AASHTO T-3 TRIAL DESIGN BRIDGE DESCRIPTION

State: *Illinois*

Trial Design Designation: *IL-5*

Bridge Name: ______________________________

Superstructure Type: *Simply supported PPC-I beam composite with concrete deck*

Span Length(s): *3@50 ft. (total 150 ft.)*

Substructure Type: *Trapezoidal pier columns supported on a deep grade beam*

Foundation: *Steel piles at abutments and bents*

Abutments: *Seat type supported on steel piles*

Seismic Design Category (SDC): *"C"*

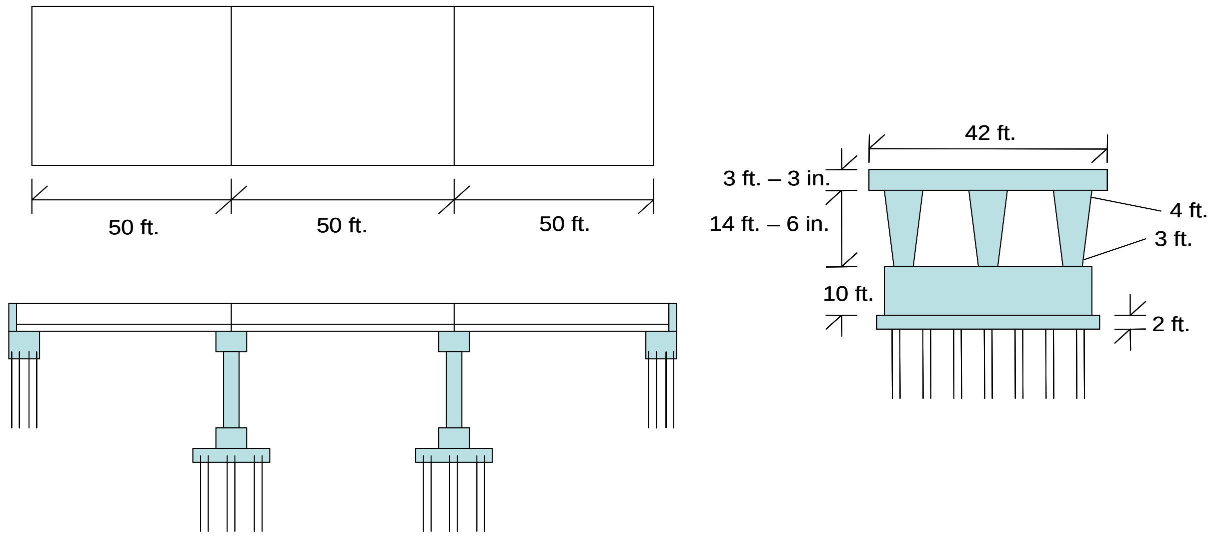
Seismic Design Strategy (Type 1, 2 or 3): *Type 1*

Design Spectral Acceleration at 1-second Period (S_{D1}): *0.487g*

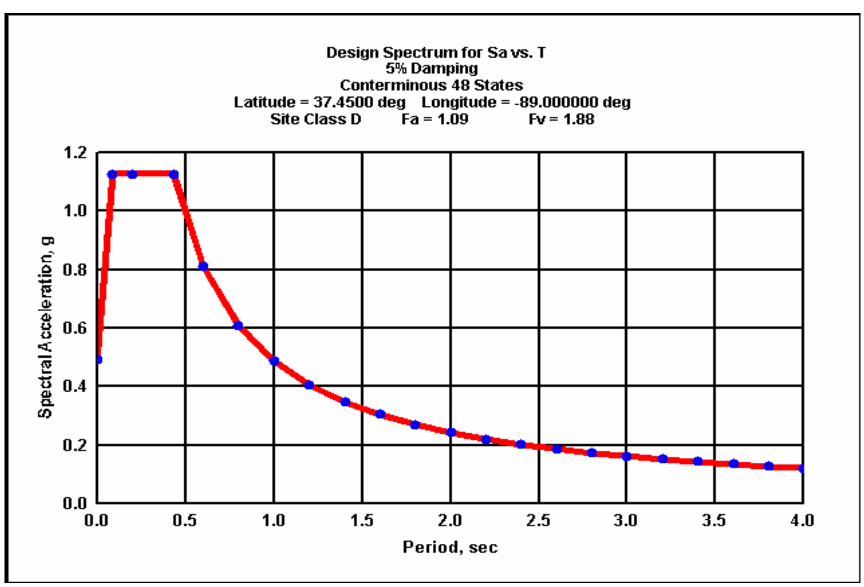
Additional Description (Optional): ______________________________

