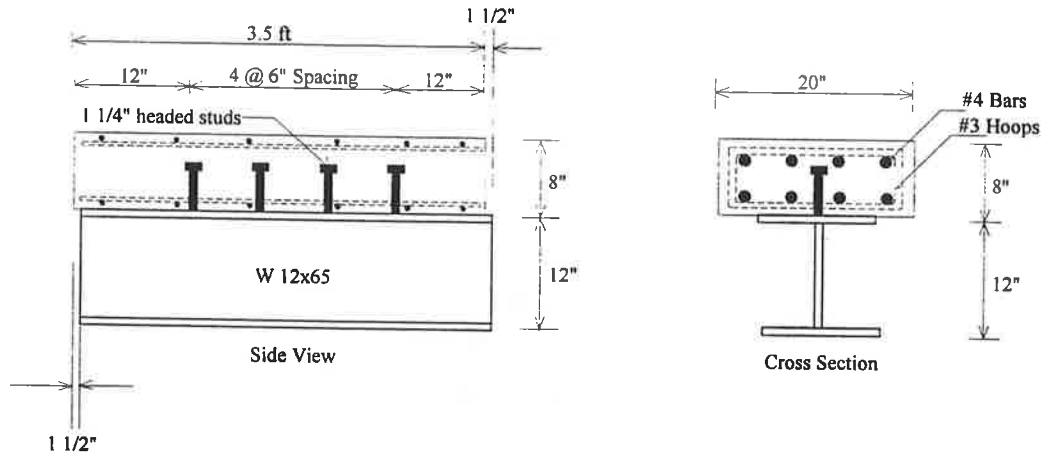


Fig. N.2 Steel Stud Push-off Specimen Series II&IV, (TTTT & RTTTT)

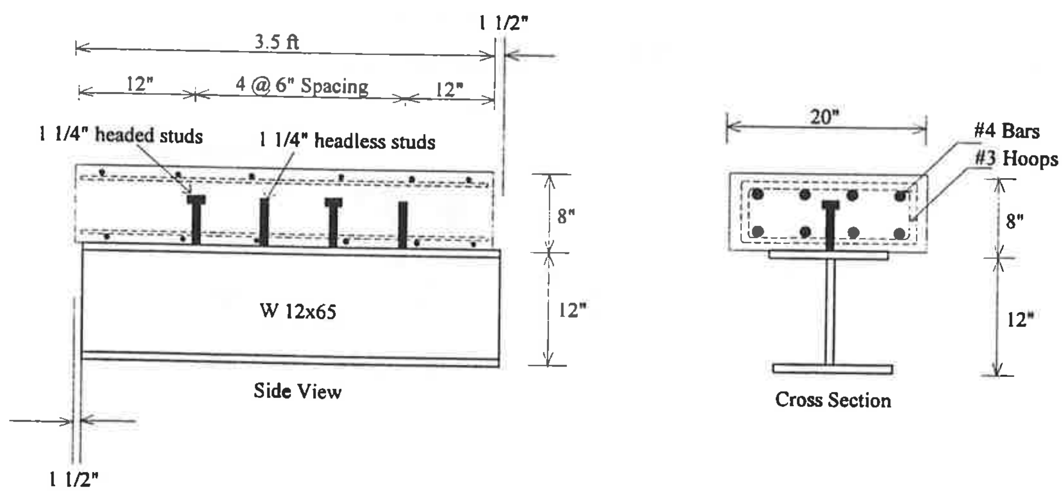


Fig. N.3 Steel Stud Push-off Specimen Series II, (TITI)

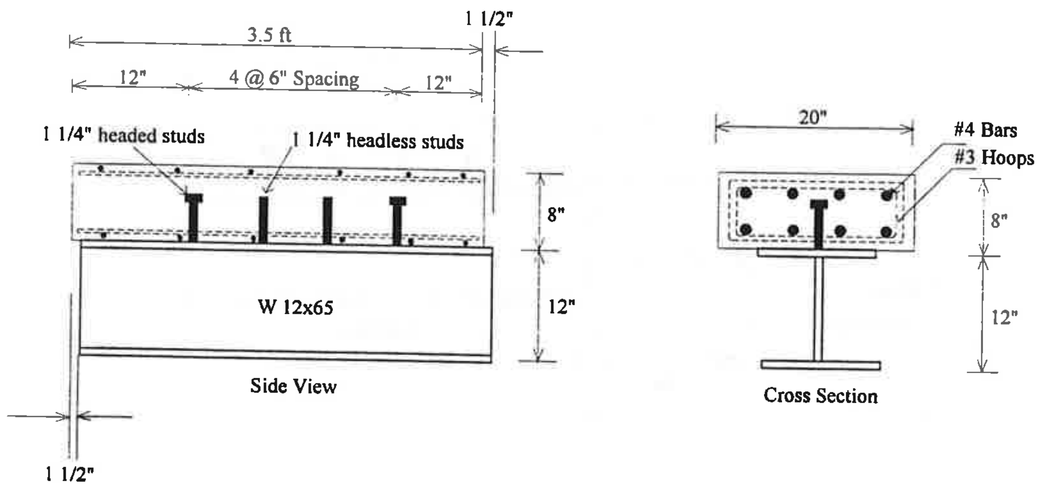


Fig. N.4 Steel Stud Push-off Specimen Series II, (TIIT)

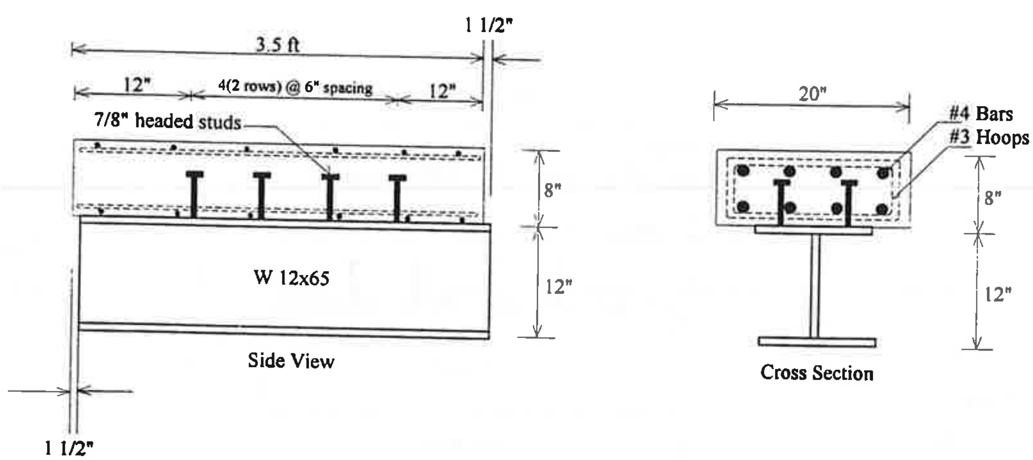


Fig. N.5 Steel Stud Push-off Specimen Series III, (tttt)

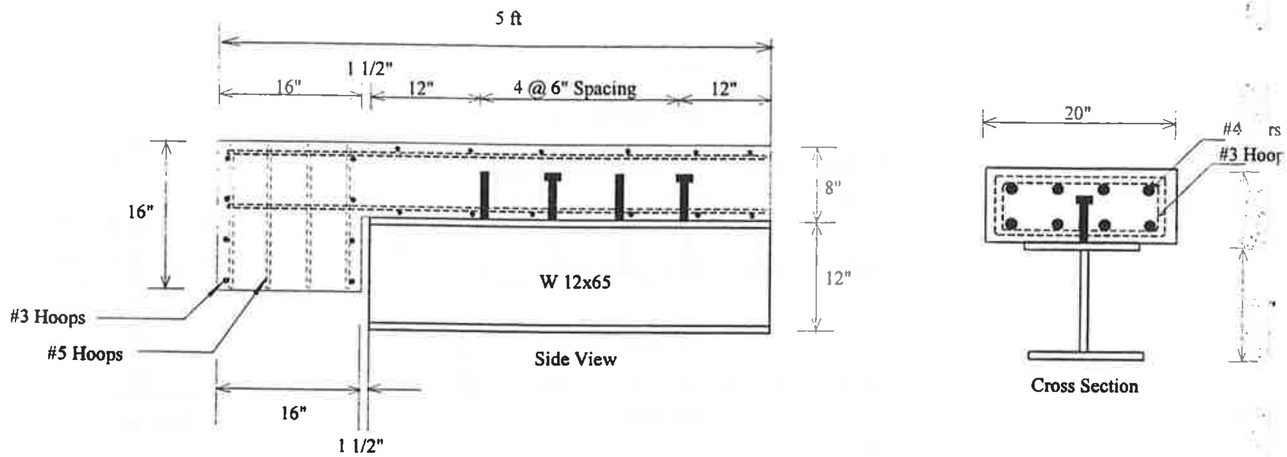


Fig. N.6 Steel Stud Push-off Specimen Series V, (STITI)

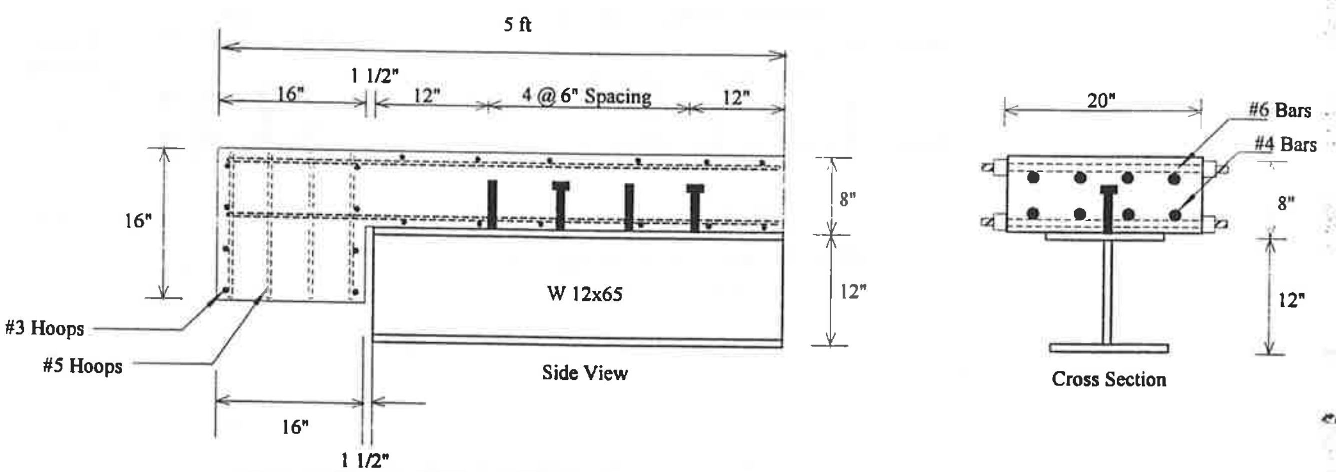


Fig. N.7 Steel Stud Push-off Specimen Series V, (ATITI)

Fabrication of Test Specimen

Welding of both 7/8 in. (22.2 mm) and 1-$^1/_4$ in. (31.8 mm) diameter studs was done by Tri-Sales Associates in Omaha, Nebraska. The reinforcement for the specimens was assembled and placed into the plywood formwork. Concrete was placed and consolidated around the reinforcement using a small spud vibrator. The concrete was covered with wet burlap and allowed to cure for four days before the formwork was removed.

Description of Tests and Test Set-up

Fatigue Tests: The first series of fatigue tests was conducted on specimens of the first group. In order to determine an allowable fatigue stress range for 1-$^1/_4$ in. (31.8 mm) diameter studs, the push-off specimens were subjected to different stress ranges until the specimen failed in horizontal shear. A minimum applied stress of 10 ksi (0.07 MPa) for all specimens was adopted from previous research by Slutter et al. (1966) which was conducted on smaller diameter studs. Another series of fatigue tests was conducted on four specimens of the second group and on two specimens of the fourth group. One reason behind this series was to determine the validity of the new α values which were established from Series I. Another reason was to study the effect of fatigue on the ultimate strength. Table N.2 gives the fatigue stress range for each specimen tested.

A photo of the test set up for the fatigue tests of 1-$^1/_4$ in. (31.8 mm) diameter studs is shown in Fig. N.8. An MTS actuator was positioned in the steel frame and aligned with the push-off specimen. The fatigue load for Series I was applied at a frequency of 8 to 10

Table N.2 Steel Stud Fatigue Specimen

Specimen No.	Stress Range (ksi)	f_{min} Minimum Stress (ksi)	f_{max} Maximum Stress (ksi)
Series I TT1	15.00	10	25.00
TT2	16.25	10	26.25
TT3	17.50	10	27.50
TT4	18.75	10	28.75
TT5	20.00	10	30.00
TT6	21.25	10	31.25
TT7	22.50	10	32.50
TT8	25.00	10	35.00
Series II (all specimens)	18.33	10	28.33

cycles per second depending on the applied stress range. The test was run continuously until the specimens failed in horizontal shear. The fatigue load for Series II was applied at a frequency of 7 cycles per second. The test was run continuously until 2,000,000 cycles were applied to the specimen.

Ultimate Tests: The rest of the specimens were tested to determine the ultimate strength of the 1-$^{1}/_{4}$ in. (31.8 mm) shear studs. The test set up for the ultimate strength is shown in Fig. N.9 for Series I, II, III, and IV, and in Fig. N.10 for Series V. The horizontal load was applied by two 400 kip (1779 kN) hydraulic jacks. Specimens were sheared all the way after failure to check where the failure of the stud occurred. The relative horizontal slip between the CIP concrete slab and the steel girder was measured using Linear Variable Differential Transducers(LVDT). An LVDT gage was mounted on each side of the specimen. Computer data acquisition methods were used to record the relative horizontal slip and the applied load at one second intervals during the test. The

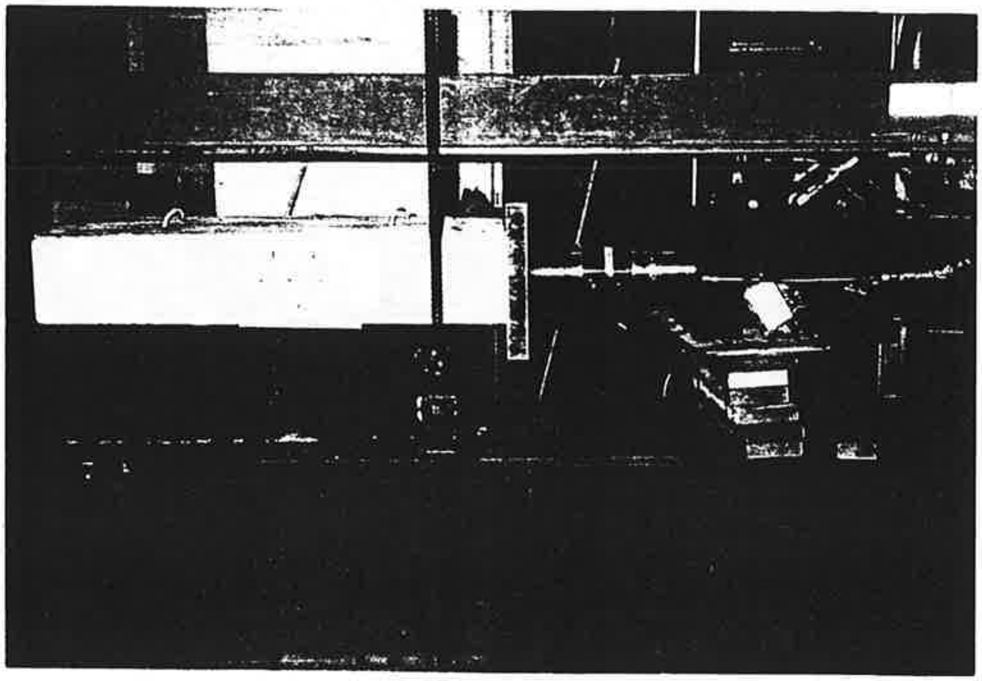


Fig. N.8 Fatigue Test

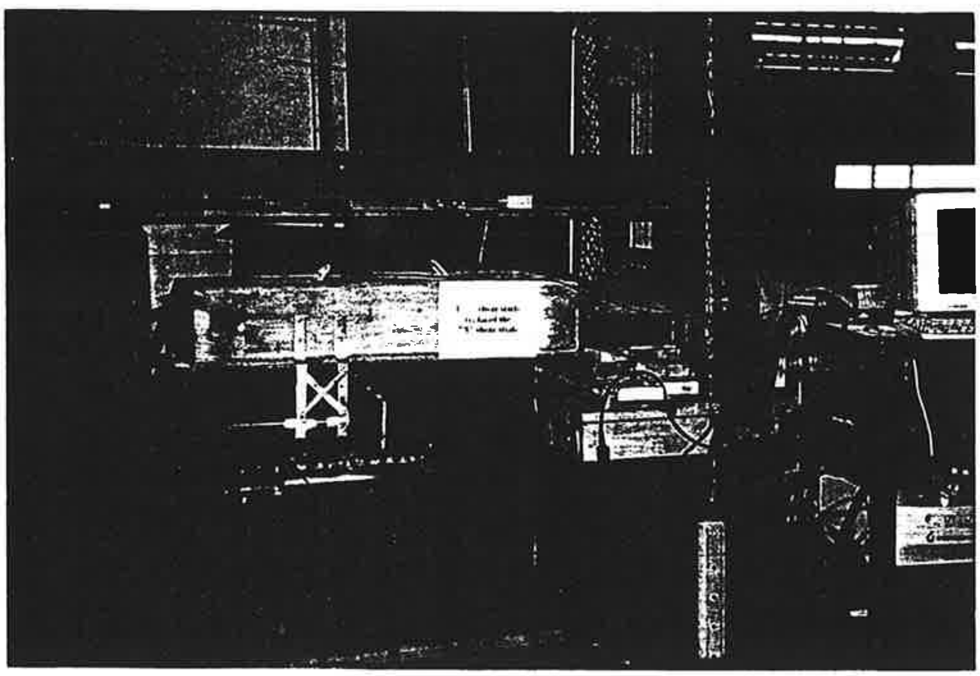


Fig. N.9 Ultimate Strength Test for Series I, II, III, and IV

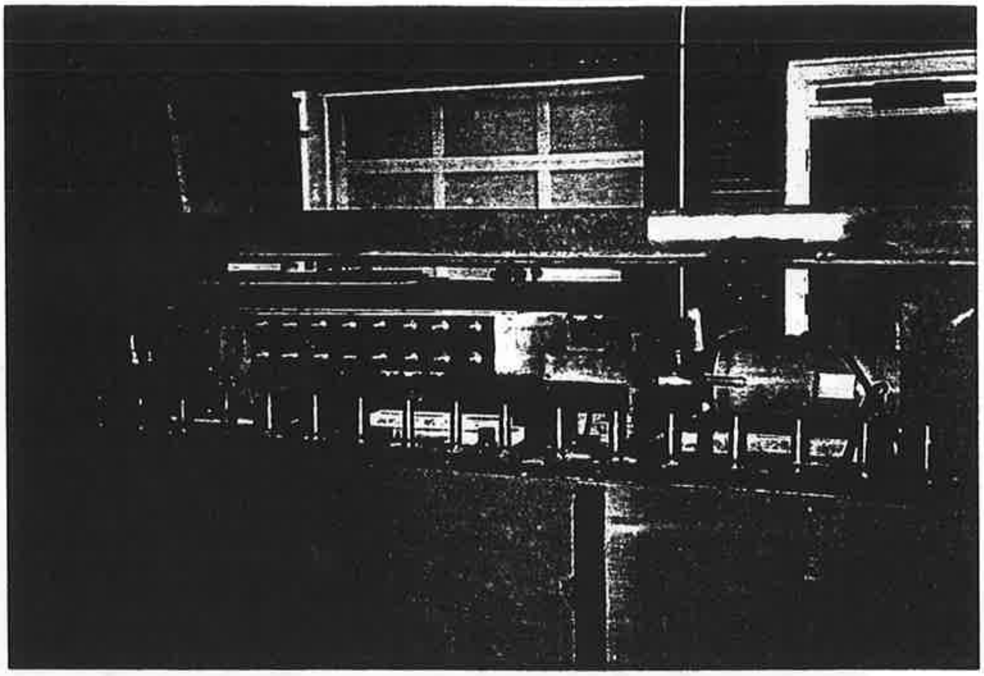


Fig. N.10 Ultimate Strength Test for Series V

recorded slip values were averaged into one value of slip. The horizontal slip was only recorded for the push-off specimens tested for ultimate strength.

Observations During Testing

Fatigue Tests: The general behavior of the specimens under fatigue load showed a crack between the steel and concrete interface at the beginning of the test. This crack indicated that virtually no adhesion occurred between the concrete and the steel plate. The fatigue push-off specimens of Series I were tested until the studs sheared or the concrete around the studs crushed and one inch of horizontal displacement occurred. Series II push-off specimens were fabricated with improved weld quality. No failure of any kind was reported at any stage of the fatigue test in any of these specimens. The tests were stopped after 2,000,000 cycles with stress range S_r of 18.33 ksi (122.8 MPa), or f_{min} = 10 ksi (67.0 MPa), and f_{max} = 28.33 ksi (190 Mpa).

Ultimate Test: The general failure mode for the 1-$^1/_4$ in. (31.8 mm) studs subjected to ultimate force was horizontal shear through the stud at the base of the weld as shown in Fig. N.11. Fig. N.12 shows no indication of yielding of the studs before or at the time of failure, the failure was a sudden brittle failure in all the specimens. A visible crack in the concrete along the stud line was noticed at the top surface of all the specimens during the failure stage as shown in Fig. N.13. The general failure mode for the 7/8 in. (22.2 mm) studs subjected to ultimate force was horizontal shear through the stud over the base of the weld as shown in Fig. N.14. After the specimens had been sheared off all the way, studs yielded a significant amount before they broke in the stud as shown in Fig. N.15. No cracks were noticed at the top surface of the specimen, which utilized 7/8 in. (22.2 mm) diameter studs.

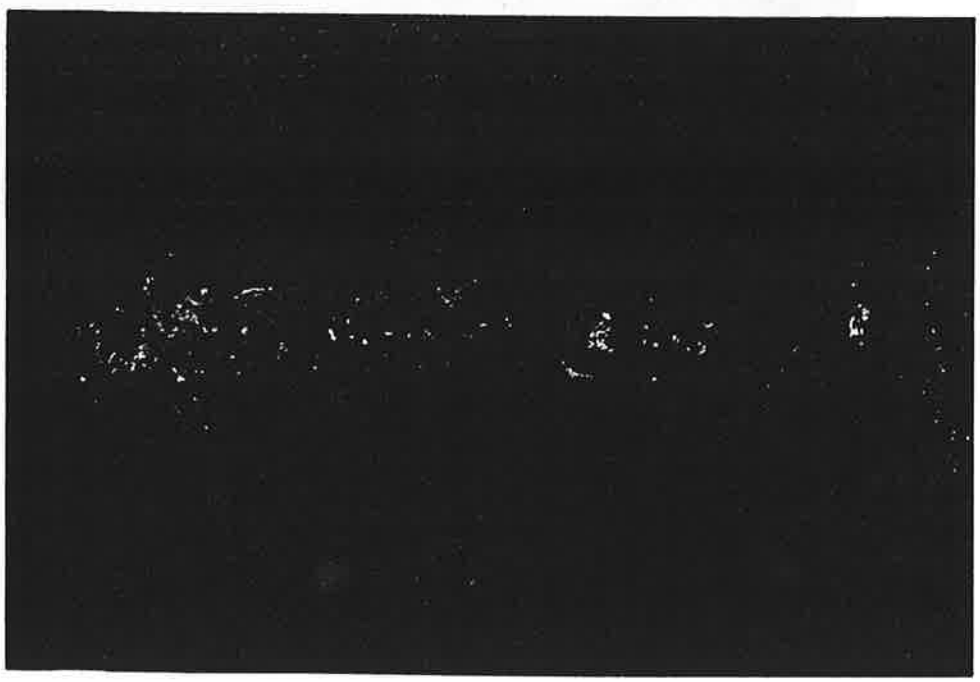


Fig. N.11 Ultimate Failure of 1-$^1/_4$ in. Steel Studs

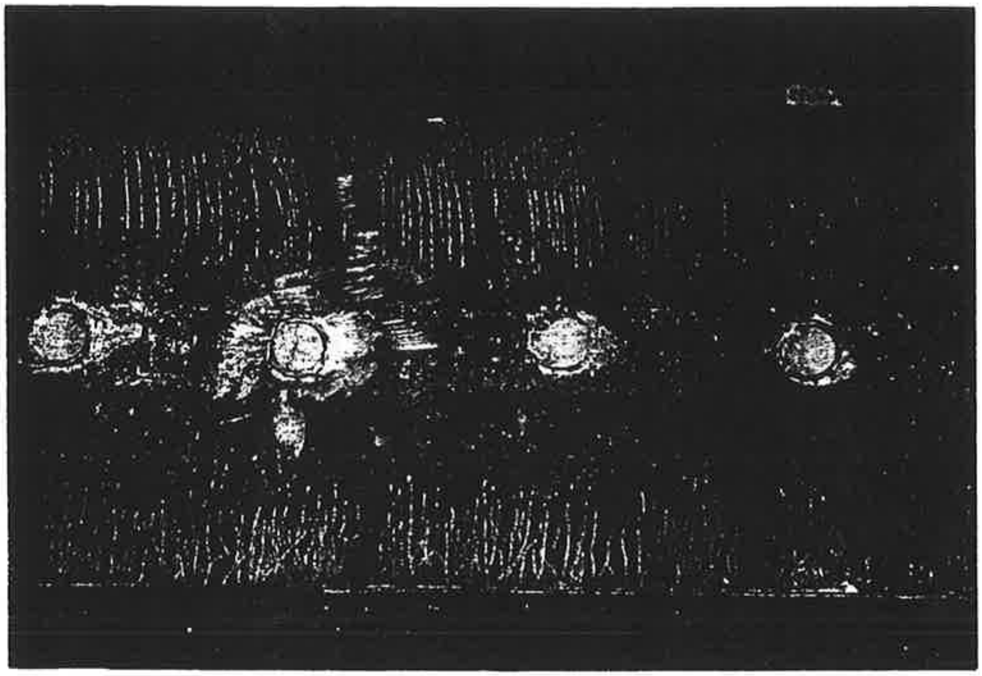


Fig. N.12 Ultimate Failure of 1-$^{1}/_{4}$ in. Steel Studs

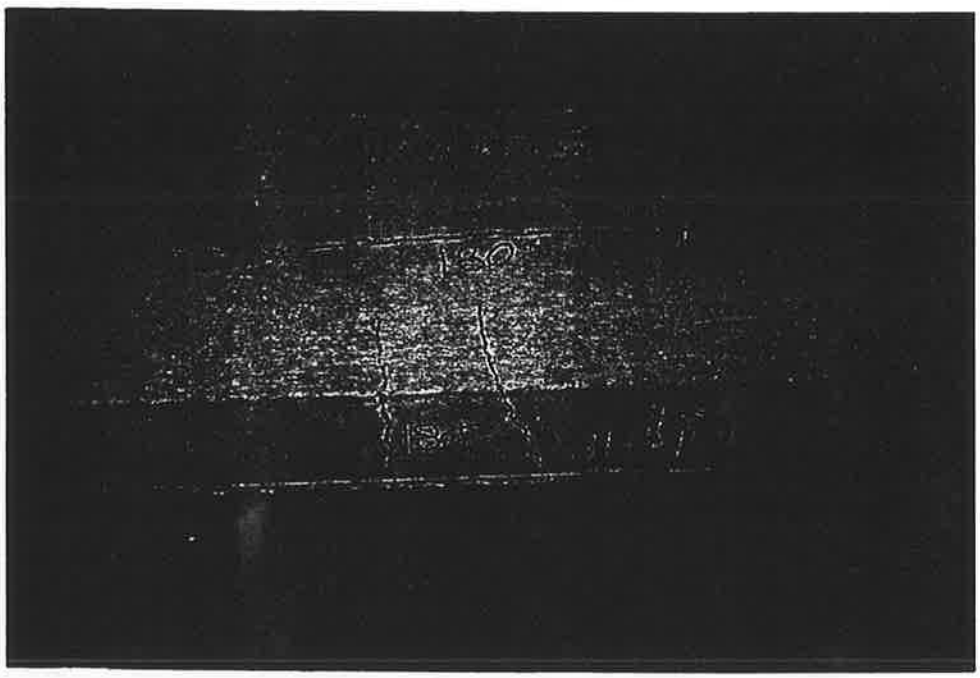


Fig. N.13 Crack in the concrete for 1-$^{1}/_{4}$ in. Steel Studs Specimens Due to Stress Concentration in the Ultimate Strength Test

Fig. N.14 Ultimate Strength Test of 7/8 in. Steel Studs

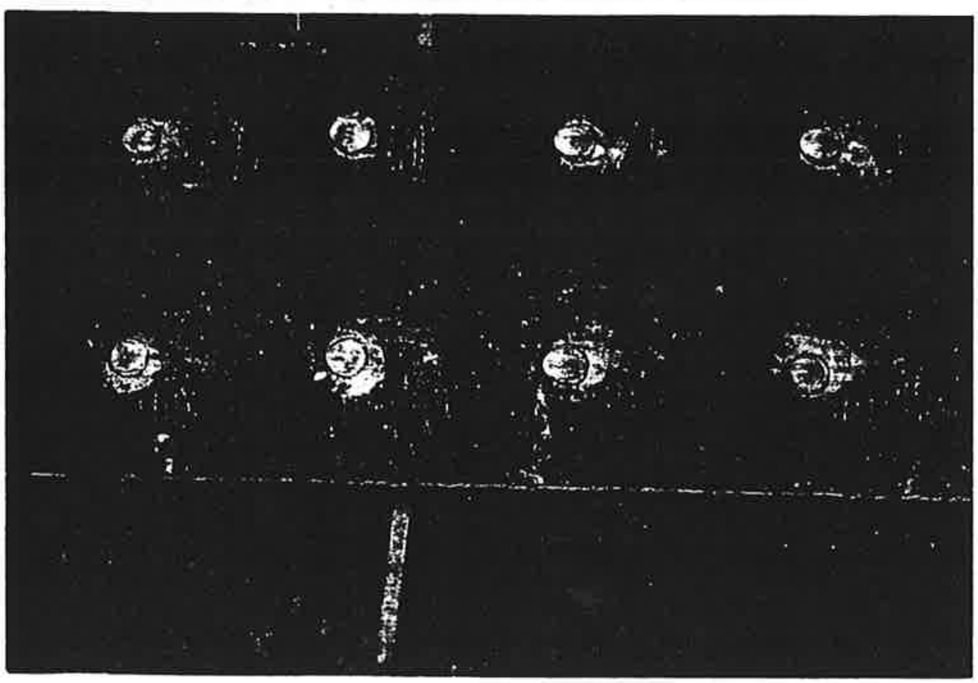


Fig. N.15 Ultimate Strength Test of 7/8 in. Steel Studs

Analysis of Test Results

Fatigue Test: Fatigue test results of the 1-$^1/_4$ in. (31.8 mm) diameter steel stud push-off specimens of Series I and II are given in Table N.3. Included in these results are the applied stress range, the number of cycles to failure, and the observed failure mode. In Series I, the number of cycles to failure ranged from 13,483 to about 6,000,000 cycles. These failure modes were observed at different ranges of applied stress. Studs subjected to high stress ranges experienced a failure mode of apparent shear through the stud. Medium stress ranges resulted in a shear failure of the stud initiated by fatigue of the weld. The weld initiated the failure which continued through the stud. For low ranges of stress, the failure mechanism was the concrete crushing around the stud. It appeared that the number of cycles to failure has an effect on the failure mode.

Table N.3 Fatigue Test Results of 1-$^1/_4$ in. Diameter Shear Studs

Specimen	Minimum Stress (ksi)	Stress Range (S) (ksi)	Cycles to Failure (N)	Observed Failure Mode
Series I TT1	10	15.00	3,961,625	concrete crushing
TT2	10	16.25	6,000,000	concrete crushing
TT3	10	17.50	1,721,370	weld and stud failure
TT4	10	18.75	3,054,277	concrete crushing
TT5	10	20.00	1,903,122	weld and stud failure
TT6	10	21.25	24,758	stud shear
TT7	10	22.50	13,483	stud shear
TT8	10	25.00	24,861	stud shear
Series II all specimens	10	18.33	2,000,000	no failure

Series I test results are compared with the results obtained by Slutter and Fisher (1966) for $^3/_4$ in. (19.1 mm) and $^7/_8$ in. (22.2 mm) diameter studs in Fig. N.16.

All specimens had a minimum stress of 10 ksi. (69.0 MPa). It appeared that the 1-$^{1}/_{4}$ in. (31.8 mm) diameter studs are superior to the smaller diameter studs for fatigue. The type of stud material used by Slutter and Fisher was not available in the literature review. Slutter et al. reported no significant difference between the fatigue strength of $^{3}/_{4}$ in. (19.1 mm) and $^{7}/_{8}$ in. (22.2 mm) diameter shear studs. However, a difference does

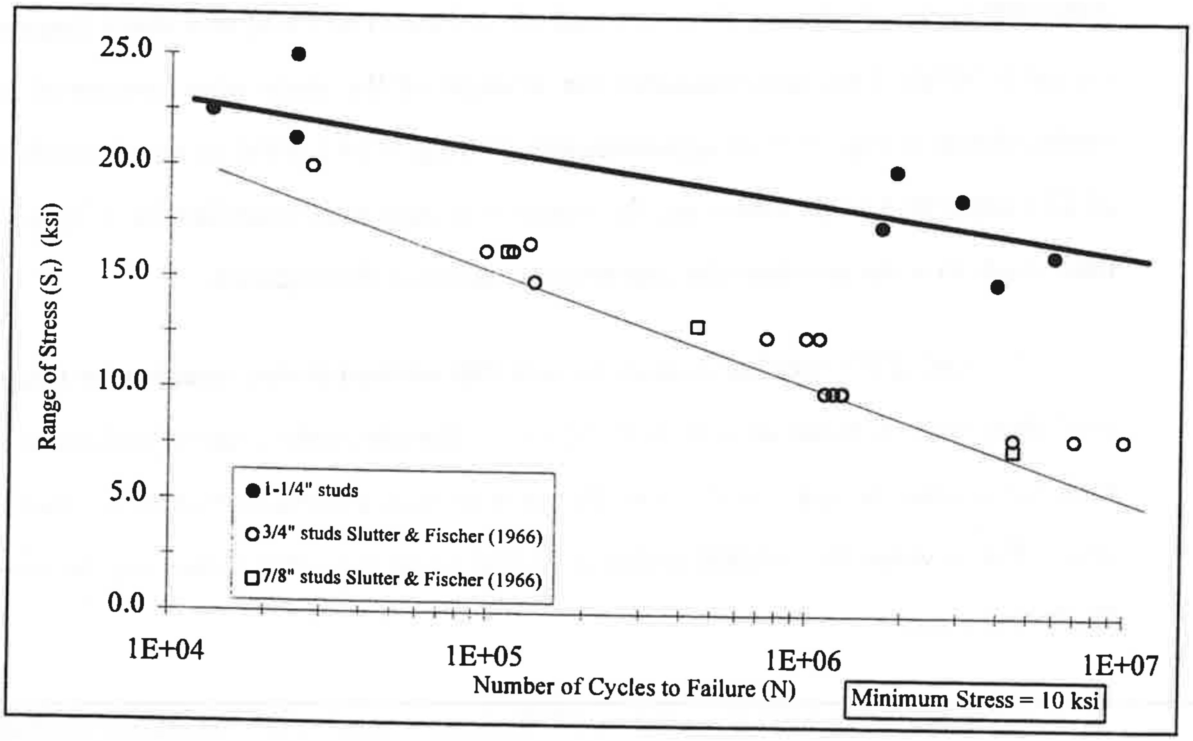


Fig. N.16 Stud Fatigue Results Compared to Slutter and Fisher's

appear between the $^{3}/_{4}$, $^{7}/_{8}$, and 1-$^{1}/_{4}$ in. (31.8 mm) shear studs. Slutter et al. recommended a design formula (N.1) for the allowable range of horizontal shear force per stud as:

$$Z_r = \alpha\, d_s^2 \qquad \text{(N.1)}$$

$\alpha =$ 13,800 for 100,000 cycles

10,600 for 500,000 cycles

7,850 for 2,000,000 cycles

They recommend that this equation be applied to studs smaller than $^{7}/_{8}$ in. (22.2 mm) diameter. If this equation is applied to 1-$^{1}/_{4}$ in. (31.8 mm) diameter studs at 2,000,000 cycles, it yields a force per stud of 12.3 kips (54.7 kN) or a stress range of 10 ksi (67.0 MPa). This underestimates the strength of the studs when compared to the results shown in Fig. N.16 of approximately 18.0 ksi (124.1 MPa) or an allowable force of 22.1 kips (98.3 kN). Therefore, the values of α need to be modified for 1 $^{1}/_{4}$ in. (31.8 mm) studs in order to reduce the conservative nature of this equation.

The AASHTO Standard Specifications (15th edition) design equation for fatigue in steel shear studs is found in section 10.38.5.1.1. The allowable range of horizontal shear force is the same as equation (N.1) by Slutter et al. with a slight difference for the values of α. The α value for 100,000 cycles is 13,000 while the other values are the same as Slutter et al. results.

The AASHTO LRFD (1st edition, 1995) design equation for the allowable range of horizontal shear force is found in section 6.10.7.4.2 as (N.2):

$$Z_r = \alpha\, d_s^2 \geq 5.5 d_s^2 \quad \text{(kips)} \quad \text{(N.2)}$$

$$\text{with } \alpha = 34.5 - 4.28 \text{ Log N}$$

The corresponding values of α are equal to:

$\alpha =$ 13.1 for 100,000 cycles

10.1 for 500,000 cycles

7.5 for 2,000,000 cycles

These α values correspond to a shear force in kips and are comparable to the AASHTO Standard Specifications (15th edition). Both AASHTO equations are conservative for 1-$^1/_4$ in. (31.8 mm) diameter studs. Modified values for α can be established from Fig. N.16. These values are:

$\alpha =$	16,900 for	100,000 cycles
	15,350 for	500,000 cycles
	14,150 for	2,000,000 cycles

These values would result in a more accurate allowable stress which would reduce the number of 1-$^1/_4$ in. (31.8 mm) studs on the bridge girder. This design equation would favor bridge deck removal and replacement if fewer studs were used.

Ultimate Tests: The results of the push-off tests are used to compare the ultimate strength of the 7/8 in. (22.2 mm) studs to an equivalent number of 1-$^1/_4$ in. (31.8 mm) studs. Also, the test results are compared with the current AASHTO Standard and AASHTO LRFD design equations as applied to both the 7/8 in. (22.2 mm) and 1-$^1/_4$ in. (31.8 mm) diameter steel studs. The results for all the specimens are shown in Table N.4. The concrete strength for each specimen at the test day is given. Also included are strength values at 0.02 (0.52 mm) slip and 0.10 in. (2.54 mm) slip. These values of slip were chosen to compare the shear strength to limits established by Viest (1956) for such cases when the strength of the composite connection is limited by slip. Slutter and Driscall (1965) indicated that the magnitude of slip does not reduce the ultimate strength of the shear stud if the equilibrium condition is satisfied and the magnitude of slip is not greater than the lowest values of slip at which an individual connector might fail. These results are given as a force since the design equations for ultimate strength produce values

of force per stud. The results for the ultimate strength of all the specimens are also plotted in Figs. N.17, N.18, N.19 and N.20, which show the horizontal shear force of the specimen vs. the relative horizontal slip.

The AASHTO Standard Specification (15th edition) gives equation (N.3) for the nominal strength of welded shear studs as:

$$S_u = 0.4 d_s^2 \sqrt{f_c' E_c} \qquad \text{(N.3)}$$

The AASHTO LRFD Specification (1995) gives equation (N.4) for the nominal strength of welded shear studs as:

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc}\, f_u\,. \qquad \text{(N.4)}$$

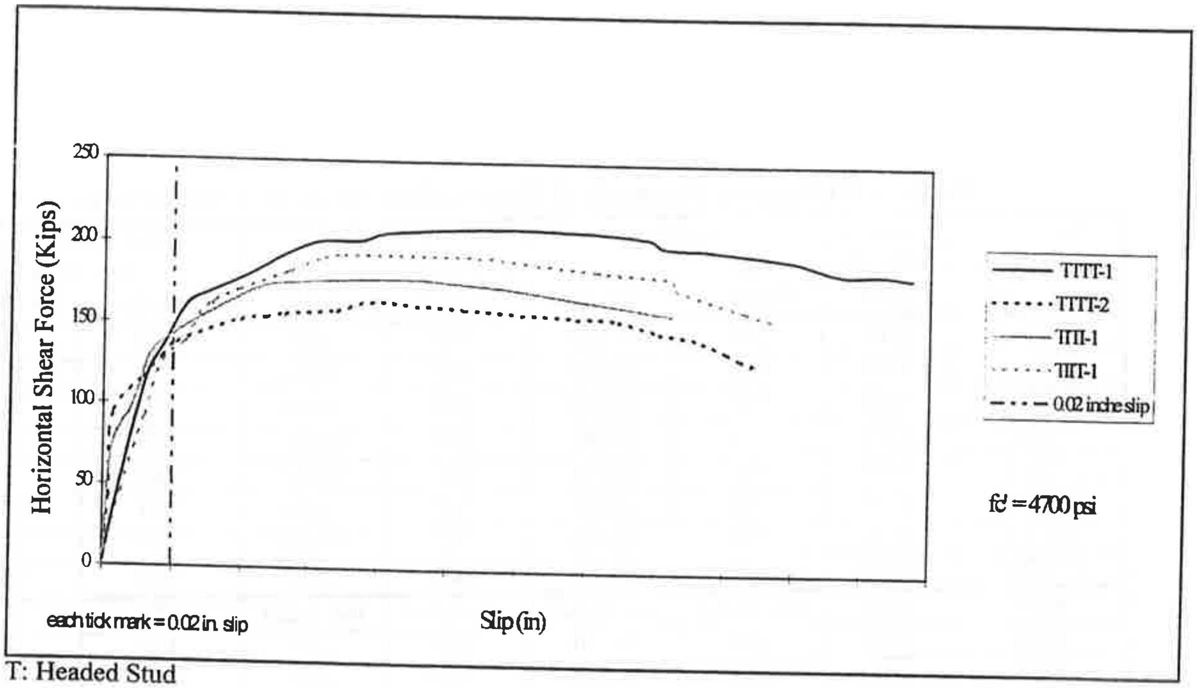
Comparing the test results of both the 7/8 in. (22.2 mm) and $1\text{-}^1/_4$ in. (31.8 mm) diameter steel studs in the first four groups shows a favorable comparison between $1\text{-}^1/_4$ in. (31.8 mm) and the 7/8 in. (22.2 mm) diameter steel stud capacity. However, these values did not compare favorably with both the AASHTO Standard and the AASHTO LRFD. Slutter and Driscall (1965) have indicated that eccentricity of loading and insufficient transverse reinforcement can cause the test results to be far below the expected average capacities. Eccentricity is a valid issue in this research, regarding the way the load was applied to the specimens. To study the effect of the eccentricity on the capacity of the stud, series V was constructed and tested. The test results of this series showed excellent results concerning the ultimate capacity. These results compare favorably with AASHTO LRFD Specifications as shown in Table N.4.