Bridge No.: 5 Transverse Seismic Calculations

Description: 3-Span PPC-I Beam with Trapezoidal Pier Columns and Steel Piles at Piers and Abutments
(Skew Simplified to 0 degrees)
(Pile Design Method Similar for Imbsen and LRFD, therefore not shown - See Bridge No. 2)



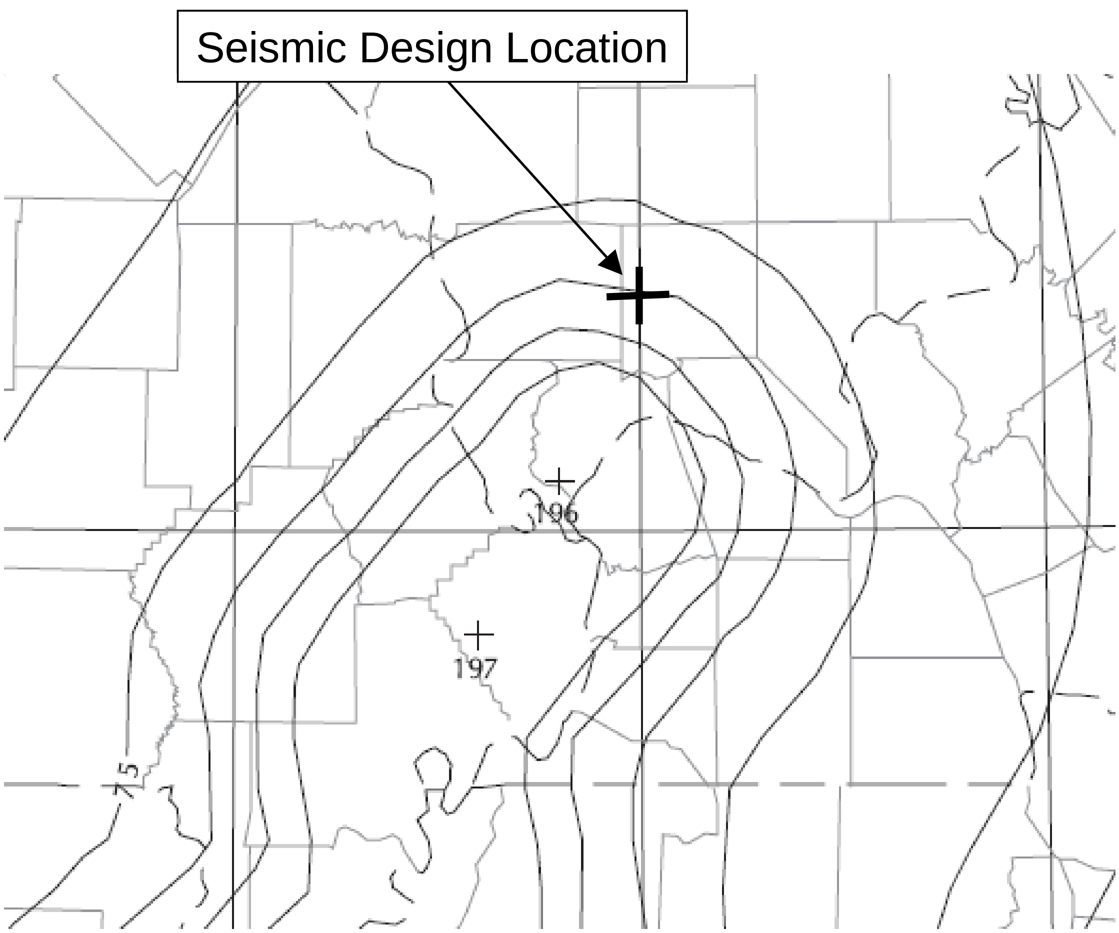
Design Response Specturm



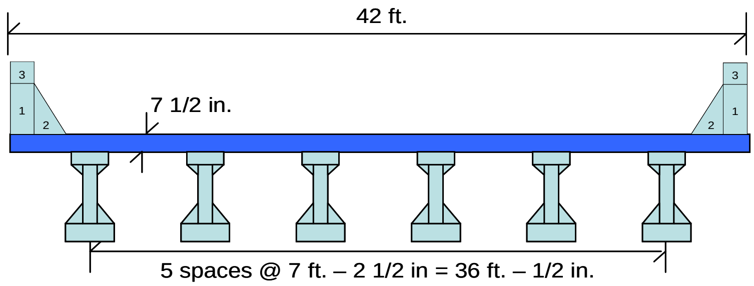
SDC and Other Pertinent Design Spectrum Information