Table N.4 Ultimate Strength of Steel Girder-to-Deck Connections

Specimen Designation	Concrete strength (psi)	Ultimate force (kips)	Ultimate slip (inches)	Force at 0.02 inches (kips)	Force at 0.1 inches (kips)	Stud strength by AASHTO Standard	Stud strength by AASHTO LRFD
TTTT-1	4700	217	0.120	142	205	339	314
TTTT-2	4700	164	0.080	135	broke	339	314
FTTTT-3	4700	242	0.060	200	broke	339	314
FTTTT-4	4700	213	0.080	189	broke	339	314
TITI-1	4700	195	0.074	133	broke	339	314
FTITI-2	4700	183	0.065	162	broke	339	314
TIIT-1	4700	178	0.090	142	broke	339	314
FTIIT-2	4700	190	0.064	168	broke	339	314
tttt-1	4700	test failed	test failed	test failed	test failed	332	308
tttt-2	4700	244	0.230	154	225	332	308
tttt-3	4700	231	0.220	87	207	332	308
tttt-4	4700	210	0.300	120	196	332	308
RTTTT-1	5600	208	0.080	170	broke	386	314
RTTTT-2	5600	185	0.078	120	broke	386	314
FRTTTT-3	5600	216	0.043	188	broke	386	314
FRTTTT-4	5600	209	0.053	188	broke	386	314
ATITI-3	6800	330	0.076	202	broke	447	314
ATITI-4	6800	298	0.062	191	broke	447	314
STITI-5	6800	267	0.067	147	broke	447	314
STITI-6	6800	test failed	test failed	test failed	test failed	447	314

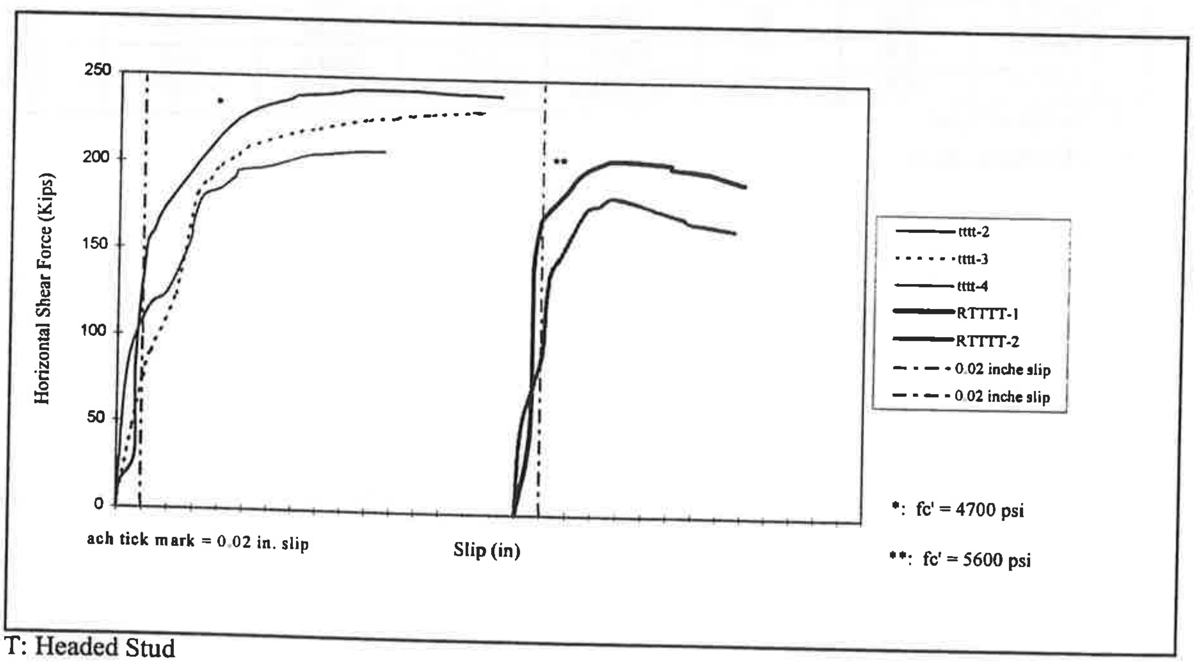
T: Headed Stud

I: Headless Stud



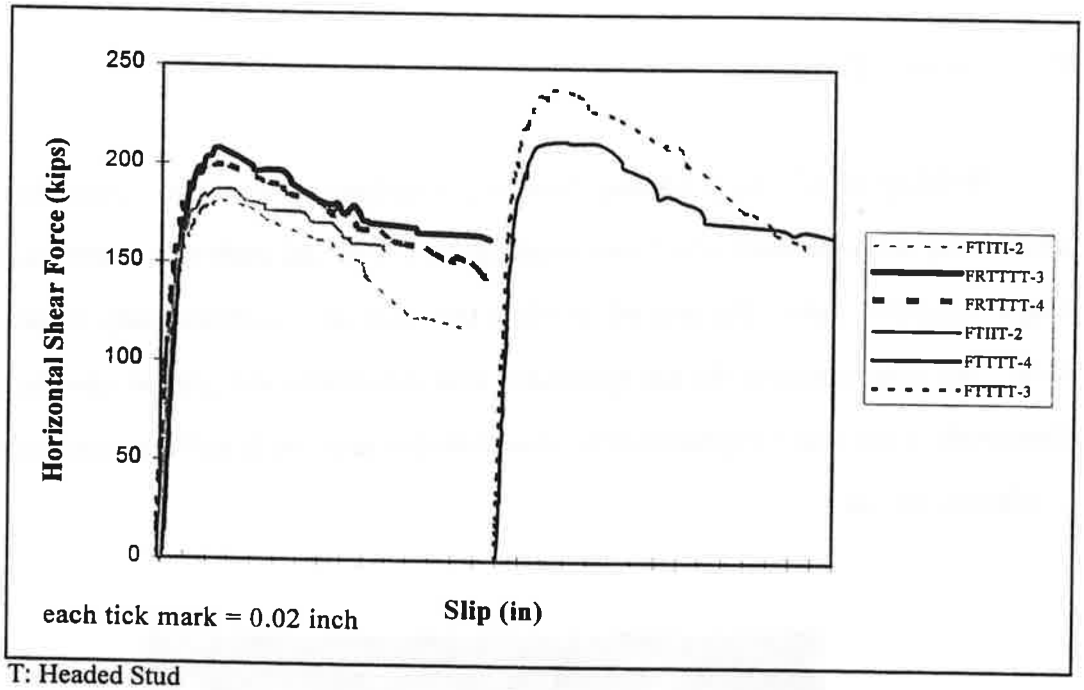
T: Headed Stud
I: Headless Stud

Fig. N.17 Ultimate Tests of 1-1/4 in. Steel Studs



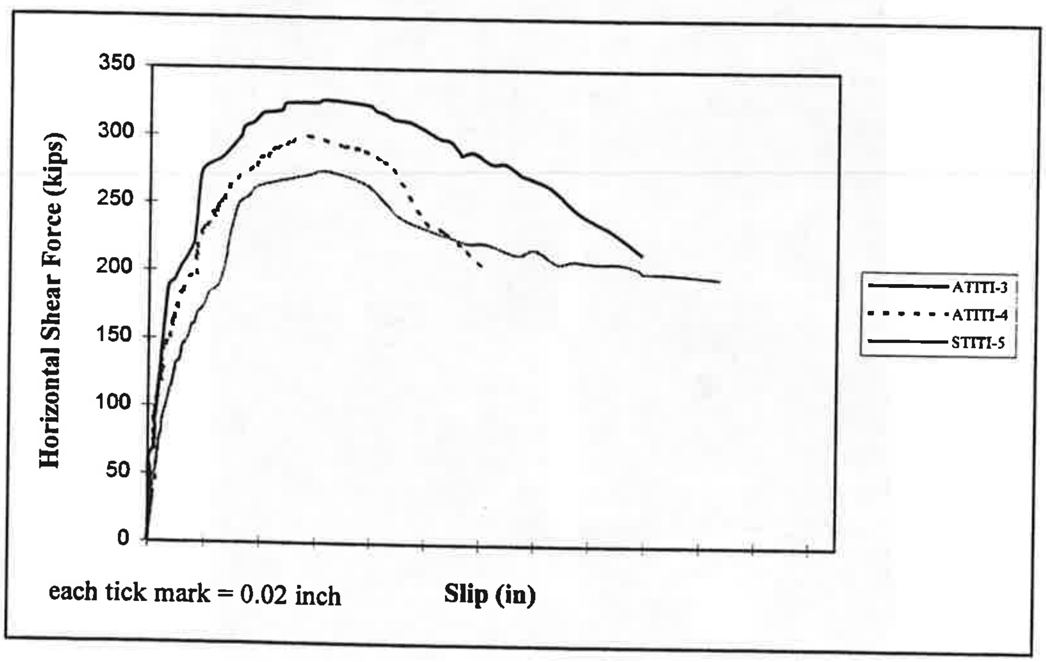
T: Headed Stud
I: Headless Stud

Fig. N.18 Ultimate Tests of 7/8 in. and 1-1/4 in. Steel Studs



T: Headed Stud
I: Headless Stud

Fig. N.19 Ultimate Tests of 1-$^1/_4$ in. Steel Studs After Fatigue was Applied



T: Headed Stud
I: Headless Stud

Fig. N.20 Ultimate Tests of 1-$^1/_4$ in. Steel Studs for Group V

Full Scale Test

Fabrication of Test Specimen

Welding of 1-$^{1}/_{4}$ in. (31.8mm) shear stud was done by Tri-Sales Associates. A total of 82 studs were welded to the beam, welding quality for all studs was excellent without a single defective weld. The rate of welding the studs was approximately 30 seconds per stud. The reinforcement for the specimens was assembled and placed into the plywood formwork. Concrete was placed and consolidated around studs and reinforcement using a small stud vibrator.

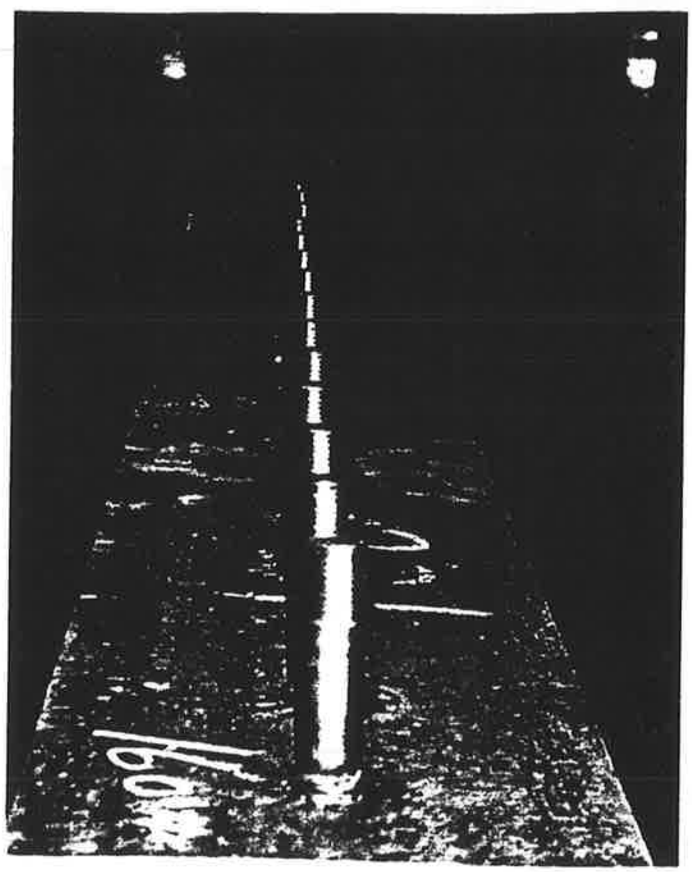


Fig. N.21 Stage I of Construction

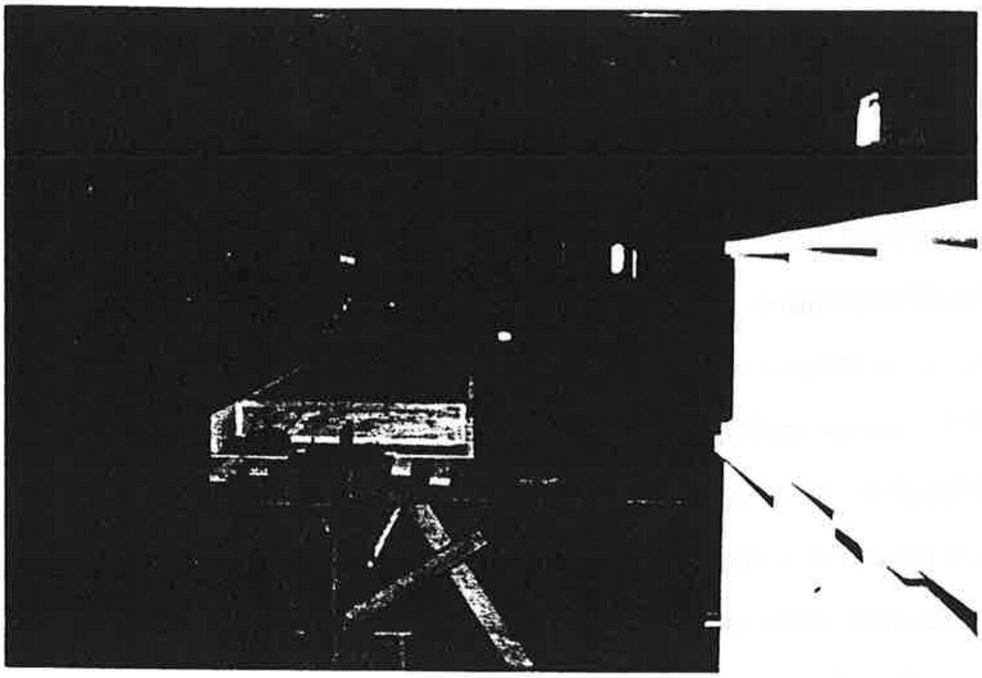


Fig. N.22 Stage II of Construction

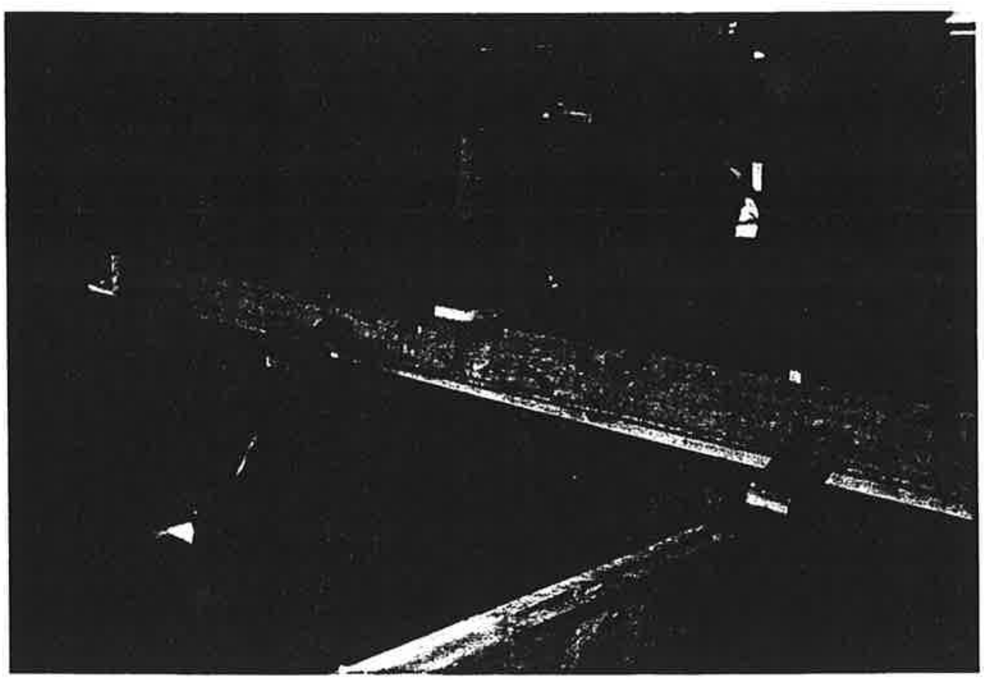


Fig. N.23 Stage III of Construction

Experimental Program and Test Procedures

In order to apply the critical shear force amplitude for fatigue testing, a MTS actuator was positioned at 4 ft (1.22 m) from the end support with the set-up shown in Fig. N.24 and N.26. The range of fatigue load was 10 kips (44.5 kN) minimum to 90 kips (400 kN) maximum. The load range was applied at a frequency of 2 cycles per second. Prior to the fatigue test, the specimen was loaded up to 90 kips (422.5 kN) monotonically with the set-up and location shown in Figs. N.25 and N.27. Slippage between the girder and the deck and stresses distributions were measured at 4 ft (1.22 m) from the support and at the center of the span. The intention for this reading was to compare slippage and stresses distribution measurement before and after fatigue loading. After 1.76 million cycles of loading was reached, a monotonic load of 90 kips was applied to measure slippage and stress distribution again. There was no difference between these slip figures and the slip obtained at the beginning of the test. This indicated no concrete crushing around the shear studs or stud failure. The fatigue test then continued to 4.8 million cycles, after which the monotonic load was applied again to measure slippage and stress distributions at the same locations. Strain gages were provided across the full depth of the girder deck assembly at end and mid span of the girder as shown in Fig. N.28. Linear Variable Displacement Transducers (LVDT) were located at 4 ft from the end support and at midspan.

The MTS actuator then was positioned at midspan and a load of 100 kips was applied to produce a moment of 1000 ft-kip (impact live loads); slippage and stress distributions at 4 ft from the end support and at midspan were measured again.

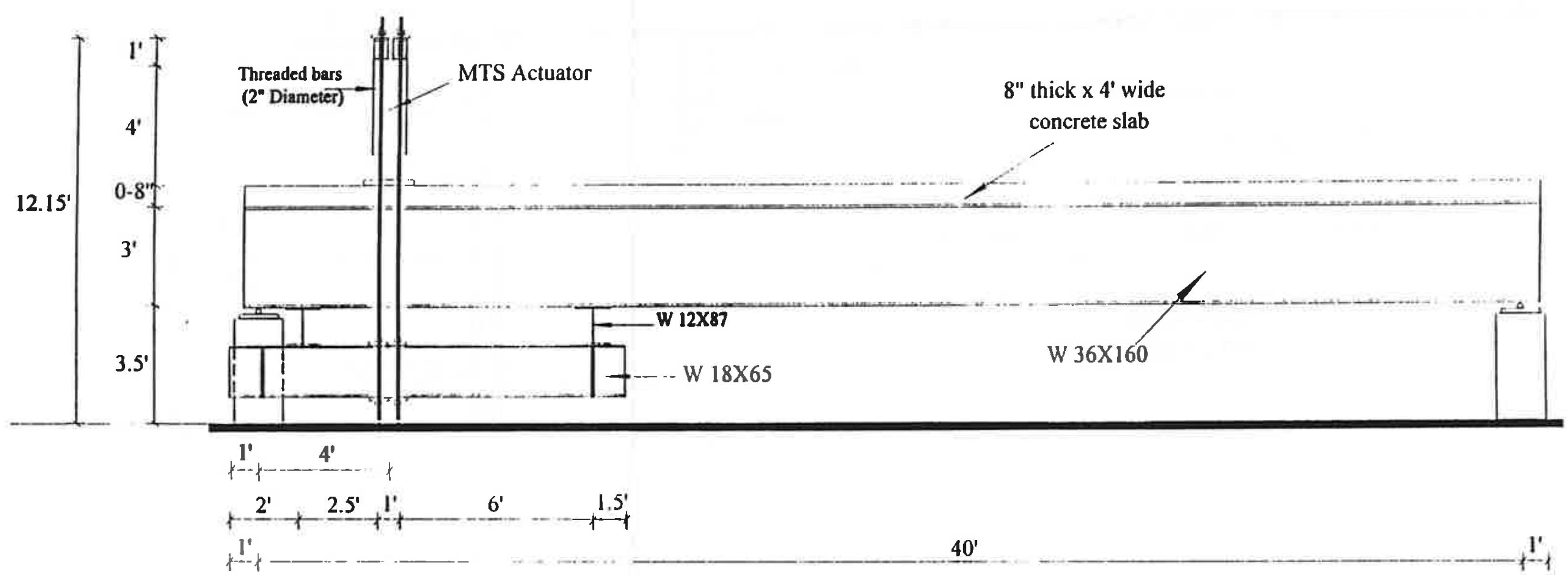


Fig. N.24 Test Setup for Fatigue Loading

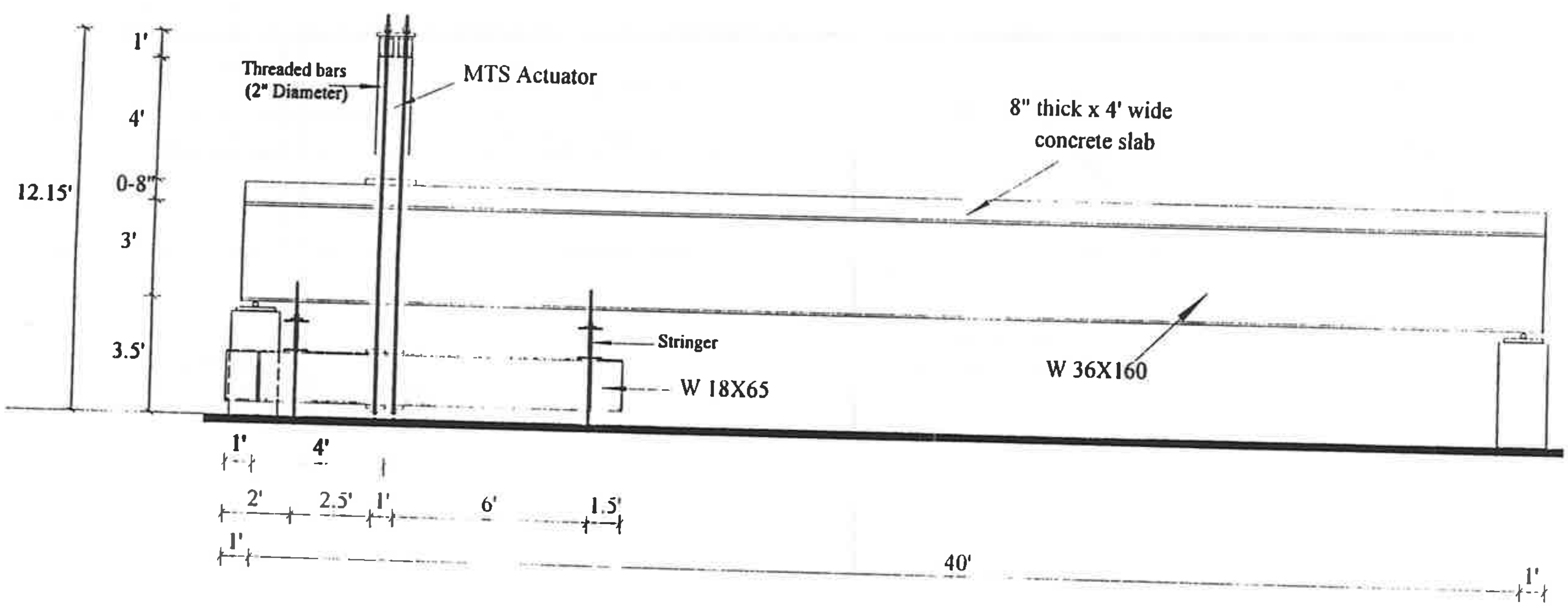


Fig. N.25 **Test Setup forMonotonical Loading**

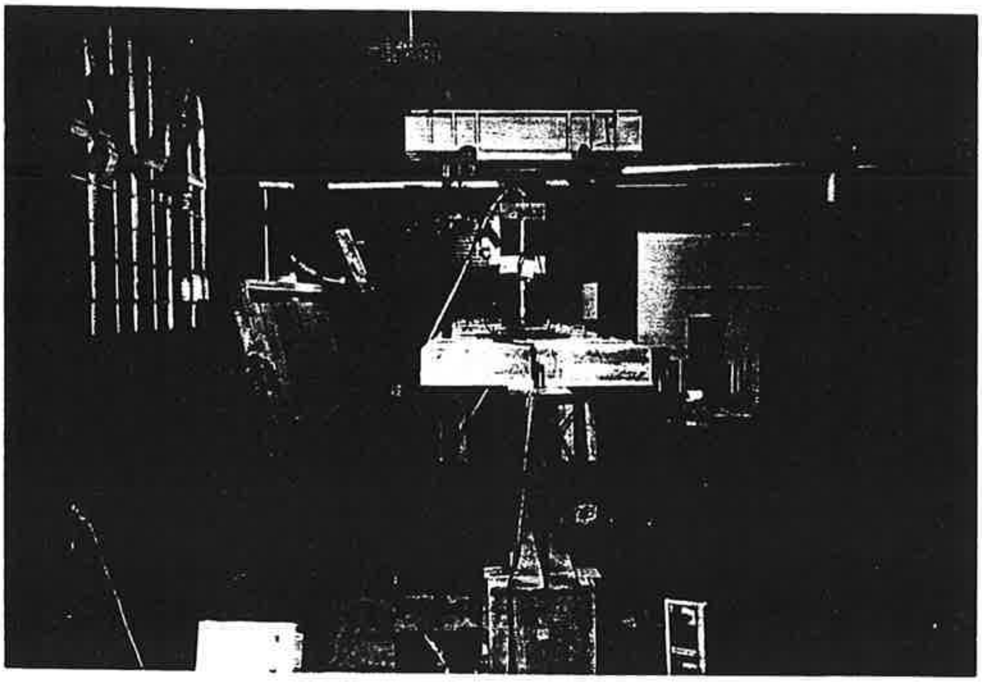


Fig. N.26 Set Up for Fatigue Loading

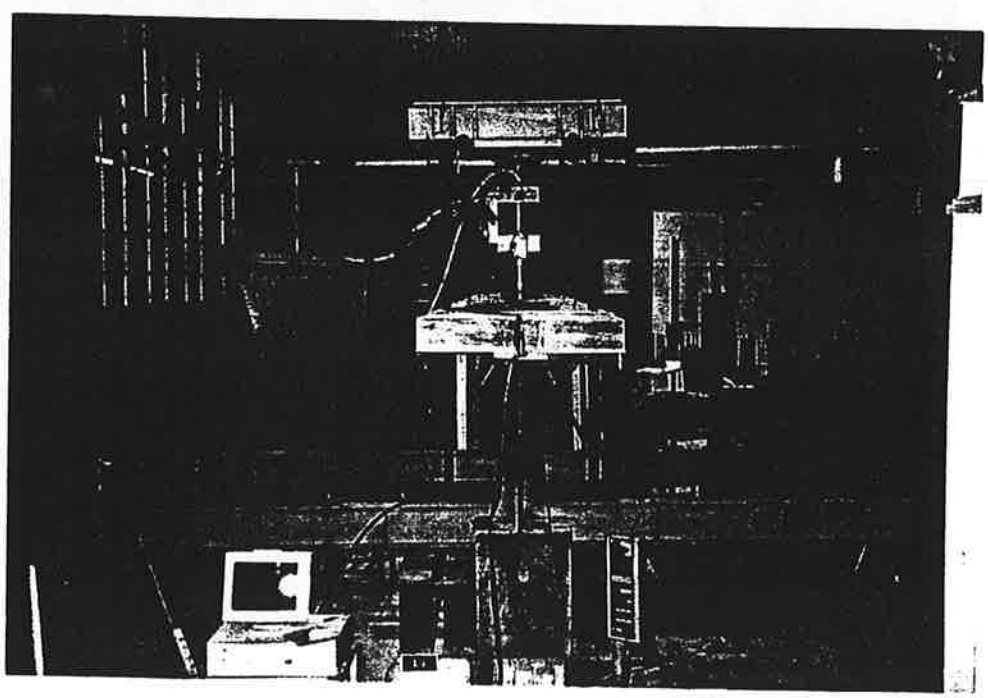


Fig. N.27 Set Up for Monotonic Loading

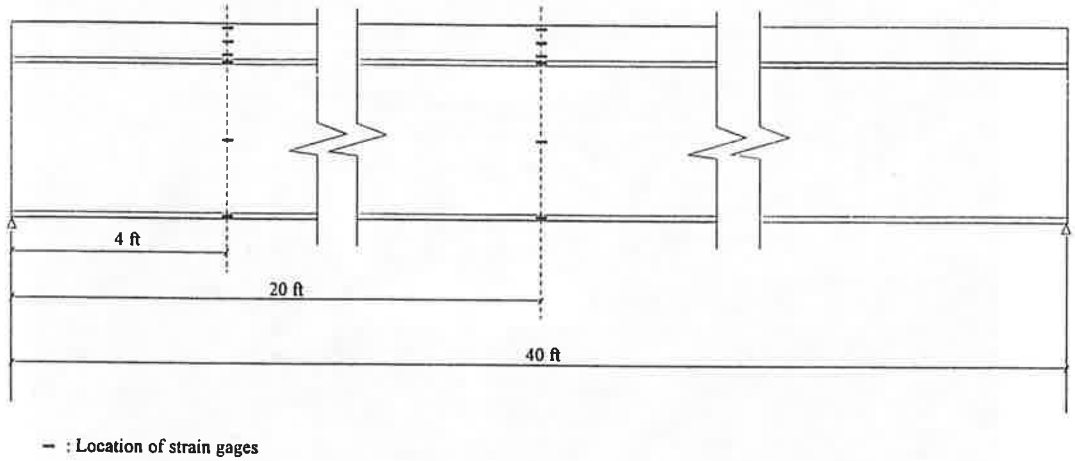


Fig. N.28 Strain Gages Locations at Full Scale Test for Steel Girder

Deck Removal

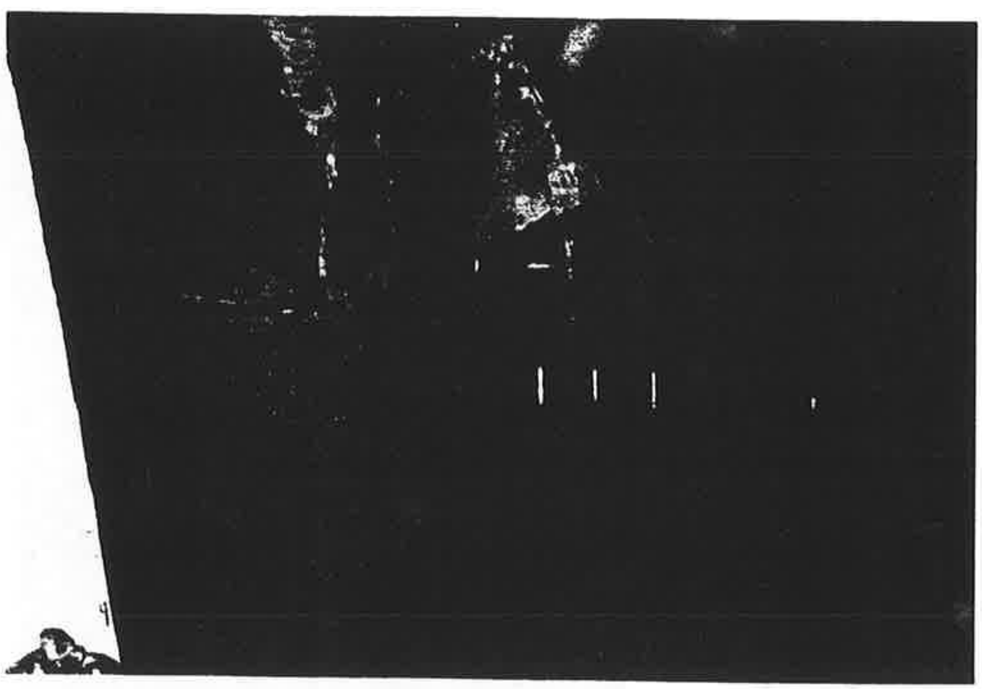


Fig. N.29 Cutting along the Longitudinal Direction

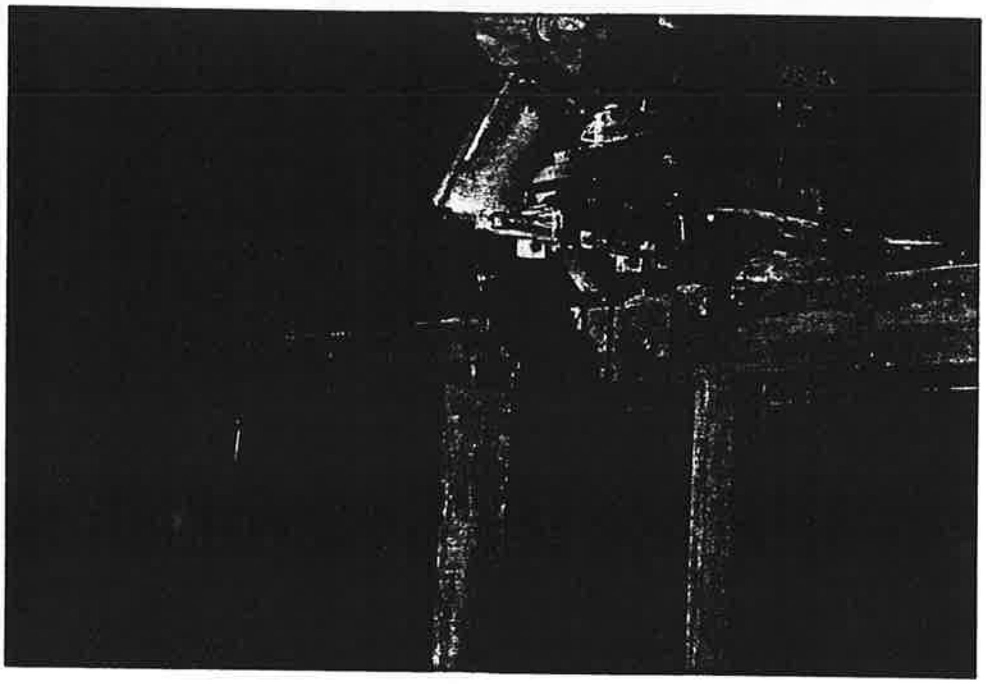


Fig. N.30 Cutting along the Trasverse Direction

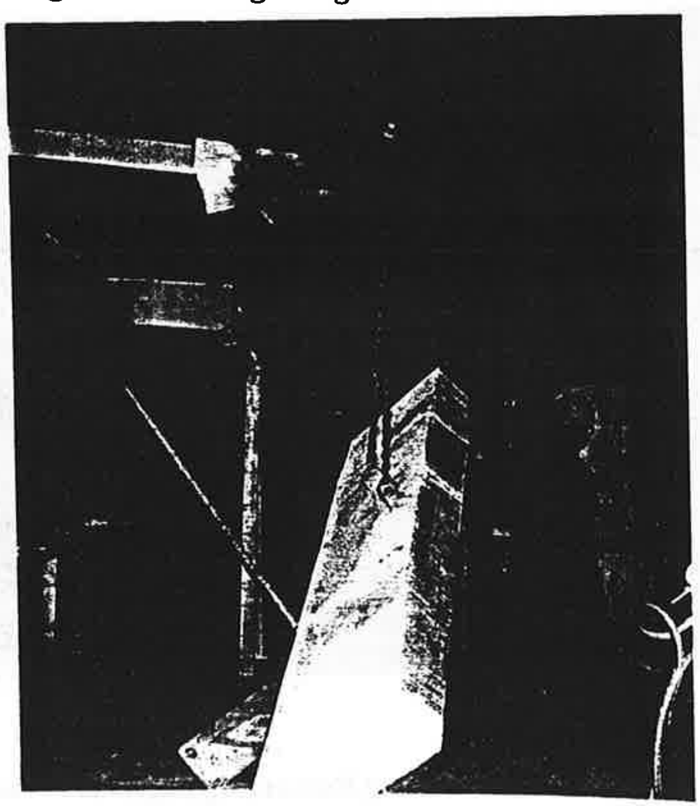


Fig. N.31 Removing Section 1a

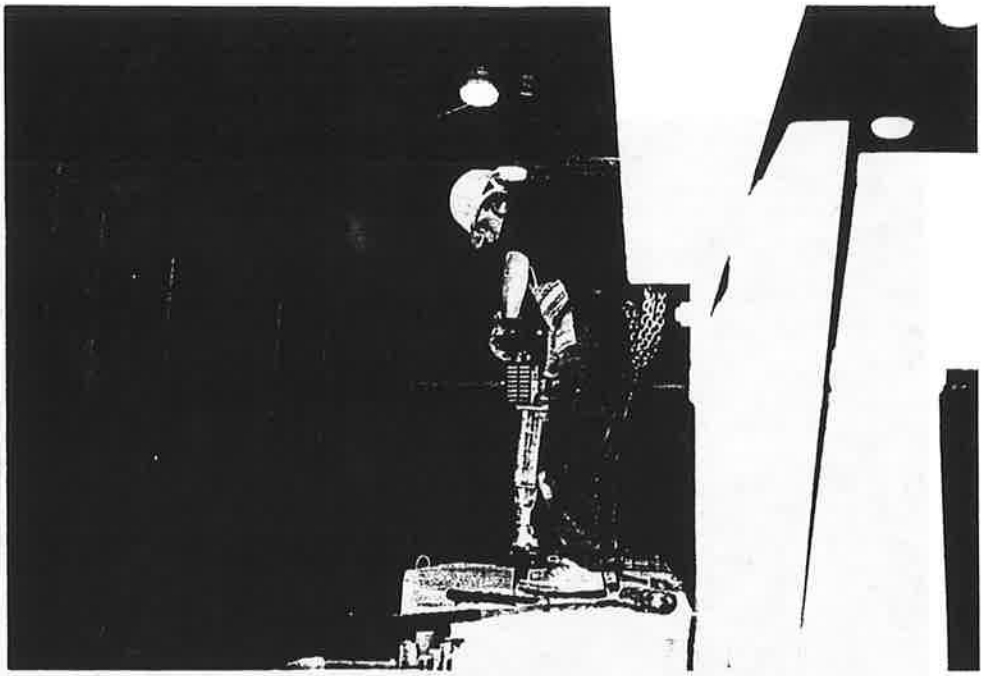


Fig. N.32 Jack Hammering Around the Studs

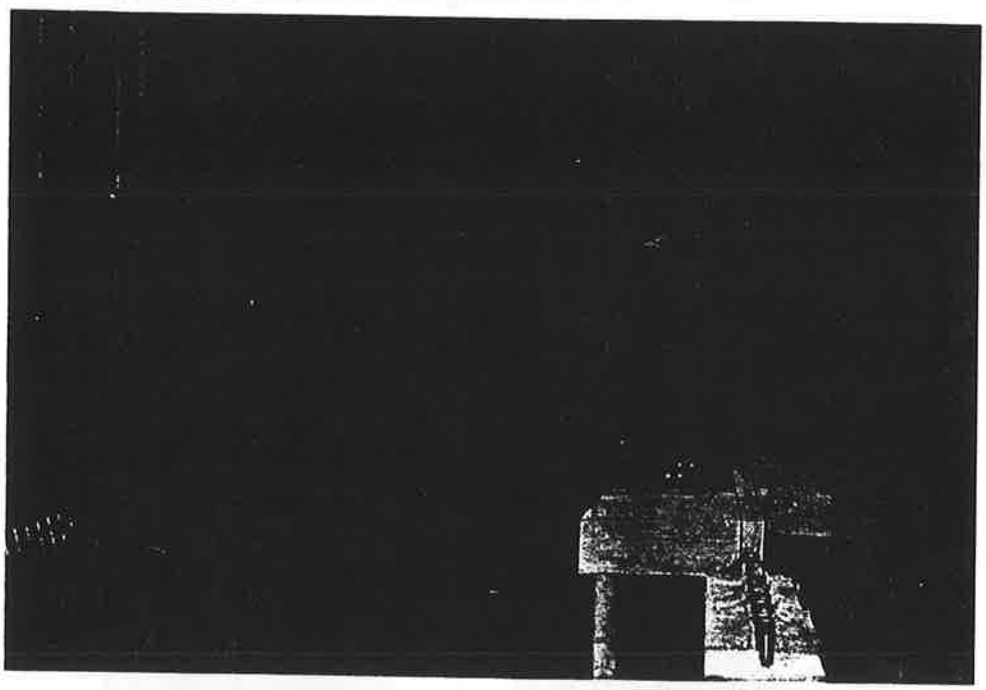


Fig. N.33 Removing Remaining Sections of the Slab

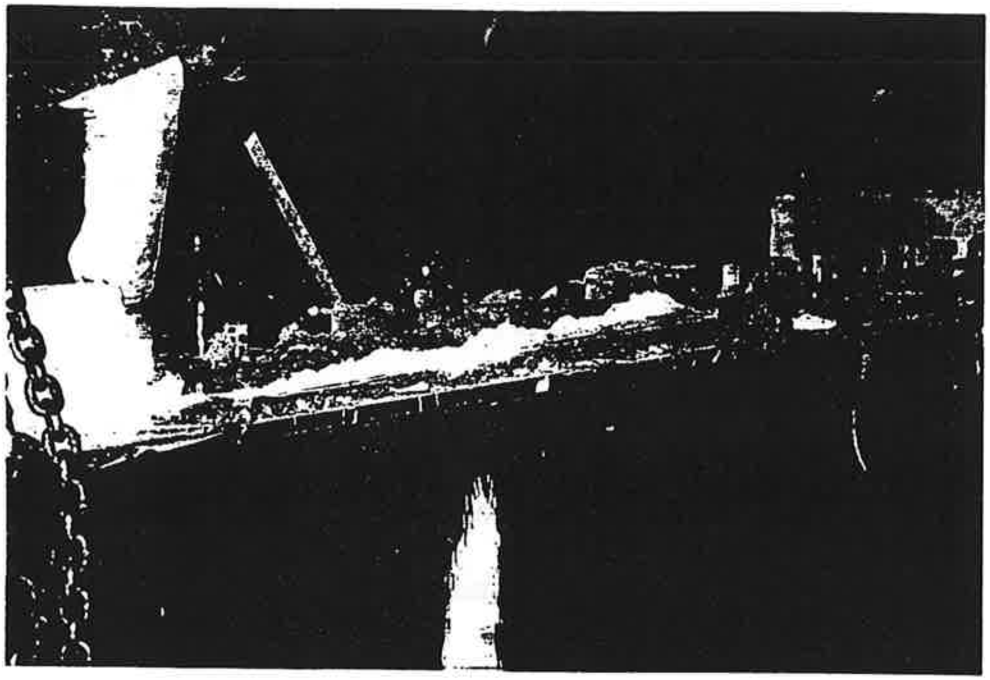


Fig. N.34 Breaking Concrete from Around the Studs

Fig. N.35 Studs after Deck Removal

APPENDIX O

Steel Girder-to-Deck Connectors

Steel Full Scale Design Calculations

Single Span W36 X160

1. DESIGN CONDITIONS

This bridge consists of a 40 ft single span structure. The superstructure consists of four W36 X 160 (f_y = 36 ksi) girders, spaced at 12′-0″ on center. The total width of the bridge is 44′-0″ as shown in Fig. O-1. Girders are designed using AISI Beam Design Program, the girders are designed to act compositely with the 8″ cast-in-place concrete deck (f'_c = 4000 psi) to resist all superimposed dead loads, live load and impact. New Jersey type concrete barrier is used. This bridge will also be designed to resist a 25 psf superimposed dead load; and an AASHTO HS 25-44 truck load.

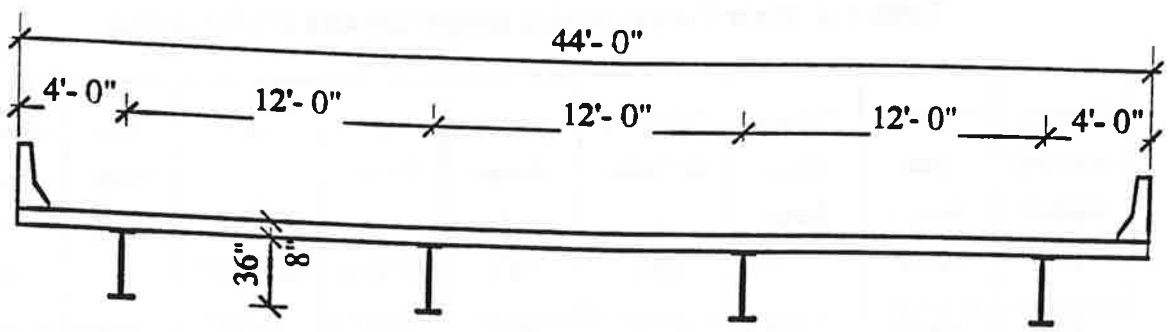


Fig. O-1 Bridge Cross-section

2. MATERIALS

Cast in place slab:	Thickness	t_s	=	8	inch
	28-Days concrete strength	f'_{cs}	=	4000	psi
Steel girder:	Section W36X160				
	Yield strength	f_y	=	36000	psi
Reinforcing bars:	Yield strength	f_{sy}	=	60,000	psi

3. Design for Horizontal Shear:

3.1 Fatigue Design: Art. (10.38.5)

AISI Beam Design Program was used for both analysis and design. One row of 1-$^1/_4$ in. shear stud was used in the design. AASHTO's values for Alfa paremeter for 2,000,000 cycles were used in the program. Fatigue test results at the University of Nebraska, showed much higher alfa (α) values for 1-$^1/_4$ in. studs. A value of α=14,150 for 2,000,000 cycles was derived from these tests. Table L.1 shows shear values, section properties and studs spacing according to both AASHTO and test results values for Alfa.

Table L.1 shear values, section properties and studs spacing

Distance from Left Support (ft)	V_r-Truck Shear Range (kips)	V_r-Lane Shear Range (kips)	Moment of Inertia I (in^4)	Statical Moment Q (in^3)	Sr Truck (kip/in)	Z_r (kip/stud)	Alfa Truck	Maximum Pitch of Studs (in)
0-4	106.13	75.51	24,826.8	677.9	2.90	12.26	7,850	4.23
4-8	90.77	64.70	24,826.8	677.9	2.48	12.26	7,850	4.95
8-12	83.68	61.53	24,826.8	677.9	2.28	12.26	7,850	5.37
12-16	76.59	59.26	24,826.8	677.9	2.09	12.26	7,850	5.87
16-20	73.75	57.90	24,826.8	677.9	2.01	12.26	7,850	6.10
Using Alfa(α) equal to 14,150								
0-4	106.13	75.51	24,826.8	677.9	2.90	22.10	14,150	7.60
4-8	90.77	64.70	24,826.8	677.9	2.48	22.10	14,150	8.90
8-12	83.68	61.53	24,826.8	677.9	2.28	22.10	14,150	9.70
12-16	76.59	59.26	24,826.8	677.9	2.09	22.10	14,150	10.60
16-20	73.75	57.90	24,826.8	677.9	2.01	22.10	14,150	11.00

3.2 Ultimate Strength Design: Art. (10.38.5)

The number of shear connectors required between points of maximum positive moment and adjacent end support, shall equal or exceed the number given by the formula:

$$N_1 = \frac{P}{\Phi S_u}$$

where P is the smaller value of the formulas:

$P_1 \le A_s F_y = (47)(36) = 1692$ kip Controls

$P_2 \le 0.85 f_c' b t_s = 0.85(4)(96)(8) = 2611$ kip

$$S_u = 0.4 d^2 \sqrt{f_c' E_c} = 0.4(1.25)^2 \sqrt{(4{,}000)(3{,}605{,}000)}\frac{1}{1000} = 75\,\text{kip}$$

$S_u < A_s F_y = 78$ kip O.k.

$$\Phi S_u = 0.85(75) = 63.8\,\text{kip}$$

$$N_1 = \frac{1692}{63.8} = 27\,\text{studs}$$

$$\text{Spacing} = \frac{L/2}{N_1 - 1} = \frac{20(12)}{27 - 1} = 9.23\,\text{in.} > \text{Spacing required by fatigue}$$

APPENDIX P

Proposed Special Provisions for Removal of Existing Bridge Deck

The following are the proposed special provisions for removal of existing bridge deck prepared by the research team:

Description of Work

This work shall include removal of the existing concrete bridge deck in accordance with the limits and details shown on the plans.