S_{D1} =	0.487	g	Seismic Design Category C		
S_{DS} =	1.128	g	0.3g <=	0.487g	< 0.5 g
End			(Imbsen Table 3.5-1)		
Plateau	0.432	Seconds			

Chosen Location for Bridge Study and 0.2 Second 1000 year Accleration Map (2006 Map)

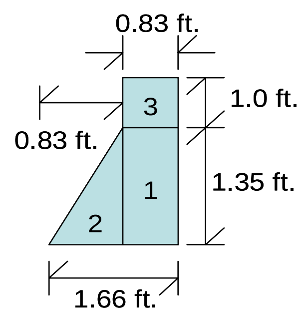


Simple Cross Section of Deck



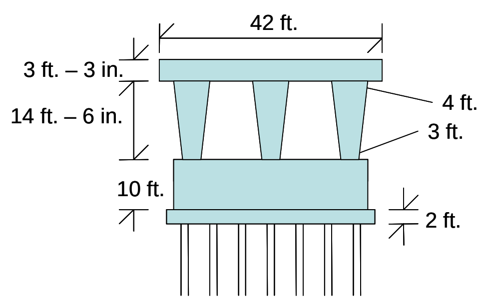
Weight of Super and Sub Structure for Seismic Calculations

Beams	42 in PPC I	
No. Beams	6.00	
Beam Spacing	7.21	ft.
Wt. of 1 Beam	0.48	k/ft.
Wt. Tot. Beams	2.90	k/ft.
Th. of Slab	7.50	in.
Th. of Surface	0.00	in.
Width	42.00	ft.
Wt. of Slab	3.94	k/ft.
Parapet		
Area 1	1.12	ft^2
Area 2	0.56	ft^2
Area 3	0.83	ft^2
Total Area	2.51	ft^2
Wt. of Parapet	0.38	k/ft.



Pier Diaphragms					
No. of Diaphr.	2.00				
Width	1.83	ft.	Height	3.5	ft.
Total Weight	80.85	kips	Length	42	ft.

Steel Parapet Rail		
(est. as	0.00	%
0.02 k/ft)	0.02	k/ft
Cap Beam		
Length	42.00	ft.
Width	2.50	ft.
Height	3.25	ft.
Wt. of 1 Beam	51.19	kips
Wt. of 2 Beams	102.38	kips



Weight of Super and Sub Structure for Seismic Calculations (Cont.)

1/2 of Columns					
Ave. Width	3.75	ft.	Thickness	2.17	ft.
Height	7.25	ft.			
Wt. of 1 Col.	8.85	kips			
No. of Columns	6				
Wt. of 6 Col.	53.10	kips			

Total Wt. for Seismic Calculations		
Super Length	150	ft.
Total Weight	1378	kips

Transverse Period Calculation

Pier Stiffness Transverse Direction

$$k_c = \frac{12 \times E_c \times I_c}{h_c^3} \qquad E_c = \frac{57000\sqrt{f'_c}}{1000}$$

Assume Columns only 3 ft. Wide

f'_c	3500	psi			
E_c	3372	ksi			
Width*	3.00	ft	Thickness	2.17	ft.
I_c	101244	in^4			
$I_c/2$	50622	in^4 Half Cracked Section			
No. columns	3				
$3 \times I_c/2$	151865.28	in^4 Half Cracked Section			
h_c	174	inches (clear column height)			
k_c	389	k/in			
k_{pier}	1167	k/in			

*Use short width for Design

I of Super-structure Transverse

$Ec_{Prestressed}$	4031	ksi
f'_c	3000	psi
E_c	3122	ksi
n (mod. Ratio)	1.29	
I_{slab}	80015040	in^4
$Area_{Parapet}$	361.4	in^2
Area 1 Beam	464.5	in^2
$Area_{Conc\ Bm}$	299.9	in^2 (Transformed)

$$Area_{ConcBm} = \frac{n \times Area}{2}$$

Transf. Area with 50% Shear Lag

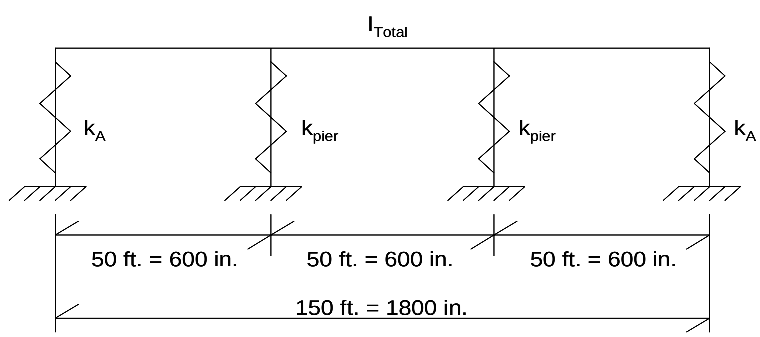
Transverse Period Calculation (Cont.)

Momen of Inertia of Superstructure Table

	No.	I_0 (In^4)	A (in^2)	x bar (in)	A (x bar)^2 (in^4)	I (in^4)
Parapet	2	----	361.44	240	20818944	41637888
Slab	1	80015040	----	----	80015040	80015040
Steel 1	2	----	299.9	43.25	560925.6003	1121851.2
Steel 2	2	----	299.9	129.75	5048330.403	10096660.8
Steel 3	2	----	299.9	216.25	14023140.01	28046280

I_{Total} 1.609E+08 in^4
A_{Total} 6302 in^2

Model the Bridge Transversely with I_{total} of the Superstr. and Springs for the Abutment Piles and Pier Cols.



Estimate the Abutment Pile Transverse Stiffness

k_A 550 k/in

Solve for the Displacement from Simple Model Above as Outlined Below for a 1 k/in Uniform Load

Find the Deflection at the Center of the Bridge Assuming No Piers and Infinitely Stiff Abutments

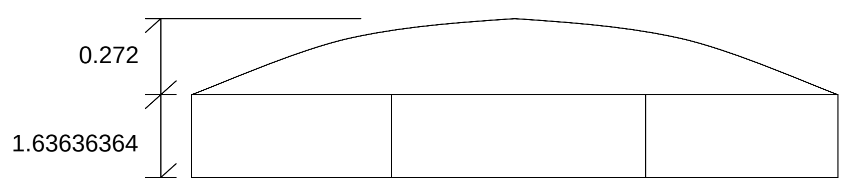
w	1	k/in
L	1800	in
E_c	3122	ksi
I_{Total}	1.609E+08	in^4
δ_c	0.272	in

$$\delta_C = \frac{5 \times w \times L^4}{384 \times E_c \times I_{Total}}$$

Find the Deflection Along the Bridge Assuming an Infinitely Stiff Superstr., No Piers, and Abut. Springs

w	1	k/in
L	1800	in
k_A	550	k/in
δ_e	1.63636364	in

$$\delta_e = \frac{\frac{w \times L}{2}}{k_A}$$

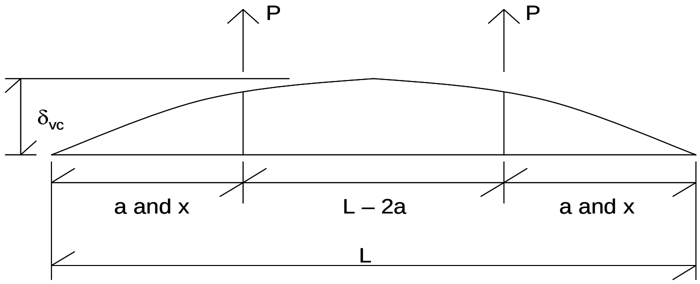


Transverse Period Calculation (Cont.)

Find the Total Estimated Displacement Without the Piers

$\delta_T = \delta_c + \delta_e$	1.908	in

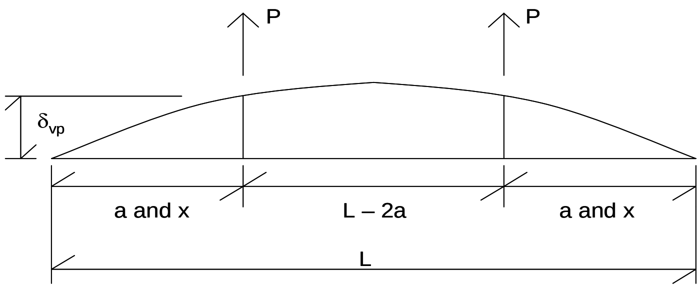
Find the Estimated Deflection at the Center of the Bridge for a Two Point Load at Piers with Infinitely Stiff Abuts.
In Terms of an Applied Load "P".