Construction Staging and Maintenance of Traffic

Work shall be staged in accordance with the requirements contained in the special provisions and the construction staging scheme shown on the plans. Lane closures will be allowed only during the time periods designated on the plans or in the special provisions. The contractor shall develop a sequencing plan to accomplish the removal and placement of the deck, or incremental portions thereof, during the allowable traffic closure periods. The contractor shall submit to the engineer, for review and comment, a detailed schedule of activities to be accomplished during the typical closure periods. The schedule shall include time allowances for the setting and removal of temporary barriers, barricades, signing and other traffic control devices.

Signing, barricades, lighting, traffic control devices and traffic control operations used in maintaining traffic shall be in accordance with the applicable provisions of the latest issue of the Manual on Uniform Traffic Control Devices for Streets and Highways as prepared by the National Joint Committee on Uniform Traffic Control Devices.

No existing structure shall be removed or closed to traffic until traffic has been satisfactorily provided for as required by the plans or as directed by the engineer. The contractor shall employ such methods as are required during the removal operation to avoid any interference with railroad or vehicular traffic beneath the structure. These methods may include but shall not be limited to channeling vehicular traffic beneath the bridge such that removal operations will not occur directly above traffic or the use of a containment system installed prior to removal of the concrete bridge deck and sized to adequately prevent debris from falling on the railroad tracks or roadway beneath the structure.

Removal Methods

At least 10 days before beginning bridge deck removal, the contractor shall submit in writing to the engineer details of the removal operation describing the methods and sequence of removal, equipment to be used and proposed schedule of removal. The plan shall describe the methods for the protection and safety of the general public and the public utilities. Demolition work may not proceed until the plan has been reviewed and accepted. When the bridge deck to be removed is over an existing railroad, the demolition plan shall also be submitted to the railroad for comment. Demolition work may not proceed until written acceptance is received from the railroad company. A copy of this written acceptance shall be provided to the Engineer. For structures over navigational waters the project will be under the authority of the United States Coast Guard permiting process, and the work of removal or partial removal of any structures involved therein is subject to inspection and acceptance by the U.S. Coast Guard prior to acceptance by the Engineer.

Flame cutting and saw cutting may be used for removing the existing bridge deck provided the contractor complies with appropriate protection, safety and damage control

requirements. The use of explosives or a headache ball shall not be permitted for bridge deck removal.

Where portions of existing structures or bridge decks are to either permanently remain in service or temporarily remain for maintenance of traffic, the portions to be removed shall be removed in such a manner as to leave the remaining structure undamaged and in proper condition for the use contemplated. In such cases, a saw cut approximately 1" (25 mm) deep shall be made to a true line along the limits of the removal to initiate the removal process. The use of drop hammers, free falling type equipment or other heavy duty demolition equipment shall be excluded within 24" (600 mm) of the saw cut line. Jackhammers up to the nominal 30 lb. class may be used to within 6" (150 mm) of the edges of the areas to be removed. Chipping hammers up to a nominal 15 lb. class shall be used to remove the remaining 6" (150 mm) edge of concrete. In order to prevent damage to sound concrete, pneumatic hammers shall be worked to an angle of 45°-60° to the plane of the concrete surface being removed.

During removal of deck concrete over beams, girders, diaphragms or other structural members which are to remain in place, the contractor shall exercise care so as not to notch, gouge, or damage the top flanges with jackhammers or other tools. Damage to members to remain in service shall be repaired by the contractor at his/her own expense including assessment for additional inspection and engineering costs. Repairs shall be performed as authorized by the engineer and may include grinding, welding and/or member replacement depending on the location and severity of the damage. All material, labor, equipment and traffic control required for the repair or replacement of contractor damaged beams, diagrams and bearings shall be considered subsidiary to other items for which direct payment is made.

After removal of the concrete deck, the top surfaces of old structural steel girders shall be cleaned of concrete, rust, scale or other extraneous material which will be in contact with the new deck. In the case of prestressed concrete girders, the top surface of the beam along the entire span shall be cleaned by sand blasting, water blasting or other approved methods to remove all remaining fractured or loose concrete and to provide a suitable bond between the beam and the concrete deck. Where reinforcing bars are to extend from existing structures into the new construction, concrete shall be carefully removed from around the reinforcing bars and the reinforcing bars shall be cleaned and straightened for lapping with new reinforcement as indicated on the plans. In the event that reinforcing bars to remain is inadvertently cut or damaged such that the required lap splice cannot be made, a mechanical connector capable of developing 125% of the specified yield strength of the reinforcing bars may be substituted in lieu of the lap splice. In some cases, drilling and grouting in new reinforcing bars to replace changed bars may be an appropriate repair alternative.

Removal and construction techniques shall be employed which will minimize falling and flying debris which could strike vehicles or pedestrians. The contractor shall exercise caution when deck removal operations fall within 30' (9000 mm) horizontal to an area open to the public. In these cases, protective shields shall be installed to protect areas accessible to the public and shall be of sufficient size and strength to prevent debris or equipment from endangering the public. The protective shields shall be designed and proportioned as required by the size of equipment, size of debris produced and method of operations employed. Personal injuries, damages to vehicles or other property, public or private, and damage claims which may arise shall be the sole responsibility of the contractor.

The removal plans submitted for approval by the contractor shall include a listing of protection measures to be employed by the contractor to shield public areas, streets and

existing buildings from damage from free falling debris or removal equipment. The cost of labor and other items required to protect property and life including the designing, furnishing, installing, maintaining and removal of any temporary shoring shall not be paid for directly but shall be considered subsidiary to other items for which direct payment is made.

Environmental Restrictions

When the contract plans require reuse of concrete rubble as riprap, the concrete which is to be used as rubble or riprap shall be free of bituminous material and protruding reinforcing steel. The bituminous wearing surface shall be removed from the bridge deck by milling or other approved techniques prior to commencing deck demolition procedures. Other materials that are not to be salvaged shall become the property of the contractor and shall be removed from the right-of-way.

During deck demolition, concrete rubble shall not be allowed to fall into rivers or streams or into impoundments so near to such waters they will be carried into any river or stream by normal stream flow or surface runoff. The dropping of parts or components of structures into any body of water will not be permitted unless there is no other practical method of removal and approval of such practice is obtained from the engineer, the coast guard, the corps of engineers and other approriate regulatory agencies. Removal from the water of any part or component of a structure shall be done so as to keep any resulting siltation to a minimum. The use of equipment in any body of water shall be limited to those operations which would be impossible or impractical to perform in any other way and shall be so controlled as to minimize any erosion or siltation resulting from its operation. Airborne dust from removal operations shall also be controlled to minimize pollution and visual impacts.

Experimental Equipment

To encourage the development and use of new or improved equipment, the contractor may request, in writing, permission to use alternative equipment or methods in place of that specified or shown on the plans. The engineer, before considering or granting such a request, may require the contractor to furnish, at his/her own expense, evidence satisfactory to the engineer that the equipment proposed to be used is capable of producing work equal to that which can be produced with the specified equipment. If such permission is granted, it shall be with the understanding that the permission may be withdrawn at any time at which the engineer determines the results, in his/her opinion, are not at least equal to the results obtained with currently acceptable equipment. Upon withdrawal of such permission by the engineer, the contractor will be required to use the equipment originally specified and shall, in accordance with the direction of the engineer, remove and dispose of or otherwise remedy, at the contractor's expense, any defective or unsatisfactory work produced with the alternative equipment.

The approval for use of particular equipment or construction method on a project shall in no way be considered as approval for use of such equipment on other projects and shall not relieve the contractor of his/her responsibility to produce finished work of the quality required by the plans and specifications. No additional compensation will be allowed the contractor for any delays or additional costs which are incurred as a result of the application of this provision.

Basis Of Payment

The item "Removal of Existing Bridge Deck" as described herein and within the limits shown on the plans shall be measured for payment as a single unit. The lump sum

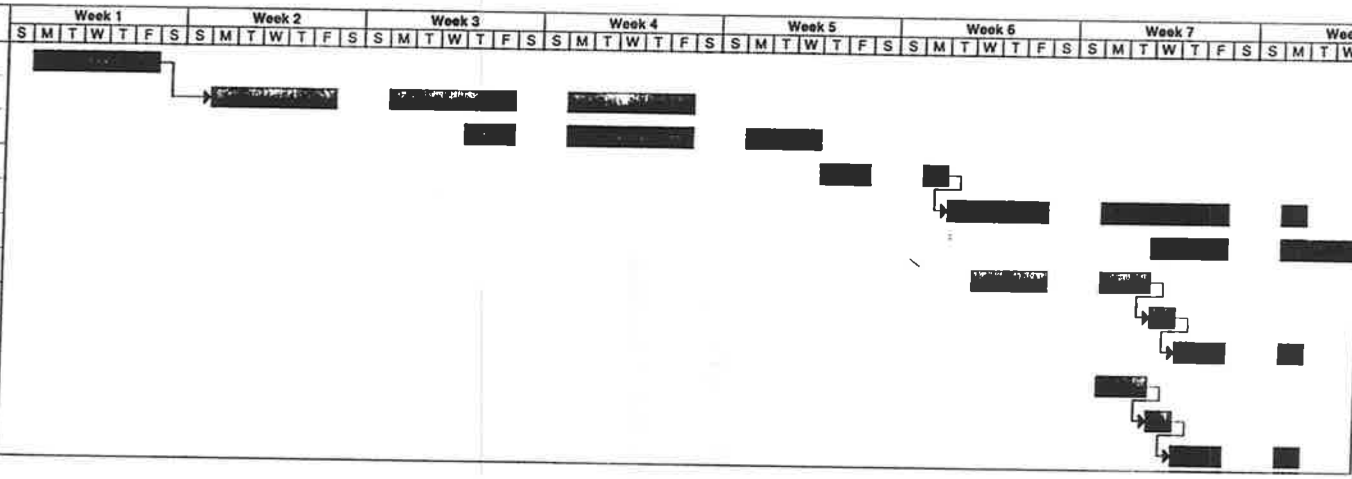
price paid shall be full compensation for removal of existing concrete, cutting, drilling, hammering and all other work involved in the removal and disposal of the concrete bridge deck and other materials from the existing structure necessary to provide for the new bridge deck. The lump sum price shall also include full compensation for cleaning, extending and lapping to existing reinforcing bars at portions of the existing structure to remain; maintenance of traffic; providing protection to the public from removal operations; repair of damaged bridge members to remain; disposal of all materials which are not specified to be salvaged; and furnishing of all materials, labor, tools, equipment and incidentals necessary to complete the work.

APPENDIX Q

Constuction Timelines

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Conventional CIP Deck, No Overtime

ID	Task Name	Duration	Notes
1	Remove Deck	5d	
2	Set Forms	15d	
3	Tie Deck Reinforcing	10d	
4	Check Screed & Pour Deck	3d	
5	Cure Deck	10d	Working days shown; 14 days actual cure time
6	Remove Forms	6d	Form removal completed after opened to traffic
7	Tie Rail Reinforcing	5d	
8	Pour Rail	1d	
9	Cure Rail	3d	Working days shown; 5 days actual cure time
10	Set Expansion Joints	2d	
11	Pour Concrete at Exp. Joints	1d	
12	Cure Concrete at Exp. Joints	3d	Working days shown; 5 days actual cure time

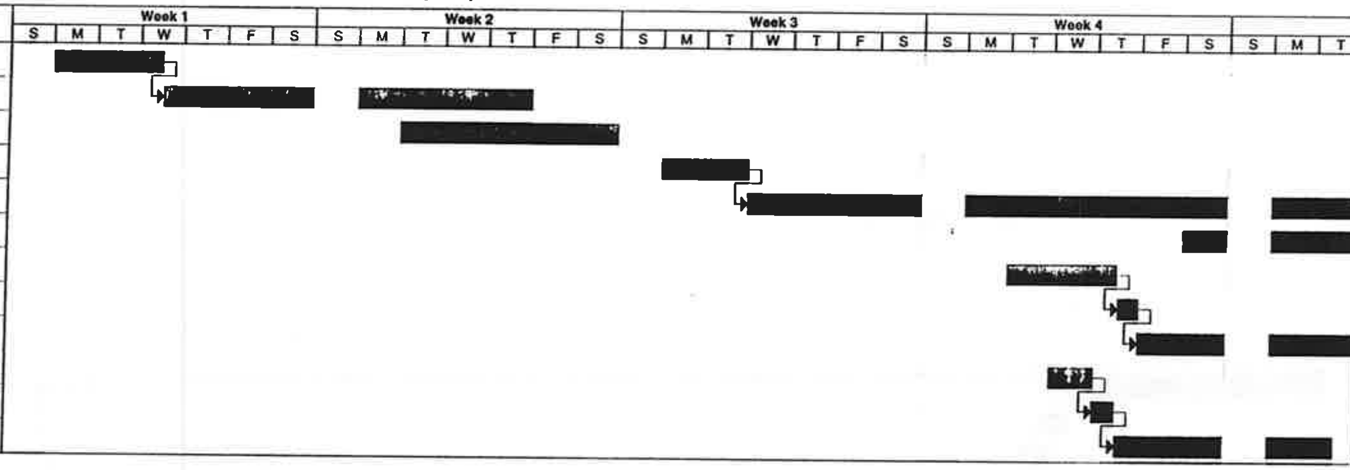


Task | Milestone ◆ | Rolled Up Task | Rolled Up Progress

ress | S | Rolled U ne ◇

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Conventional CIP Deck, 16hour days/6 day weeks

ID	Task Name	Duration	Notes
1	Remove Deck	2.5d	
2	Set Forms	7.5d	
3	Tie Deck Reinforcing	5d	
4	Check Screed & Pour Deck	2d	
5	Cure Deck	12d	Working days shown; 14 days actual cure time
6	Remove Forms	3d	
7	Tie Rail Reinforcing	2.5d	
8	Pour Rail	0.5d	
9	Cure Rail	4d	Working days shown; 5 days actual cure time
10	Set Expansion Joints	1d	
11	Pour Concrete at Exp. Joints	0.5d	
12	Cure Concrete at Exp. Joints	4d	Working days shown; 5 days actual cure time

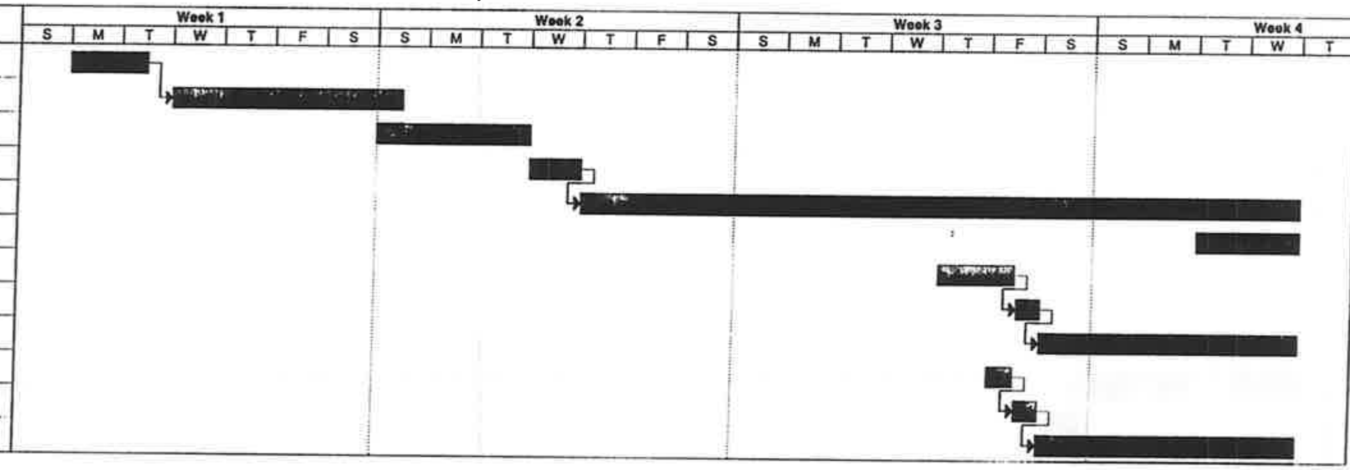


ject: NCHRP Proj # 12-41
e: 2/11/97

Task | Milestone | Rolled Up Task | Rolled Up Progress
Progress | Summary | Rolled Up Milestone

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Conventional CIP Deck, 24 hour shifts, 7 day weeks

ID	Task Name	Duration	Notes
1	Remove Deck	1.5d	
2	Set Forms	4.5d	
3	Tie Deck Reinforcing	3d	
4	Check Screed & Pour Deck	1d	
5	Cure Deck	14d	
6	Remove Forms	2d	
7	Tie Rail Reinforcing	1.5d	
8	Pour Rail	0.5d	
9	Cure Rail	5d	
10	Set Expansion Joints	0.5d	
11	Pour Concrete at Exp. Joints	0.5d	
12	Cure Concrete at Exp. Joints	5d	

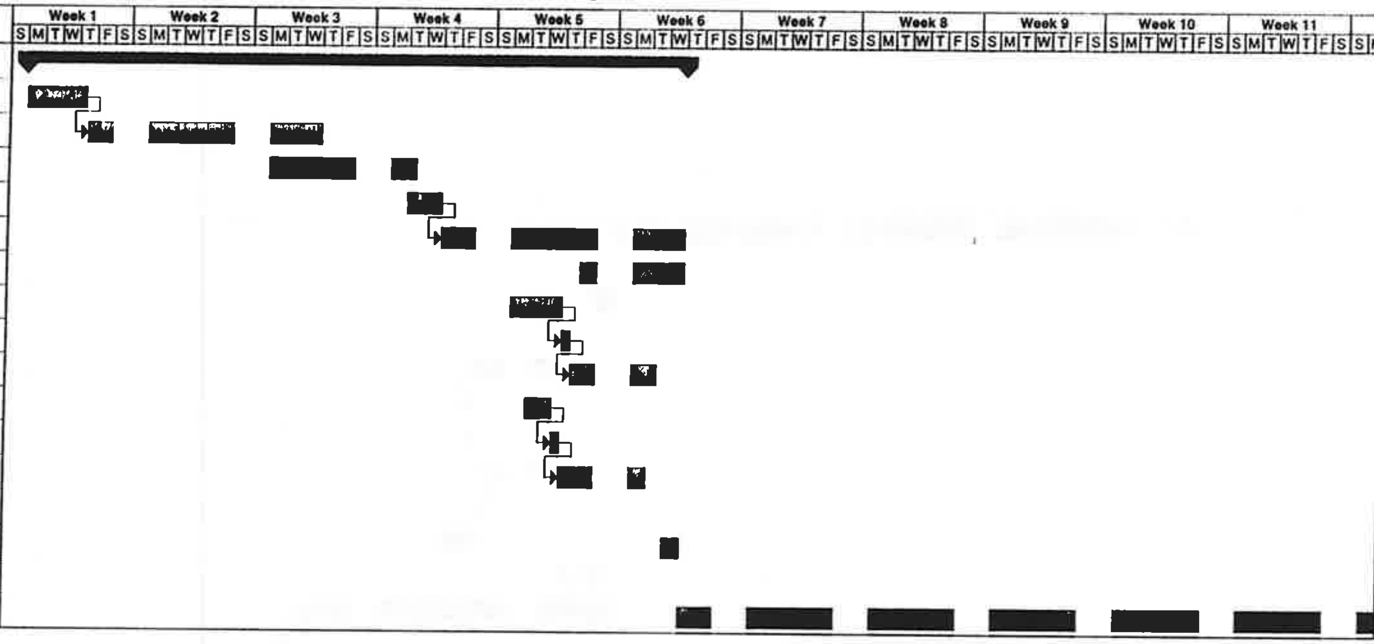


Project: NCHRP Proj # 12-41
Date: 2/11/97

Task
Progress
Milestone
Rolled Up Task
Rolled Up Milestone
Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Conventional CIP Deck, No Overtime, Maintain Traffic on 1/2 of Bridge

ID	Task Name	Duration	Notes
1	**Phase 1**	28d	Duration in working days
2	Remove Half of Deck	3.25d	
3	Set Forms	9.5d	
4	Tie Deck Reinforcing	6.5d	
5	Check Screed/Pour Deck	2d	
6	Cure Deck	10d	Working days shown;14 day actual cure
7	Remove Forms	4d	
8	Tie Rail Reinforcing	3d	
9	Pour Rail	0.5d	
10	Cure Rail	3d	Working days shown;5 day actual cure
11	Set Expansion Joints	1.5d	
12	Pour Concrete at Exp. Jnt.	0.5d	
13	Cure Concrete at Exp. Jnt.	3d	Working days shown;5 day actual cure
14			
15	**Switch Traffic**	1d	
16			
17	**Phase 2 (Repeat 2-13)**	28d	Assume same duration as Phase 1

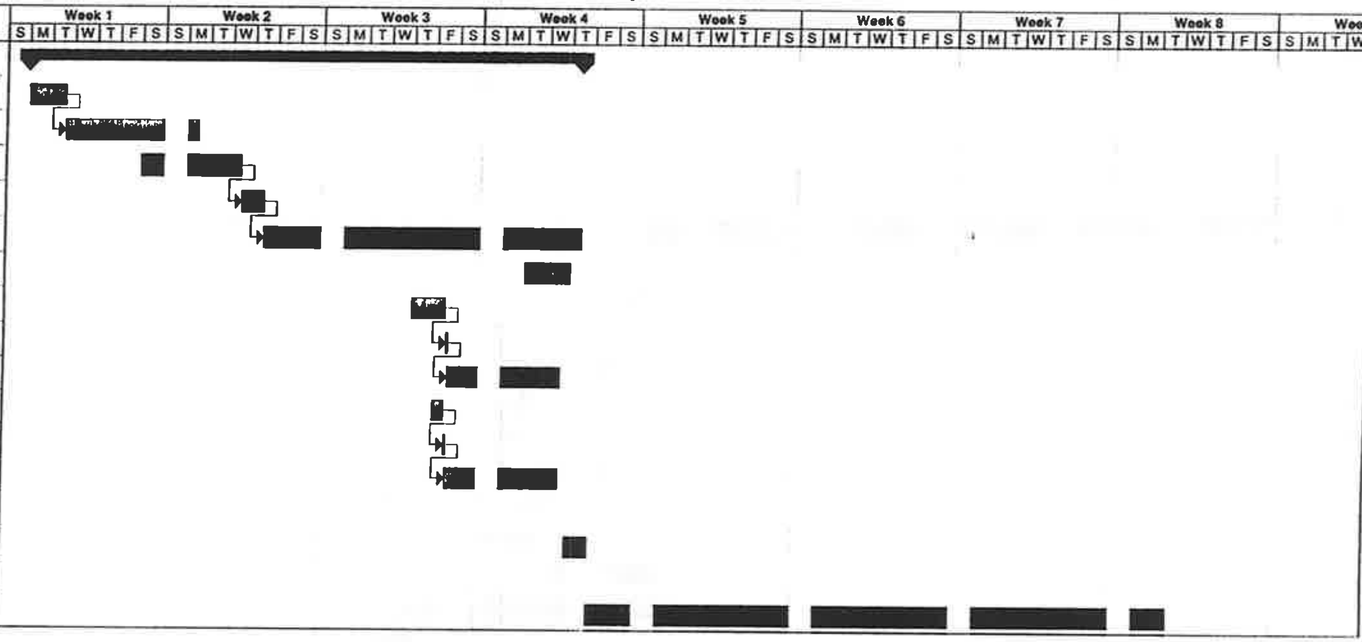


oject: NCHRP Proj. # 12-41
te: 2/11/97

Task
Progress
Milestone
Summary
Rolled Up Task
Rolled Up Milestone
Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Conventional CIP Deck, 16hour days/6 day weeks, Maintain Traffic on 1/2 of Bridge

ID	Task Name	Duration	Notes
1	**Phase 1**	21.38d	Duration in working days
2	Remove Half of Deck	1.75d	
3	Set Forms	4.75d	
4	Tie Deck Reinforcing	3.25d	
5	Check Screed/Pour Deck	1d	
6	Cure Deck	12d	Working days shown; 14 day actual cure
7	Remove Forms	2d	
8	Tie Rail Reinforcing	1.5d	
9	Pour Rail	0.25d	
10	Cure Rail	4d	Working days shown; 5 day actual cure
11	Set Expansion Joints	0.5d	
12	Pour Concrete at Exp. Jnt.	0.25d	
13	Cure Concrete at Exp. Jnt.	4d	Working days shown; 5 day actual cure
14			
15	**Switch Traffic**	1d	
16			
17	Phase 2 (Repeat 2-13)	21.5d	Assume approx. same duration as Ph. 1

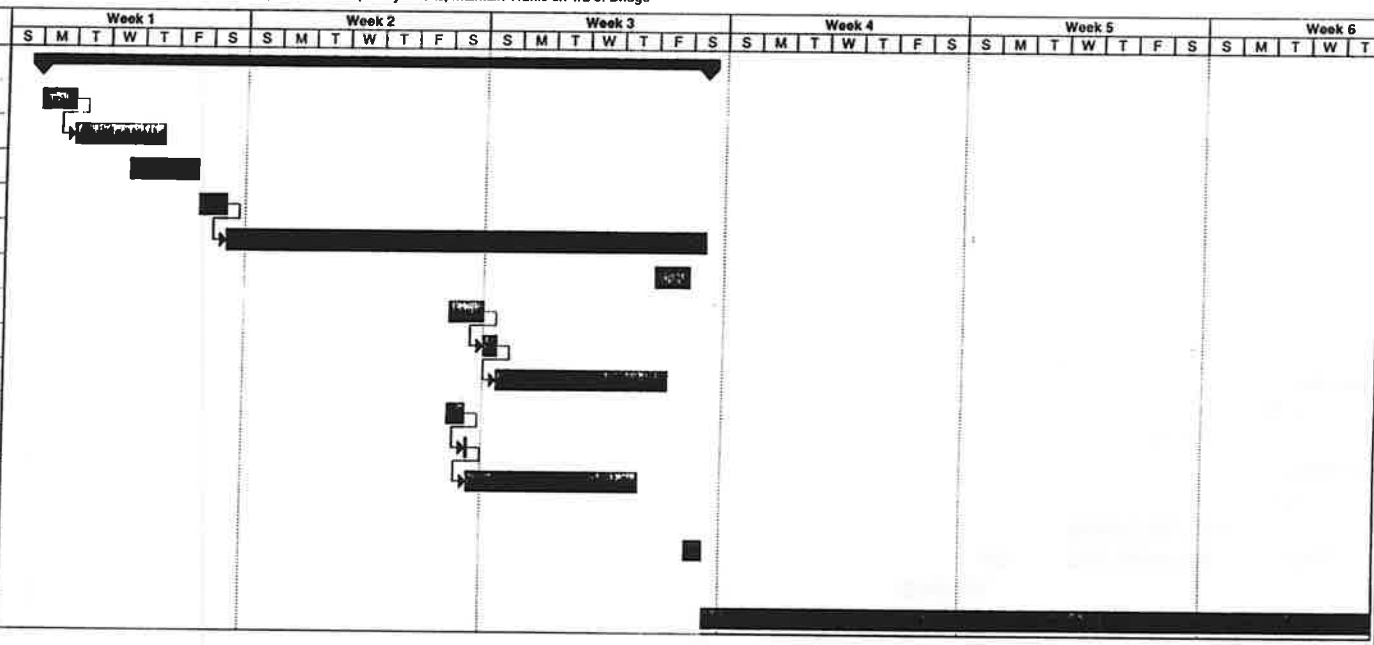


Project: NCHRP Proj # 12-41
Date: 2/11/97

Task | Milestone | Rolled Up Task | Rolled Up Progress
Progress | Rolled Milestone

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Conventional CIP Deck, 24 hour shifts, 7 day weeks, Maintain Traffic on 1/2 of Bridge

ID	Task Name	Duration	Notes
1	**Phase 1**	**19.38d**	
2	Remove Half of Deck	1d	
3	Set Forms	2.75d	
4	Tie Deck Reinforcing	2d	
5	Check Screed/Pour Deck	0.5d	
6	Cure Deck	14d	High Early Strength Concrete preferred
7	Remove Forms	1d	
8	Tie Rail Reinforcing	1d	
9	Pour Rail	0.2d	
10	Cure Rail	5d	High Early Strength Concrete preferred
11	Set Expansion Joints	0.5d	
12	Pour Concrete at Exp. Jnt.	0.2d	
13	Cure Concrete at Exp. Jnt.	5d	High Early Strength Concrete preferred
14			
15	**Switch Traffic**	0.5d	
16			
17	**Phase 2 (Repeat 2-13)**	19.5d	Assume approx. same duration as Ph. 1

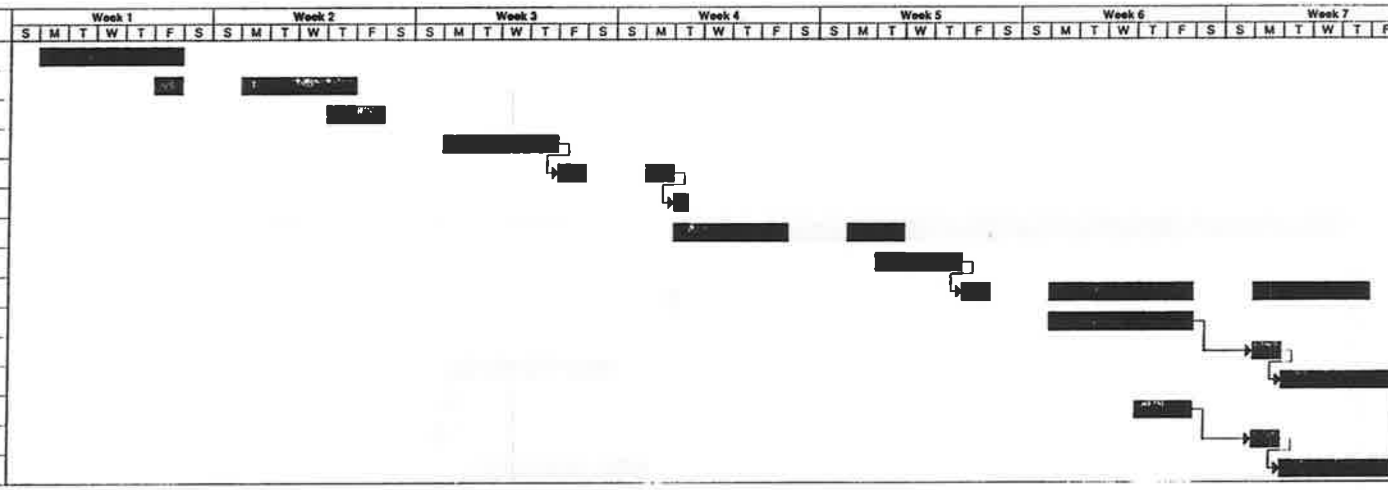


oject: NCHRP Proj. # 12-41
te: 2/11/97

Task | Milestone | Rolled Up Task | Rolled Up Progress
Progress | Summary | Rolled Up Milestone

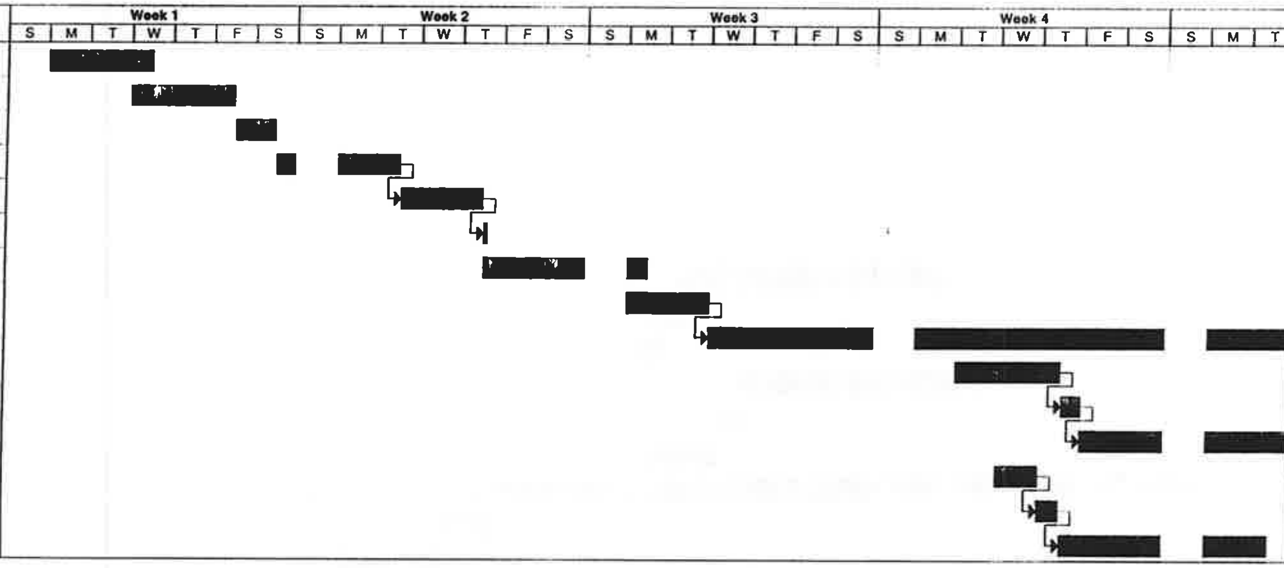
NCHRP PROJECT # 12-41
Idealized Construction Timeline
Standard Precast SIP Panels w/ CIP Topping, No Overtime

ID	Task Name	Duration	Notes
1	Remove Deck	5d	
2	Remove Existing Shear Studs	5d	
3	Install New Shear Studs	2d	
4	Set Grout Stops, Cantil. Forms & SIP Panels	4d	
5	Grout Panel Bedding	2d	
6	Cure Panel Bedding Grout	0.5d	
7	Tie Deck Reinforcing	6d	
8	Check Screed & Pour Deck	3d	
9	Cure Deck	10d	Working days shown; 14 days actual cure
10	Tie Rail Reinforcing	5d	
11	Pour Rail	1d	
12	Cure Rail	4d	Working days shown; 5 days actual cure
13	Set Expansion Joints	2d	
14	Pour Concrete at Exp. Joints	1d	
15	Cure Concrete at Exp. Joints	4d	Working days shown; 5 days actual cure



NCHRP PROJECT # 12-41
Idealized Construction Timeline
Standard Precast SIP Panels w/ CIP Topping, 16 Hour Days/6 Day Weeks

ID	Task Name	Duration	Notes
1	Remove Deck	2.5d	
2	Remove Existing Shear Studs	2.5d	
3	Install New Shear Studs	1d	
4	Set Grout Stops, Cantil. Forms & SIP Panels	2d	
5	Grout Panel Bedding	2d	
6	Cure Panel Bedding Grout	0.25d	
7	Tie Deck Reinforcing	3d	
8	Check Screed & Pour Deck	2d	
9	Cure Deck	12d	Working days shown; 14 days actual cure
10	Tie Rail Reinforcing	2.5d	
11	Pour Rail	0.5d	
12	Cure Rail	4d	Working days shown; 5 days actual cure
13	Set Expansion Joints	1d	
14	Pour Concrete at Exp. Joints	0.5d	
15	Cure Concrete at Exp. Joints	4d	Working days shown; 5 days actual cure

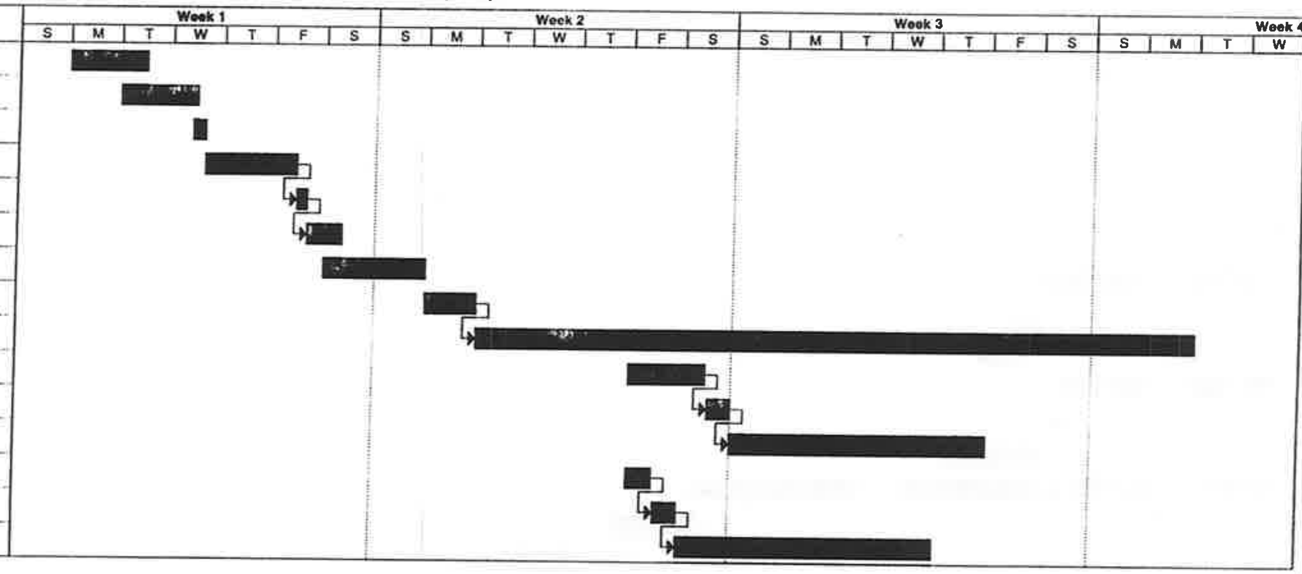


Project:NCHRP Proj # 12-41
Date: 2/11/97

Task | Milestone | Rolled Up Task | Rolled Up Progress
Progress | Summary | Rolled Up Milestone

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Standard Precast SIP Panels w/ CIP Topping, 24 Hour Days/7Day Weeks

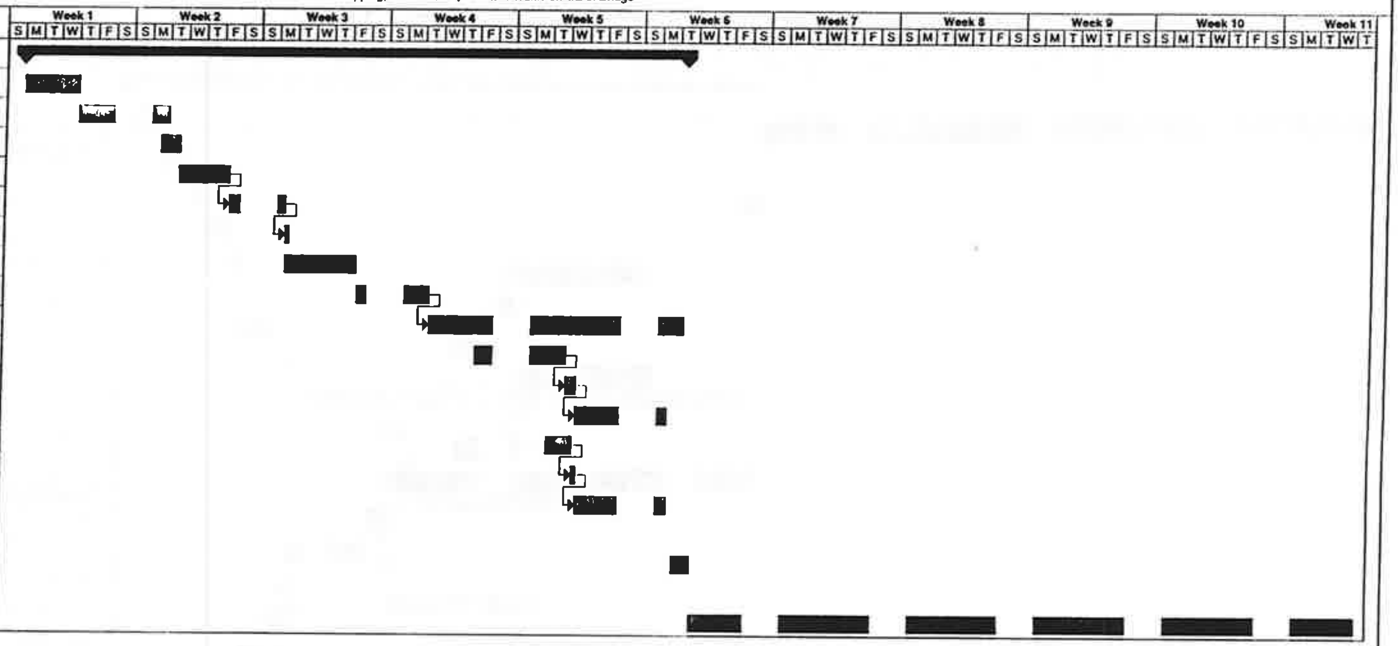
ID	Task Name	Duration	Notes
1	Remove Deck	1.5d	
2	Remove Existing Shear Studs	1.5d	
3	Install New Shear Studs	0.6d	
4	Set Grout Stops, Cantil. Forms & SIP Panels	1.5d	
5	Grout Panel Bedding	0.5d	
6	Cure Panel Bedding Grout	0.2d	
7	Tie Deck Reinforcing	2d	
8	Check Screed & Pour Deck	1d	
9	Cure Deck	14d	
10	Tie Rail Reinforcing	1.5d	
11	Pour Rail	0.5d	
12	Cure Rail	5d	
13	Set Expansion Joints	0.5d	
14	Pour Concrete at Exp. Joints	0.5d	
15	Cure Concrete at Exp. Joints	5d	



Task
Progress
Milestone
Rolled Up Task
Rolled Up Milestone
Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Standard Precast SIP Panels w/ CIP Topping, No Overtime; Maintain Traffic on 1/2 of Bridge

ID	Task Name	Duration	Notes
1	**Phase 1**	26.25d	
2	Remove Half of Deck	3d	
3	Remove Existing Studs	3d	
4	Install New Studs	1.25d	
5	Set Grout Stops, Cantil. Forms & Panels	2.5d	
6	Grout Panel Bedding	1.25d	
7	Cure Panel Bedding Grout	0.5d	
8	Tie Deck Reinforcing	3.75d	
9	Check Screed/ Pour Deck	2d	
10	Cure Deck	10d	Working days shown; 14 days actual cure
11	Tie Rail Reinforcing	3d	
12	Pour Rail	0.6d	
13	Cure Rail	3d	Working days shown; 5 days actual cure
14	Set Expansion Joints	1.25d	
15	Pour Concrete at Exp. Jnt.	0.6d	
16	Cure Concrete at Exp. Jnt.	3d	Working days shown; 5 days actual cure
17			
18	**Switch Traffic**	1d	
19			
20	**Phase 2 (Repeat 2-16)**	26.25d	Assumes same duration as Ph. 1

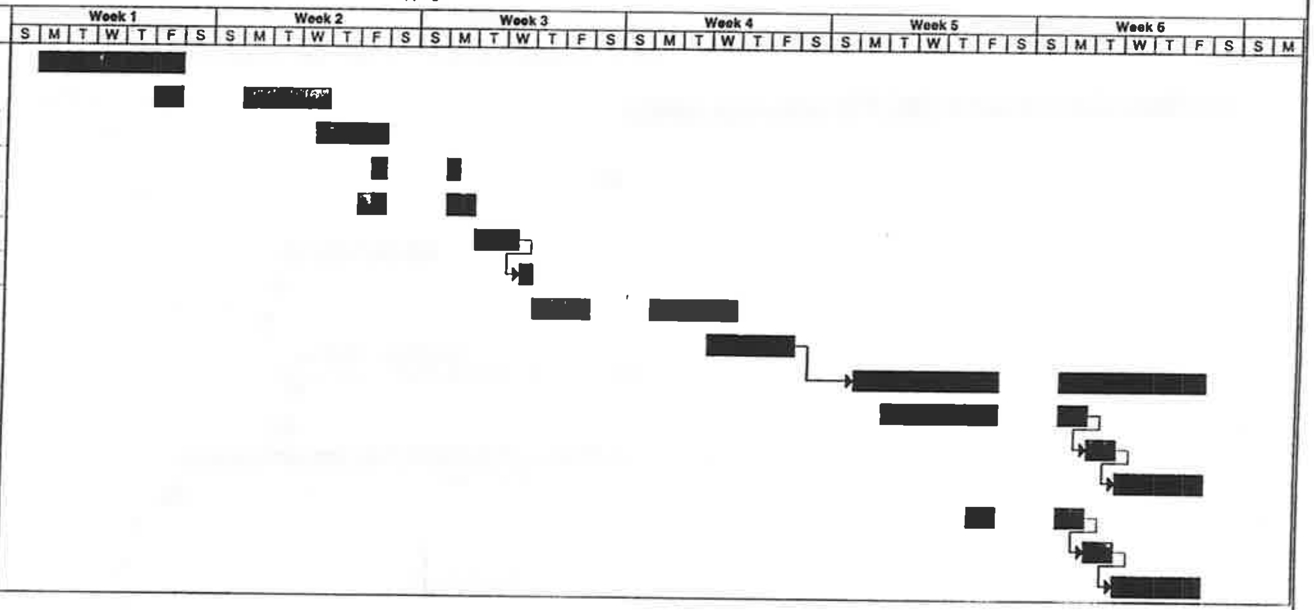


Project: NCHRP Proj # 12-41
Date: 2/11/97

Task | Progress | Milestone | Summary | Rolled Up Task | Rolled Up Milestone | Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Precast SIP Panels w/ Integral Cantilevers & CIP Topping, No Overtime