L	1800	in
x	600	in
a	600	in
E_c	3122	ksi
I_{Total}	1.609E+08	in^4
δ_{vc}	0.0004120	P

$$\delta_{vc} = \frac{P \times a}{24 \times E_c \times I_{Total}} \left(3 \times L^2 - 4 \times a^2\right)$$

Find the Estimated Deflection at the Pier of the Bridge for a Two Point Load at Piers with Infinitely Stiff Abuts.
In Terms of an Applied Load "P".

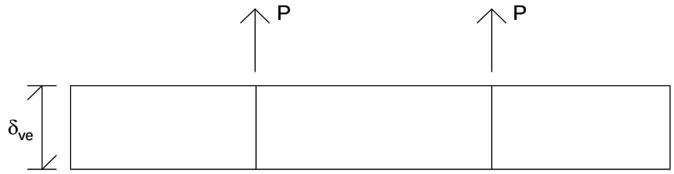


L	1800	in
x	600	in
a	600	in
E_c	3122	ksi
I_{Total}	1.609E+08	in^4
δ_{vp}	0.0003583	P

$$\delta_{vp} = \frac{P \times x}{6 \times E_c \times I_{Total}} \left(3 \times L \times a - 3 \times a^2 - x^2\right)$$

Transverse Period Calculation (Cont.)

Find the Estimated Uniform Deflection for a Two Point Load at Piers with Springs at Abuts.
In Terms of an Applied Load "P".



k_A	550	k/in
δ_{ve}	0.00181818	P

$$\delta_{ve} = P / k_A$$

Find the Fraction of the Estimated Pier Deflection at the Piers Versus that at Center Span

δ_{vc}	0.0004120	P
δ_{vp}	0.0003583	P
δ_{ve}	0.00181818	P
fr	0.976	

$$fr = \frac{\delta_{ve} + \delta_{vp}}{\delta_{ve} + \delta_{vc}}$$

Find the Pier Reactions (V_0) in Terms of δ_{max}, the Actual Estimated Deflection of the Bridge

fr	0.976	
k_{pier}	1167	k/in
V_0	1138.4	δ_{max}

$$V_0 = fr \times \delta_{max} \times k_{pier}$$

Solve for δ_{max}:

$\delta_{ve} + \delta_{vc}$ = 0.002230 P

Set:

$P = V_0$ = 1138.4 δ_{max}

Therefore:

$\delta_{ve} + \delta_{vc}$ = 0.002230 x 1138.4 x δ_{max}

$\delta_{ve} + \delta_{vc}$ = 2.538947 δ_{max}

And:

The Actual Estimated Delfection of the Bridge is the Deflection Without Piers Minus the Contribution with the Piers

δ_{max} = δ_T - 2.538947 δ_{max}

δ_{max} = 1.908 / 3.538947

δ_{max} = 0.539 in

Transverse Period Calculation (Cont.)

Solve for the "Equivalent Stiffness" of the Bridge in the Transverse Direction

w	1	k/in
L	1800	in
δ_{max}	0.539	in
k_{Bridge}	3338	k/in

$$k_{Bridge} = \frac{w \times L}{\delta_{max}}$$

Solve for the Period T

Tot. Weight (W)	1378	kips
g	386.4	in/sec^2
k_{Bridge}	3337.861	k/in
T	0.21	seconds

$$T = 2\pi\sqrt{\frac{W}{g \times k_{Bridge}}}$$

Transverse Seismic Force On Superstructure (Base Shear)

0.21 < 0.432 seconds

Therefore: 112.8% of the Mass is "Effective" and the Total Seismic Load in the Transverse Direction is:

1.128 x 1378 = 1555 kips (Base Shear)

or:

1555 / 1800 = 0.86 k/in (Base Shear)

Transverse Seismic Force on Pier (Base Shear)

$V_{Base\ Shear\ P}$ = 0.86 / 1 x 0.539 x 1138.4

$\boxed{V_{Base\ Shear\ P} = 530 \text{ kips}}$

Transverse Seismic Force on Abutments (Base Shear)

$V_{Base\ Shear\ A}$ = 1555 / 2 - 530

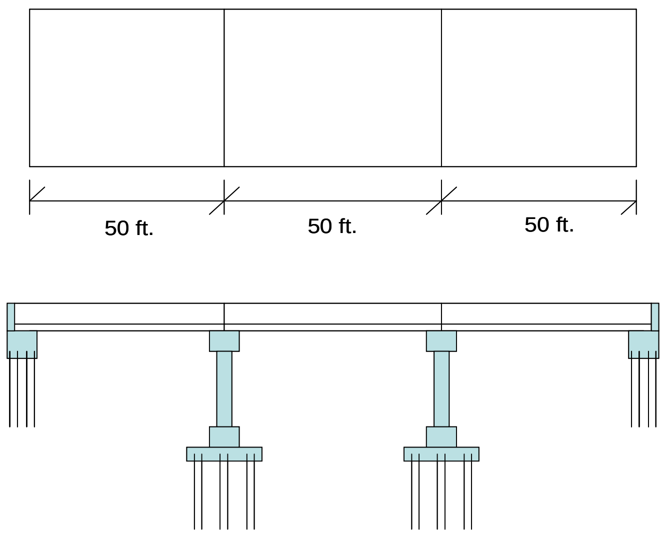
$\boxed{V_{Base\ Shear\ A} = 247 \text{ kips}}$

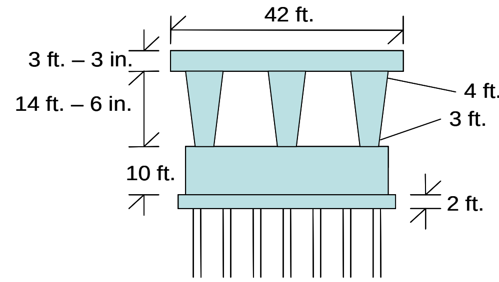
Transverse Seismic Displacement of Pier

$\boxed{\delta_{PierT} = 530 / 1167 = 0.45 \text{ in.}}$

Bridge No.: 5 Longitudinal Seismic Calculations

Description: 3-Span PPC-I Beam with Trapezoidal Pier Columns and Steel Piles at Piers and Abutments (Skew Simplified to 0 degrees)





Weight of Superstructure

Total Weight	1378	kips

Longitudinal Period Calculation

Pier Stiffness Longitudinal Direction

$$k_c = \frac{3 \times E_c \times I_c}{h_c^3} \qquad E_c = \frac{57000\sqrt{f_c'}}{1000} \qquad \theta_{TC} = \frac{P \times h_c^2}{2 \times E_c \times I_c}$$

Contribution from Column

f'_c	3500	psi		
E_c	3372	ksi		
Width*	3.00	ft	Thickness	2.17 ft.
I_c	52972	in^4		
$I_c/2$	26486	in^4 Half Cracked section		
No. columns	3			
3 x $I_c/2$	79457.6019	in^4 Half Cracked section		
h_c	174	inches (clear column height)		
k_c	51	k/in		
k_{pier}	153	k/in		

Contribution from Cap Beam (Stiffness is infinite but it deflects as a rigid body and contributes to pier stiffness)

Find the estimated deflection at the top of column for a load "P"

δ_{TC} = P / 153 in

Find the estimated rotation at the top of column for a load "P"

θ_{TC} = P / 17700.1 radians

Cap height 39 in

Find the added estimated deflection at the top of the pier

δ_A = Cap hght x θ_{TC} = P / 453.8 in

Find the total estimated delfection at the top of the pier

δ_{TD} = δ_A + δ_{TC} = P / 114.2 in

So, the stiffness of a pier is:

k_{pier} = 114.2 k/in

Find the Mass of the Superstructure

M = 1378 / 386.4 = 3.57 k-sec^2/in

Find the period T:

T = 0.79 sec.

$$T = 2\pi\sqrt{\frac{M}{2 \times k_{pier}}}$$

<u>Longitudinal Seismic Force On Superstructure (Base Shear)</u>

0.79 > 0.432 seconds

Therefore: 62% of the Mass is "Effective" and the Total Seismic Load in the Longitudinal Direction is:

0.62 x 1378 = 855 kips (Base Shear)

<u>Longitudinal Seismic Force On Each Pier assuming the abutments don't contribute (Base Shear)</u>

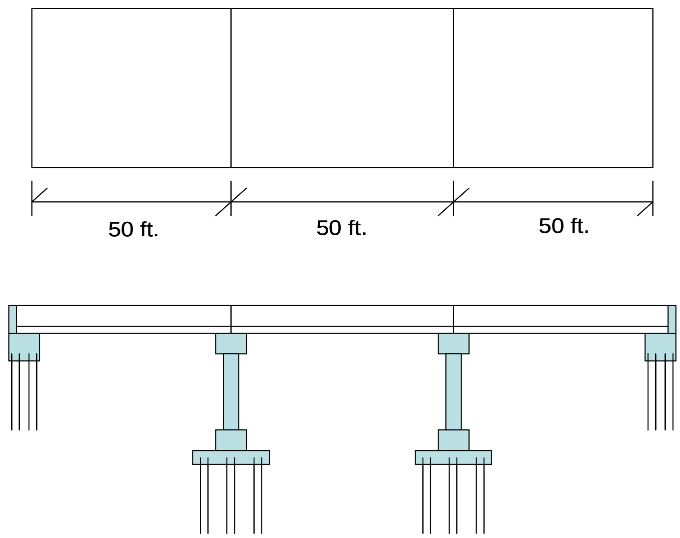
855	/	2	=	427 kips (Base Shear)