ID	Task Name	Duration	Notes
1	Remove Deck	5d	
2	Remove Existing Shear Studs (as needed)	4d	
3	Set Grout Stops & SIP Panels	2.5d	
4	Place Reinf. at Transverse Panel Joints	1d	
5	Install New Shear Studs	2d	
6	Grout Panel Bedding and Pockets at Joints	1.5d	
7	Cure Panel Bedding and Pocket Grout	0.5d	
8	Tie Deck Reinforcing	5d	
9	Check Screed & Pour CIP Topping	3d	
10	Cure CIP Topping	10d	Working days shown; 14 days actual cure
11	Tie Rail Reinforcing	5d	
12	Pour Rail	1d	
13	Cure Rail	3d	Working days shown; 5 days actual cure
14	Set Expansion Joints	2d	
15	Pour Concrete at Exp. Joints	1d	
16	Cure Concrete at Exp. Joints	3d	Working days shown; 5 days actual cure

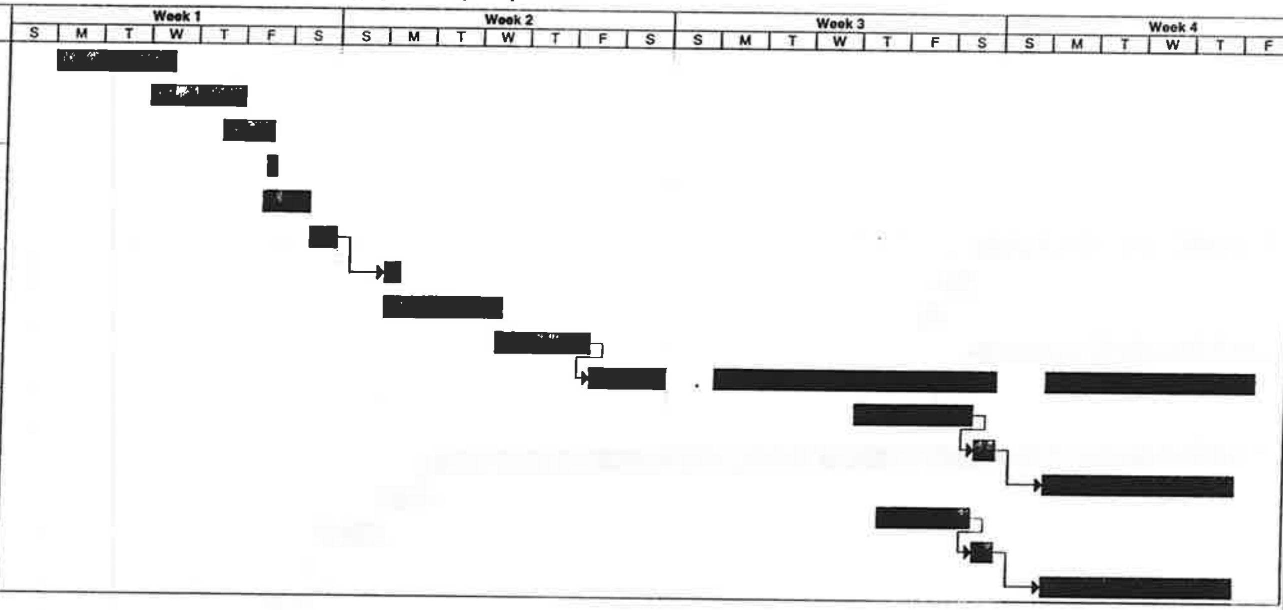


Project:NCHRP Proj. # 12-41
Date: 2/12/97

Task
Milestone
Rolled Up Task
Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Precast SIP Panels w/ Integral Cantilevers & CIP Topping,16 Hour Days/6 Day Weeks

ID	Task Name	Duration	Notes
1	Remove Deck	2.5d	
2	Remove Existing Shear Studs (as needed)	2d	
3	Set Grout Stops & SIP Panels	1.25d	
4	Place Reinf. at Transverse Panel Joints	0.5d	
5	Install New Shear Studs	1d	
6	Grout Panel Bedding and Pockets at Joints	0.75d	
7	Cure Panel Bedding and Pocket Grout	0.01d	Grout curing occurs on non-working day.
8	Tie Deck Reinforcing	2.5d	
9	Check Screed & Pour CIP Topping	2d	
10	Cure CIP Topping	12d	Working days shown; 14 days actual cure.
11	Tie Rail Reinforcing	2.5d	
12	Pour Rail	0.5d	
13	Cure Rail	4d	Working days shown; 5 days actual cure.
14	Set Expansion Joints	2d	
15	Pour Concrete at Exp. Joints	0.5d	
16	Cure Concrete at Exp. Joints	4d	Working days shown; 5 days actual cure.

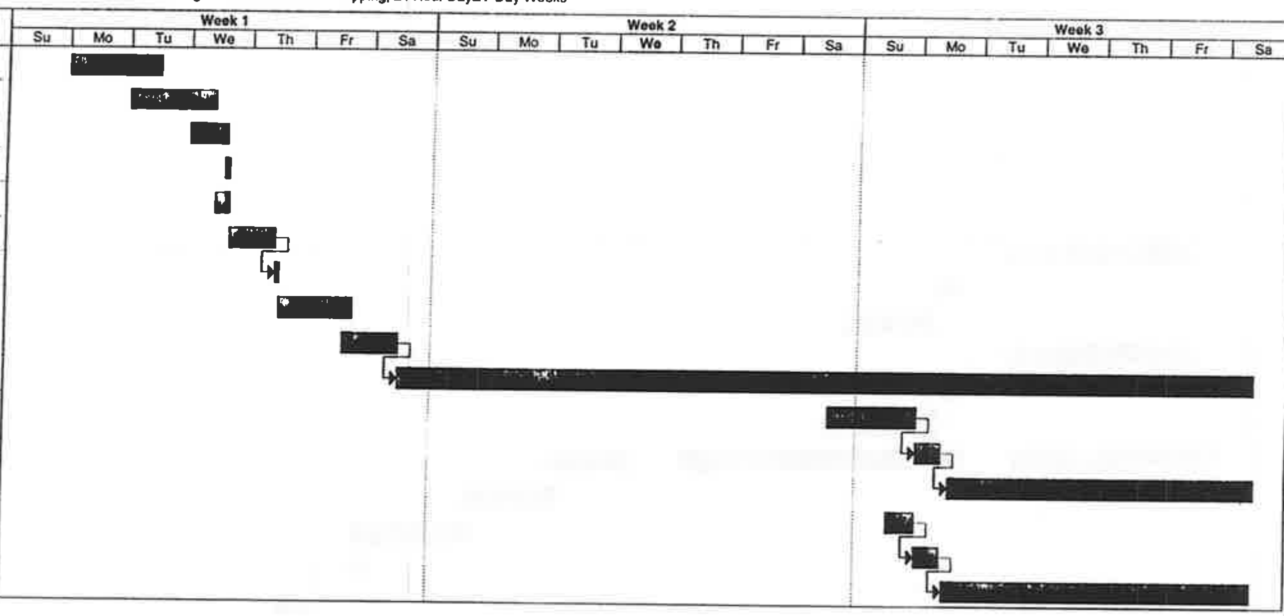


ect:NCHRP Proj. # 12-41
e: 2/13/97

Task
Progress
Milestone
Summary
Rolled Up Task
Rolled Up Milestone
Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Precast SIP Panels w/ Integral Cantilevers & CIP Topping, 24 Hour Days/7 Day Weeks

ID	Task Name	Duration	Notes
1	Remove Deck	1.5d	
2	Remove Existing Shear Studs (as needed)	1.25d	
3	Set Grout Stops & SIP Panels	0.75d	
4	Place Reinf. at Transverse Panel Joints	0.25d	
5	Install New Shear Studs	0.6d	
6	Grout Panel Bedding and Pockets at Joints	0.4d	
7	Cure Panel Bedding and Pocket Grout	0.2d	
8	Tie Deck Reinforcing	1.5d	
9	Check Screed & Pour CIP Topping	0.9d	
10	Cure CIP Topping	14d	
11	Tie Rail Reinforcing	1.5d	
12	Pour Rail	0.25d	
13	Cure Rail	5d	
14	Set Expansion Joints	0.5d	
15	Pour Concrete at Exp. Joints	0.25d	
16	Cure Concrete at Exp. Joints	5d	

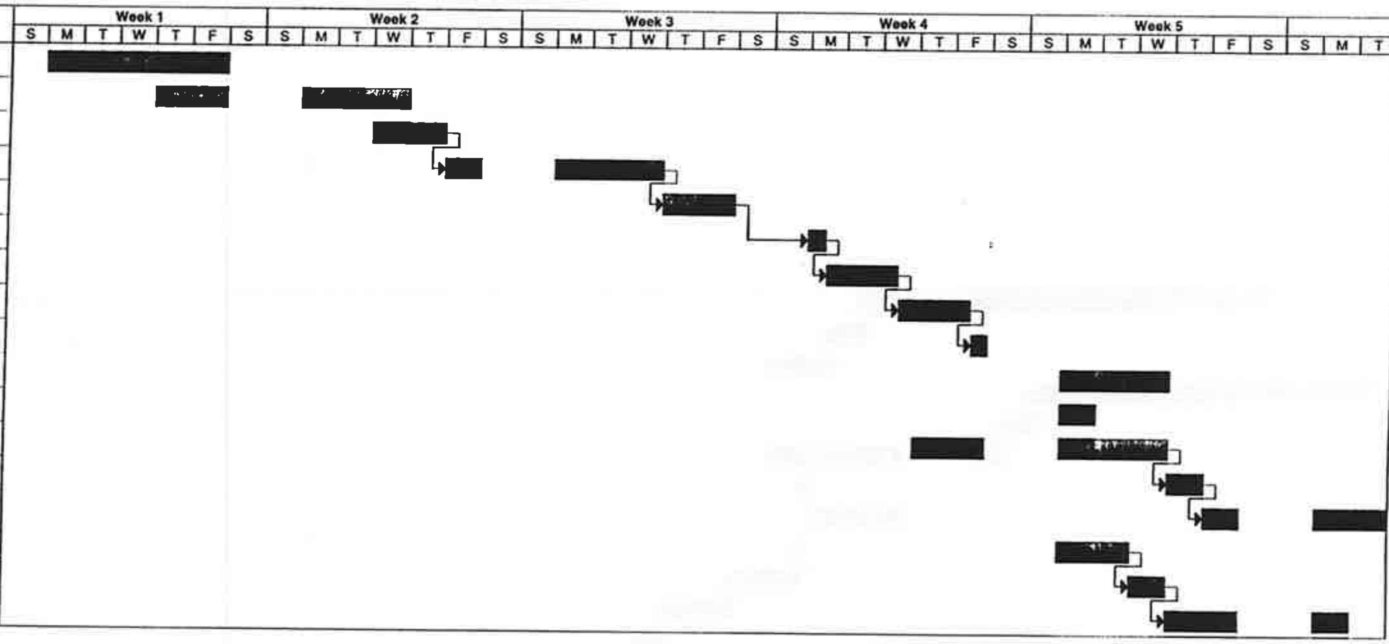


Project:NCHRP Proj. # 12-41
Date: 2/13/97

Task | Milestone | Rolled Up Task | Rolled Up Progress
gress | Rolled one

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Full Depth Precast; No Overtime

ID	Task Name	Duration	Notes
1	Remove Deck	5d	
2	Remove Existing Shear Studs	5d	
3	Install New Shear Studs	2d	
4	Set Grout Stops & Panels	4d	
5	Grout Transverse Keyways	2d	
6	Cure Keyway Grout	0.5d	
7	Apply Longit. Post-tensioning	2d	
8	Grout Voids Over Girders	2d	
9	Cure Grout @ Voids	0.5d	
10	Grind or Overlay Panels	3d	
11	Grout Tendon Ducts	1d	
12	Tie Rail Reinforcing	5d	
13	Pour Rail	1d	
14	Cure Rail	3d	Working days shown; 5 days actual cure
15	Set Expansion Joints	2d	
16	Pour Concrete at Exp. Joints	1d	
17	Cure Concrete at Exp. Joints	3d	Working days shown; 5 days actual cure

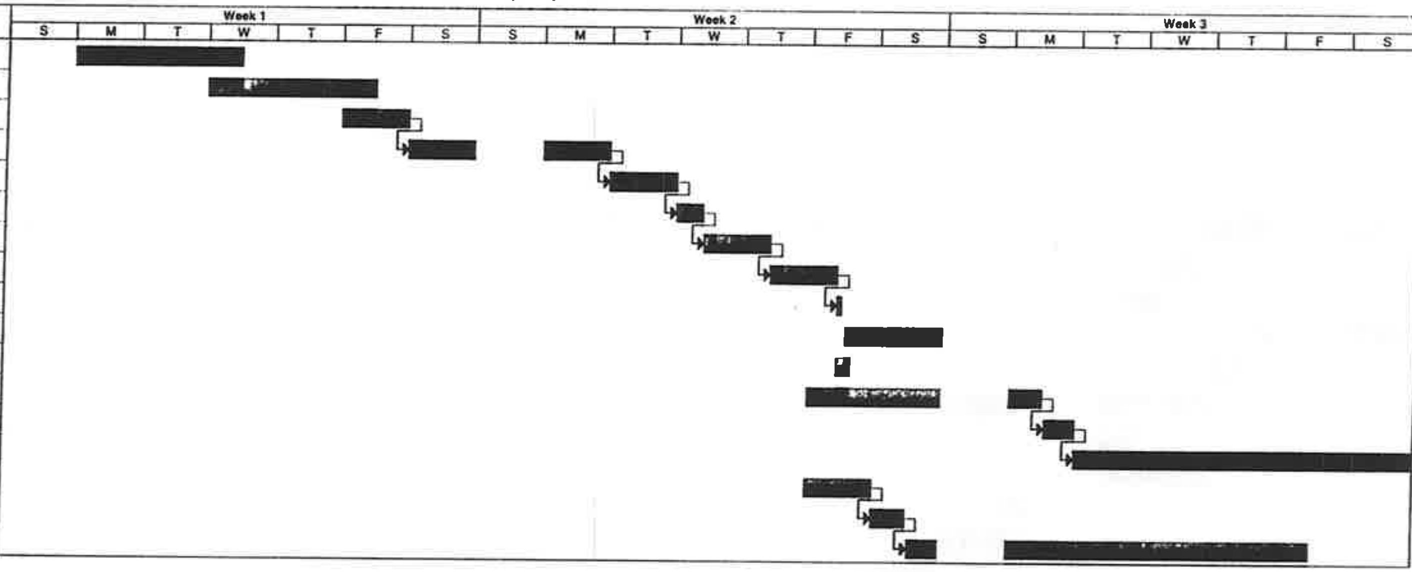


ect:NCHRP Proj. # 12-41
e: 2/11/97

Task | Progress | Milestone | Summary | Rolled Up Task | Rolled Up Milestone | Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Full Depth Precast; 16 Hour Days/6 day weeks

ID	Task Name	Duration	Notes
1	Remove Deck	2.5d	
2	Remove Existing Shear Studs	2.5d	
3	Install New Shear Studs	1d	
4	Set Grout Stops & Panels	2d	
5	Grout Transverse Keyways	1d	
6	Cure Keyway Grout	0.2d	
7	Apply Longit. Postensioning	1d	
8	Grout Voids Over Girders	1d	
9	Cure Grout @ Voids	0.2d	
10	Grind or Overlay Panels	1.5d	
11	Grout Tendon Ducts	0.5d	
12	Tie Rail Reinforcing	2.5d	
13	Pour Rail	0.5d	
14	Cure Rail	5d	
15	Set Expansion Joints	1d	
16	Pour Concrete at Exp. Joints	0.5d	
17	Cure Concrete at Exp. Joints	5d	

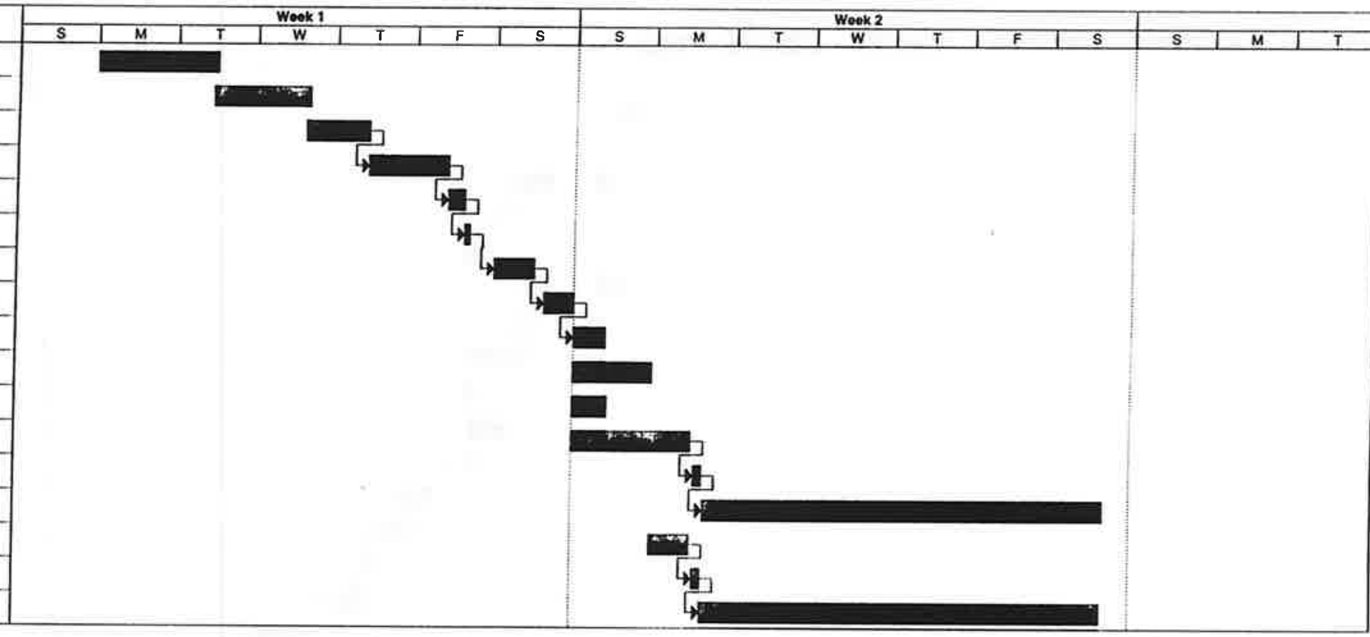


Project: NCHRP Proj # 12-41
Date: 2/11/97

Task | Milestone | Rolled Up Task | Rolled Up Progress
Summar | R lestone

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Full Depth Precast; 24 Hour Shifts, 7 Day Weeks

ID	Task Name	Duration	Notes
1	Remove Deck	1.5d	
2	Remove Existing Shear Studs	1.5d	
3	Install New Shear Studs	0.5d	
4	Set Grout Stops & Panels	1d	
5	Grout Transverse Keyways	0.5d	
6	Cure Keyway Grout	0.2d	
7	Apply Longit. Post-tensioning	0.5d	
8	Grout Voids Over Girders	0.25d	
9	Cure Grout @ Voids	0.2d	
10	Grind or Overlay Panels	1d	
11	Grout Tendon Ducts	0.3d	
12	Tie Rail Reinforcing	1.5d	
13	Pour Rail	0.3d	
14	Cure Rail	5d	
15	Set Expansion Joints	0.5d	
16	Pour Concrete at Exp. Joints	0.3d	
17	Cure Concrete at Exp. Joints	5d	

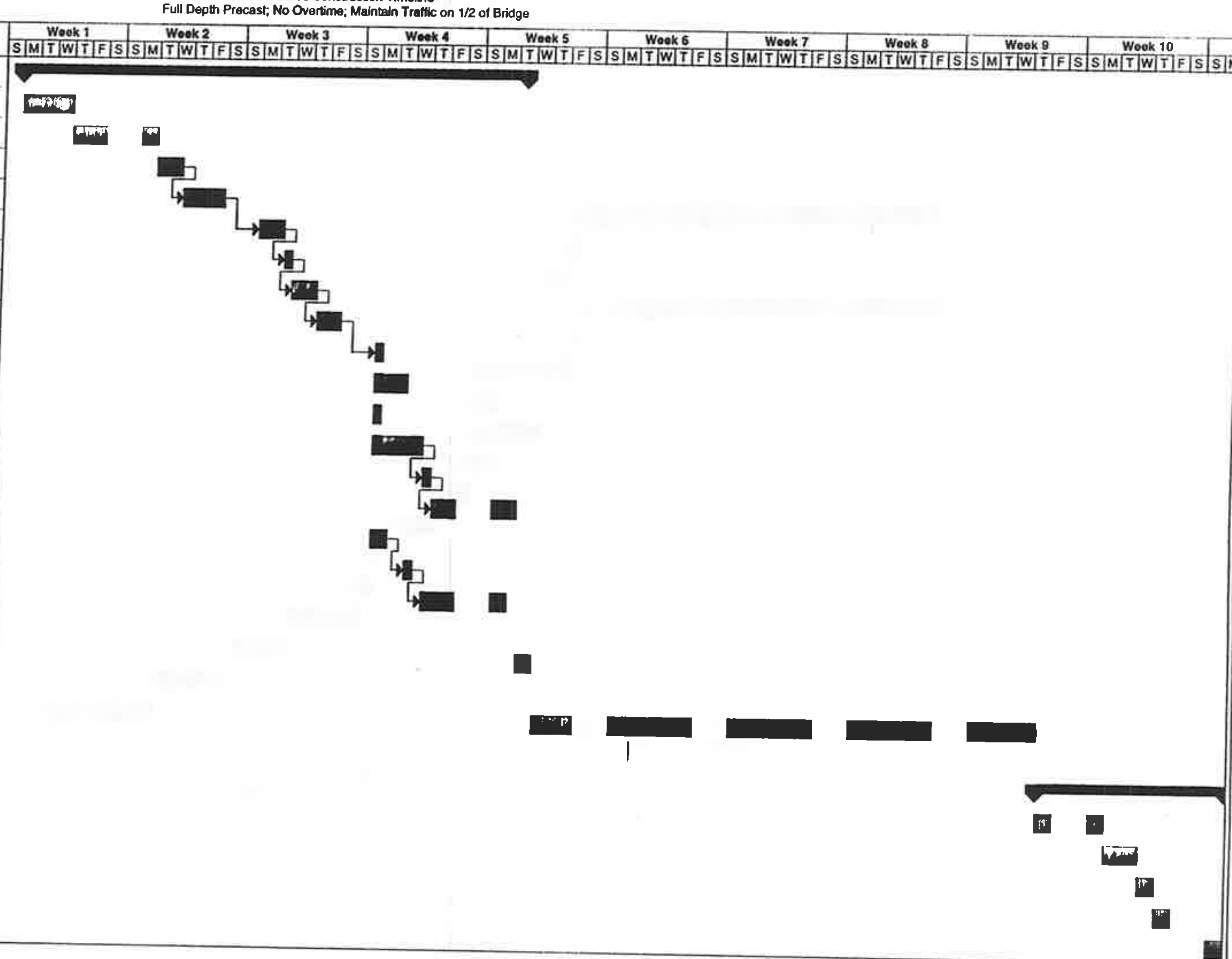


oject:NCHRP Proj. # 12-41
te: 2/11/97

Task
Progress
Milestone
Summary
Rolled Up Task
Rolled Up Milestone
Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Full Depth Precast; No Overtime; Maintain Traffic on 1/2 of Bridge