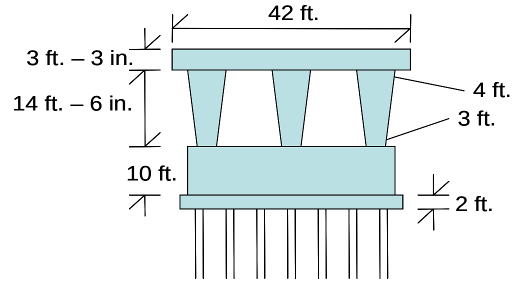
<u>Longitudinal Seismic Displacement of Pier</u>

δ_{PierL} =	427	/	114.2	=	3.74 in.

Bridge No.: 5 Pier Design Forces

Description: 3-Span PPC-I Beam with Trapezoidal Pier Columns and Steel Piles at Piers and Abutments
(Skew Simplified to 0 degrees)





Pier Forces

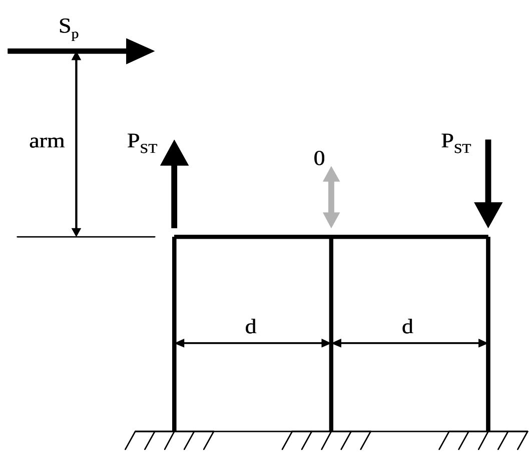
Dead

Dead Load Total	1378	kips
Bridge Length	150	kips
Dead Load per ft.	9.19	k/ft
Dead Load per pier	516.9	kips
No. of Columns	3	
Dead Ld. Per Col.	172.3	kips
Plus 1/2 1 Col.	8.85	kips
Design Dead	181.2	kips

$$\text{DL per pier} = w \times \left(\frac{5}{8} L_{\text{OuterSpan}} + \frac{1}{2} L_{\text{CenterSpan}} \right)$$

$$w = \text{Dead Load per ft.}$$

Transverse Overturning



$$M = S_p \times \text{arm}$$

$$M = P_{ST} \times 2d$$

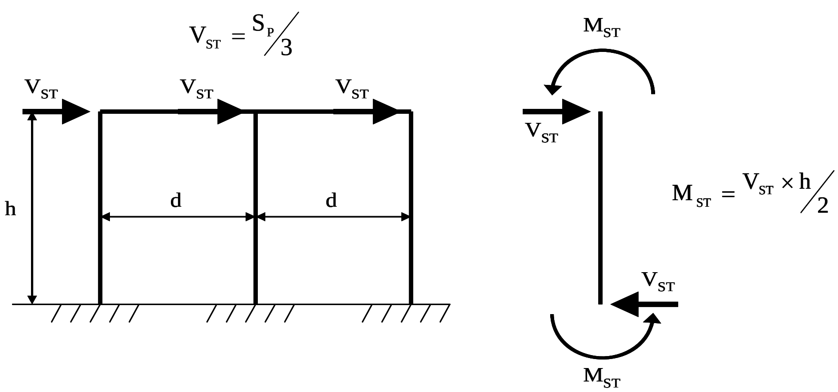
$$\therefore P_{ST} = M/2d$$

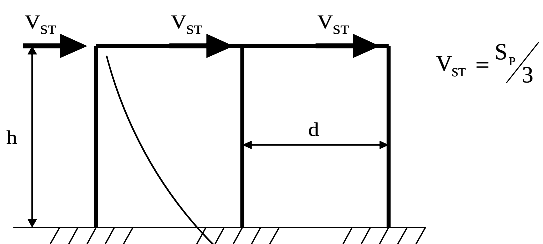
Transverse Overturning (Cont.)