ID	Task Name	Duration	Notes
1	**Phase 1**	**21.5d**	
2	Remove Half of Deck	3d	
3	Remove Existing Studs	3d	
4	Install New Studs	1.5d	
5	Set Grout Stops & Panels	2.5d	
6	Grout Transv. Keyways	1.5d	
7	Cure Keyway Grout	0.5d	
8	Apply Longit. PT	1.5d	
9	Grout Voids Over Girders	1.5d	
10	Cure Grout @ Voids	0.5d	
11	Grind or Overlay Panels	2d	
12	Grout Tendon Ducts	0.5d	
13	Tie Rail Reinforcing	3d	
14	Pour Rail	0.5d	
15	Cure Rail	3d	Working days shown; 5 days min cure
16	Set Expansion Joints	1d	
17	Pour Concrete at Exp. Jnt.	0.5d	
18	Cure Concrete at Exp. Jnt.	3d	Working days shown; 5 days actual cure
19			
20	**Switch Traffic**	1d	
21			
22	**Phase 2 (Repeat 2-18)**	21.5d	Assumes same duration as Phase 1
23			
24	Phase 3	7d	
25	Form Longit. Joint	2d	
26	PT Longit. Joint	2d	
27	Cast Longit. Joint	1d	
28	Cure Longit. Joint	1d	
29	Grind or Overlay Joint	1d	

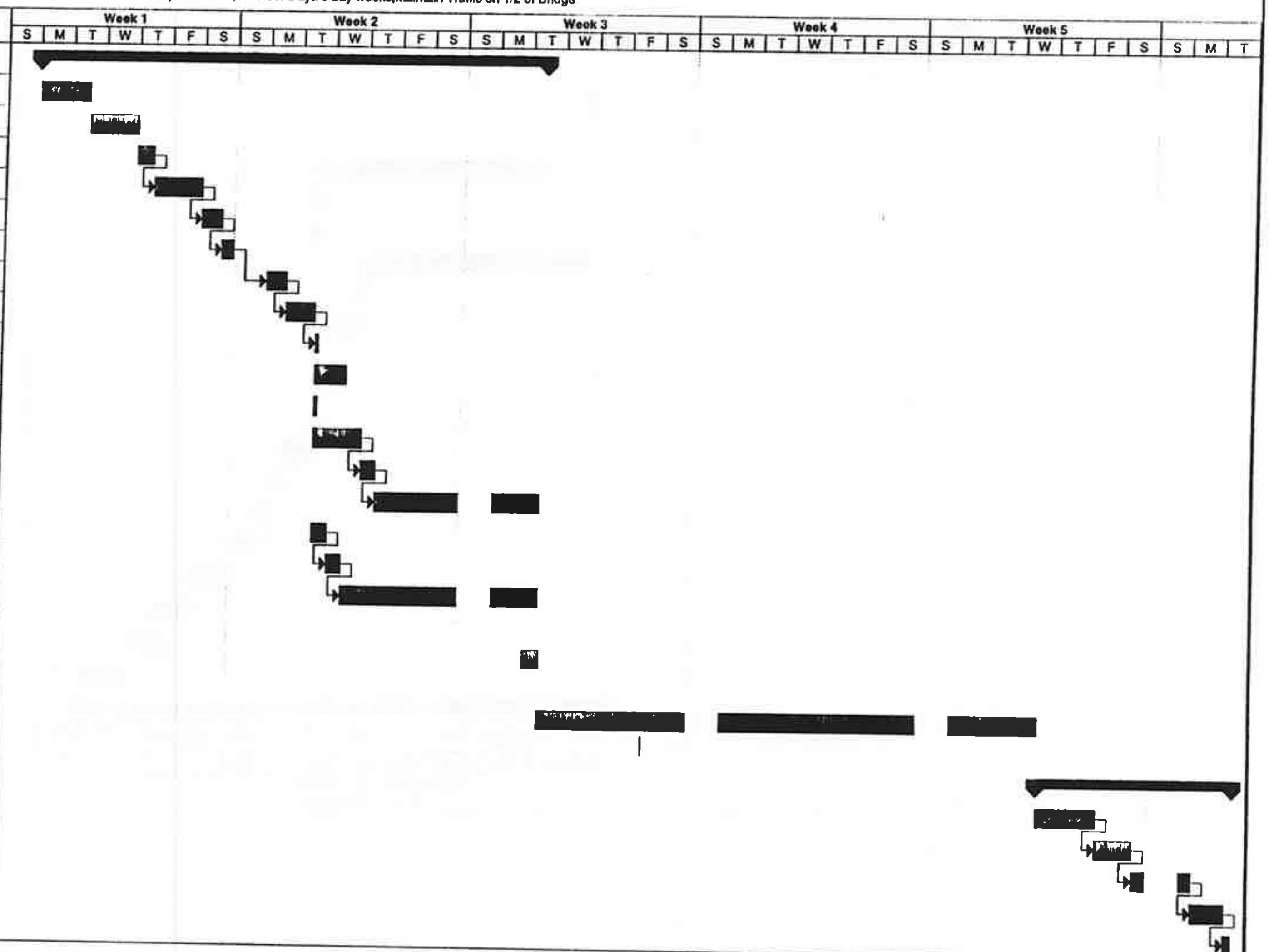


Project:NCHRP Proj. # 12-41
Date: 2/11/97

Task
…ress
Milestone
S…
Rolled Up Task
Rolled U… …ne
Rolled Up Progress

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Full Depth Precast; 16 Hour Days/6 day weeks;Maintain Traffic on 1/2 of Bridge

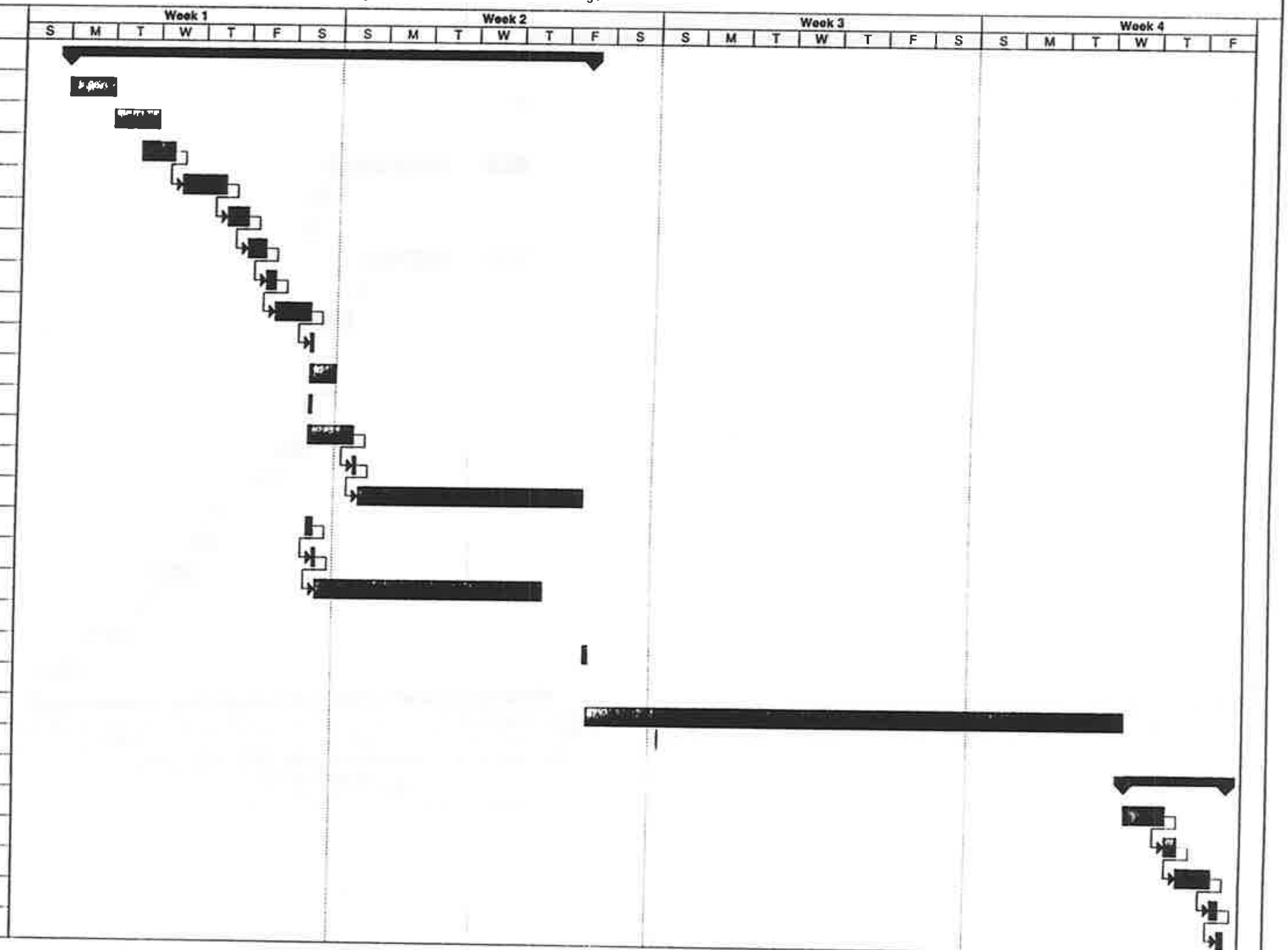
ID	Task Name	Duration	Notes
1	**Phase 1**	13.3d	
2	Remove Half of Deck	1.5d	
3	Remove Existing Studs	1.5d	
4	Install New Studs	0.5d	
5	Set Grout Stops & Panels	1.5d	
6	Grout Transv. Keyways	0.75d	
7	Cure Keyway Grout	0.25d	
8	Apply Longit. PT	0.75d	
9	Grout Voids Over Girders	0.75d	
10	Cure Grout @ Voids	0.25d	
11	Grind or Overlay Panels	1d	
12	Grout Tendon Ducts	0.3d	
13	Tie Rail Reinforcing	1.5d	
14	Pour Rail	0.3d	
15	Cure Rail	4d	Working days shown; 5 days actual cure
16	Set Expansion Joints	0.5d	
17	Pour Concrete at Exp. Jnt.	0.3d	
18	Cure Concrete at Exp. Jnt.	5d	
19			
20	**Switch Traffic**	0.5d	
21			
22	**Phase 2 (Repeat 2-18)**	13.5d	Assumes approx. same duration as Ph. 1
23			
24	Phase 3	4.75d	
25	Form Longit. Joint	1.5d	
26	PT Longit. Joint	1.25d	
27	Cast Longit. Joint	0.5d	
28	Cure Longit. Joint	1d	
29	Grind or Overlay Joint	0.5d	



Project:NCHRP Proj. # 12-41 Date: 2/11/97	Task	Milestone	Rolled Up Task	Rolled Up Progress
	Progress	Summary	Rolled Up Milestone	

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Full Depth Precast; 24 Hour Shifts, 7 Day Weeks; Maintain Traffic on 1/2 of Bridge

ID	Task Name	Duration	Notes
1	**Phase 1**	11.5d	
2	Remove Half of Deck	1d	
3	Remove Existing Studs	1d	
4	Install New Studs	0.3d	
5	Set Grout Stops & Panels	1d	
6	Grout Transv. Keyways	0.5d	
7	Cure Keyway Grout	0.2d	
8	Apply Longit. PT	0.5d	
9	Grout Voids Over Girders	0.5d	
10	Cure Grout @ Voids	0.2d	
11	Grind or Overlay Panels	0.75d	
12	Grout Tendon Ducts	0.2d	
13	Tie Rail Reinforcing	1d	
14	Pour Rail	0.2d	
15	Cure Rail	5d	High Early Strength concrete preferred
16	Set Expansion Joints	0.3d	
17	Pour Concrete at Exp. Jnt.	0.2d	
18	Cure Concrete at Exp. Jnt.	5d	High Early Strength concrete preferred
19			
20	**Switch Traffic**	0.3d	
21			
22	**Phase 2 (Repeat 2-18)**	11.5d	Assumes same duration as Phase 1
23			
24	**Phase 3**	2.6d	
25	Form Longit. Joint	0.7d	
26	PT Longit. Joint	0.7d	
27	Cast Longit. Joint	0.4d	
28	Cure Longit. Joint	0.4d	
29	Grind or Overlay Joint	0.4d	

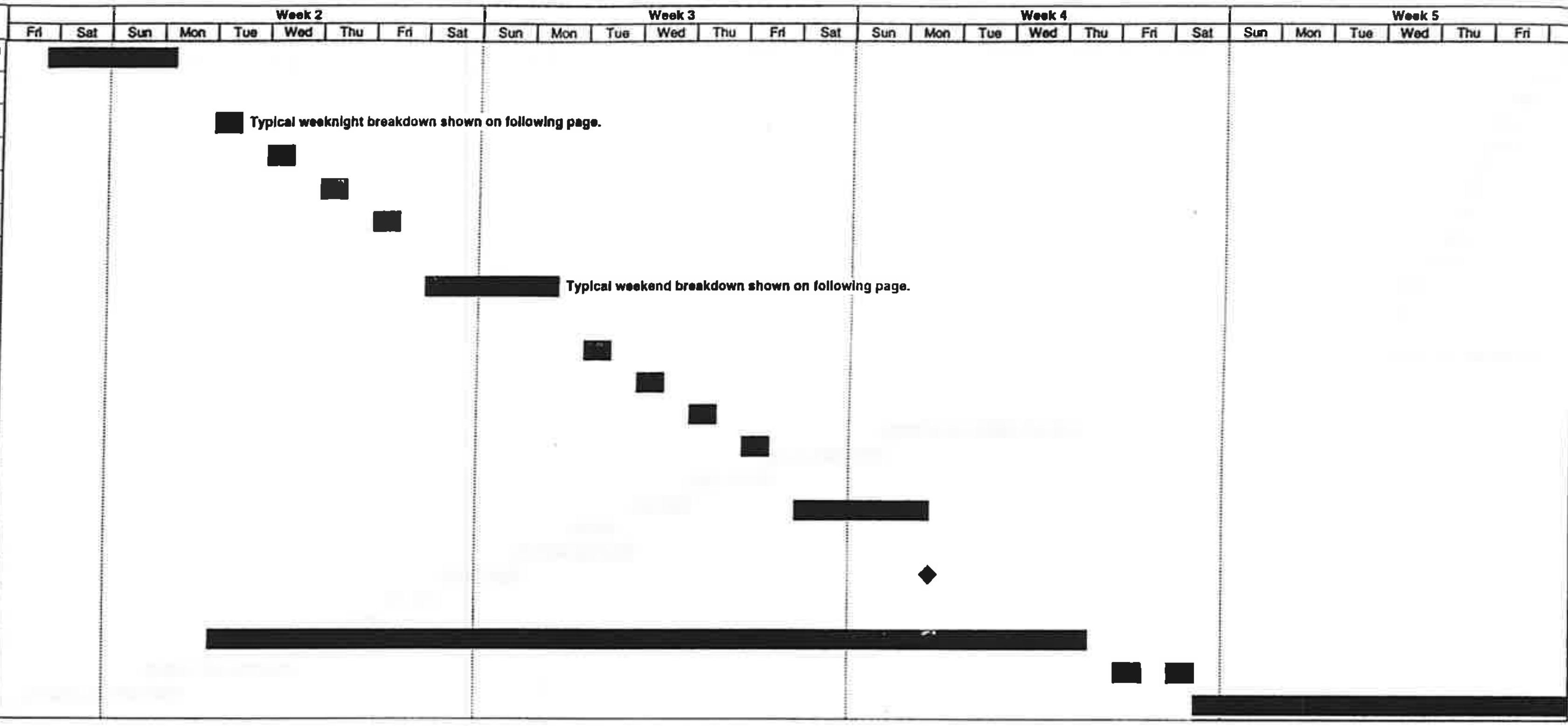


Project:NCHRP Proj # 12-41
Date: 2/11/97

Task
Milestone
Rolled Up Task
Rolled Up Progress
gress
Rolled
one

NCHRP PROJECT # 12-41
Idealized Construction Timeline
Full Depth Precast; Nighttime and Weekend Closure Only

ID	Task Name	Duration
1	Weekend #1 (7 Panels)	55.5h
2		
3	Weeknight (2 Panels)	10h
4	Weeknight (2 Panels)	10h
5	Weeknight (2 Panels)	10h
6	Weeknight (2 Panels)	10h
7		
8	Weekend #2 (8 Panels)	55.5h
9		
10	Weeknight (2 Panels)	10h
11	Weeknight (2 Panels)	10h
12	Weeknight (2 Panels)	10h
13	Weeknight (2 Panels)	10h
14		
15	Weekend #3 (8 Panels)	55.5h
16		
17	Redecking Complete	0h
18		
19	Tie Rail Reinforcing	226h
20	Pour Rail	16h
21	Cure Rail	168h

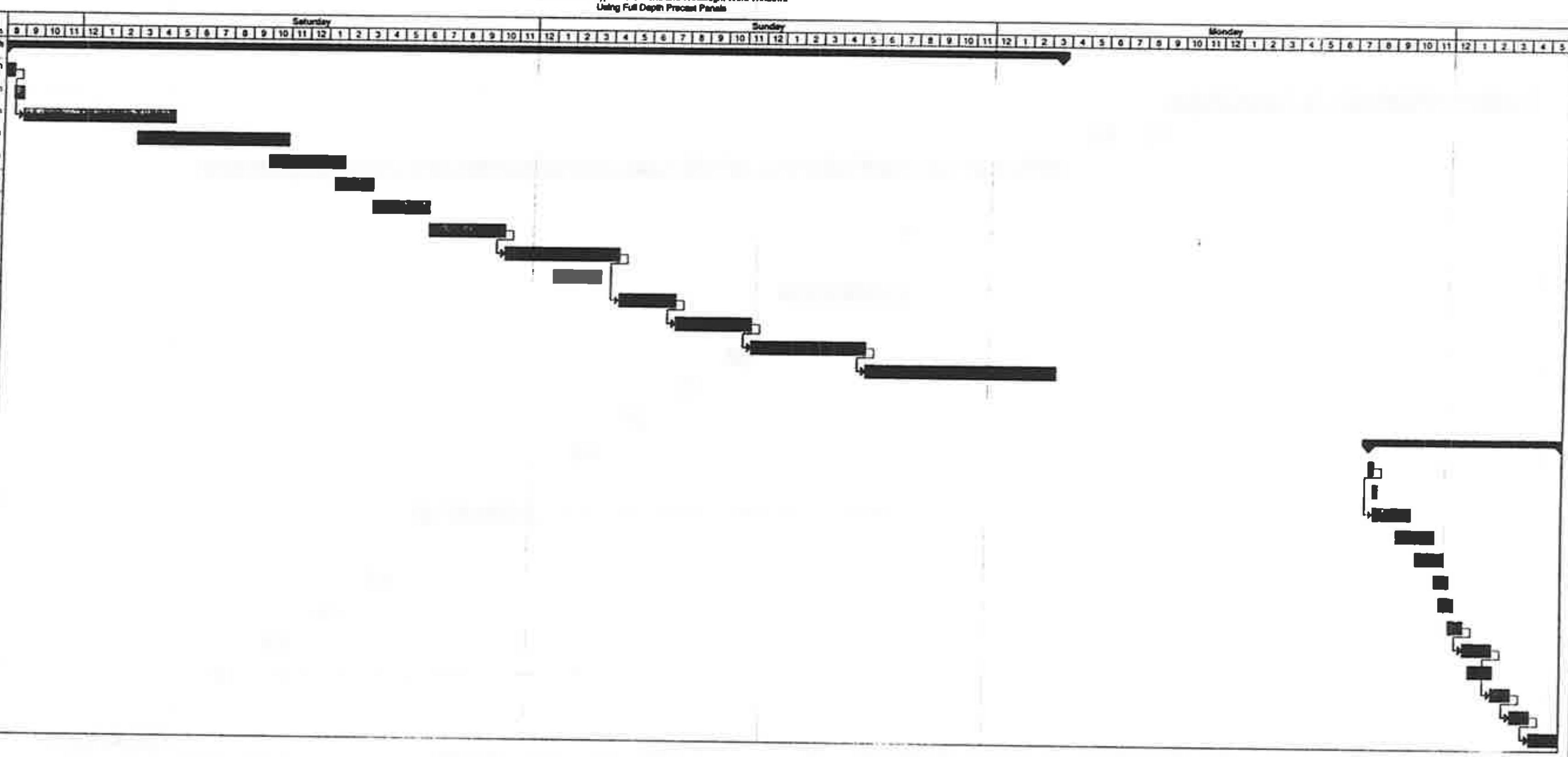


ject: NCHRP Proj. # 12-41
e: 2/12/97

Task
Progress
Milestone
Summary
Rolled Up Task
Rolled Up Milestone
Rolled Up Progress

NCHRP PROJECT # 12-41
Breakdown of Typical Weekend and Weeknight Work Windows
Using Full Depth Precast Panels

ID	Task Name	Duration
1	TYP. WEEKEND WINDOW (7-8 Panels)	68.5h
2	Close bridge/mobilize	0.5h
3	Move work area barriers	0.5h
4	Remove work area deck	8h
5	Remove old shear studs	8h
6	Install new shear studs	4h
7	Set grout stops	2h
8	Set panels	3h
9	Grout transv. keyways	4h
10	Cure transv. keyways	6h
11	Prepare/couple longit. PT	2.5h
12	Apply longit. PT	3h
13	Grout voids over girders	4h
14	Cure grout @ gird. voids	6h
15	Grind panels to profile	10h
16		
17		
18	TYP. WEEKNIGHT WINDOW (2 Panels)	10h
19	Close bridge/mobilize	0.25h
20	Move work area barriers	0.25h
21	Remove work area deck	2h
22	Remove old shear studs	2h
23	Install new shear studs	1.5h
24	Set grout stops	0.75h
25	Set panels	0.75h
26	Grout transv. keyways	0.75h
27	Cure transv. keyways	1.5h
28	Prepare/couple longit. PT	1.25h
29	Apply longit. PT	1h
30	Grout voids over girders	1h
31	Cure grout @ gird. voids	1.5h



Project: NCHRP Proj. # 12-41
Date: 2/12/97

Task | Progress | Milestone ◆ | Summary | Rolled Up Task | Rolled Up Milestone ◇ | Rolled Up Progress