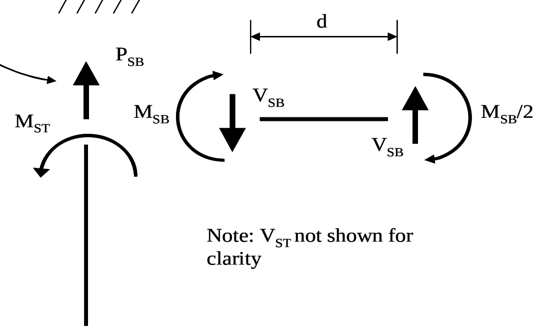
S_P (Pier Base Shear)	530	kips
arm	7.5	ft.
d	12.00	ft.
M	3977.5	k-ft.
P_{ST}	165.7	kips

Frame Action Transverse

S_P (Pier Base Shear)	530	kips
No. of Columns	3	
Column Height (h)	14.50	ft
V_{ST} (Shear per col)	176.8	kips
M_{ST} (Mom. per col)	1281.7	k-ft







$M_{SB} = M_{ST}$

$P_{SB} = V_{SB}$

$$P_{SB} = \frac{M_{ST} + M_{ST}/2}{d} = \pm 1.5 M_{ST}/d$$

P_{SB} = Tension left col., zero center, & Comp. right col.

For Design Use $P_{SB(Design)} = 1.25 \times P_{SB}$

d	12.00	ft.
M_{ST} (Mom. per col)	1281.7	k-ft
P_{SB}	200.3	kips

Longitudinal Shear and Moments (Simple Cantilever Statics)

S_L (Pier Base Shear)	427	kips
No. of Col.	3	
Cap arm	3.25	ft.
Col arm (h)	14.50	ft.
V_{SL}	142.5	kips
$M_{ColTop\ (SLT)}$	463.1	k-ft.
$M_{ColBot\ (SLB)}$	2529.0	k-ft.

P-Δ Moment Amplification for Column Design

Assume $\delta =$ 1.05 for all cases

R-Factor

Assume R = 3.5 for Transverse and Longitudinal Moments (Avg. of Critical and Essential)

Design Axial Forces and Moments For Columns (Orthogonally Combined)

Axial Force Combinations - Transverse Direction Dominant

$$P_{DesignT} = P_{DeadMax} \pm P_{ST} \pm P_{SB}$$

Axial Force Combinations - Longitudinal Direction Dominant

$$P_{DesignL} = P_{DeadMax} \pm 0.3P_{ST} \pm 0.3P_{SB}$$

Note : P_{ST} and P_{SB} can either positive negative or zero depending on the column

P – Δ Amplified and R - Factor Reduced Moment Combinations - Transverse Dir. Dom.

$$M_{DesignT} = \delta \times M_{ST} / R_{Transverse}$$

$$M_{DesignL} = 0.3 \times \delta \times M_{SLB} / R_{Longitudinal}$$

P – Δ Amplified and R - Factor Reduced Moment Combinations - Longitudinal Dir. Dom.

$$M_{DesignT} = 0.3 \times \delta \times M_{ST} / R_{Transverse}$$

$$M_{DesignL} = \delta \times M_{SLB} / R_{Longitudinal}$$

/ ' `Longitudinal

Design Axial Forces and Moments For Columns (Orthogonally Combined) (Cont.)

Transverse Dominant

Column	1	2	3	
$P_{DesignT}$	-184.8	181.2	547.2	kips
$M_{DesignT}$	384.5	384.5	384.5	k-ft.
$M_{DesignL}$	227.6	227.6	227.6	k-ft.
$M_{Combined}$	446.8	446.8	446.8	k-ft.

$\lambda =$ $\tan^{-1}(M_T/M_L)$ = 59.4 degrees

$\theta \cong$ = 40.5 degrees

Longitudinal Dominant (Governs the Design)

Column	1	2	3	
$P_{DesignL}$	71.4	181.2	291.0	kips
$M_{DesignT}$	115.3	115.3	115.3	k-ft.
$M_{DesignL}$	758.7	758.7	758.7	k-ft.
$M_{Combined}$	767.4	767.4	767.4	k-ft.

$\lambda =$ $\tan^{-1}(M_T/M_L)$ = 8.6 degrees

$\theta \cong$ = 5.5 degrees

Elastic (Not combined Orthogonal Shears)

Transverse

V_{ST} 176.8 kips

Longitudinal

V_{SL} 142.5 kips

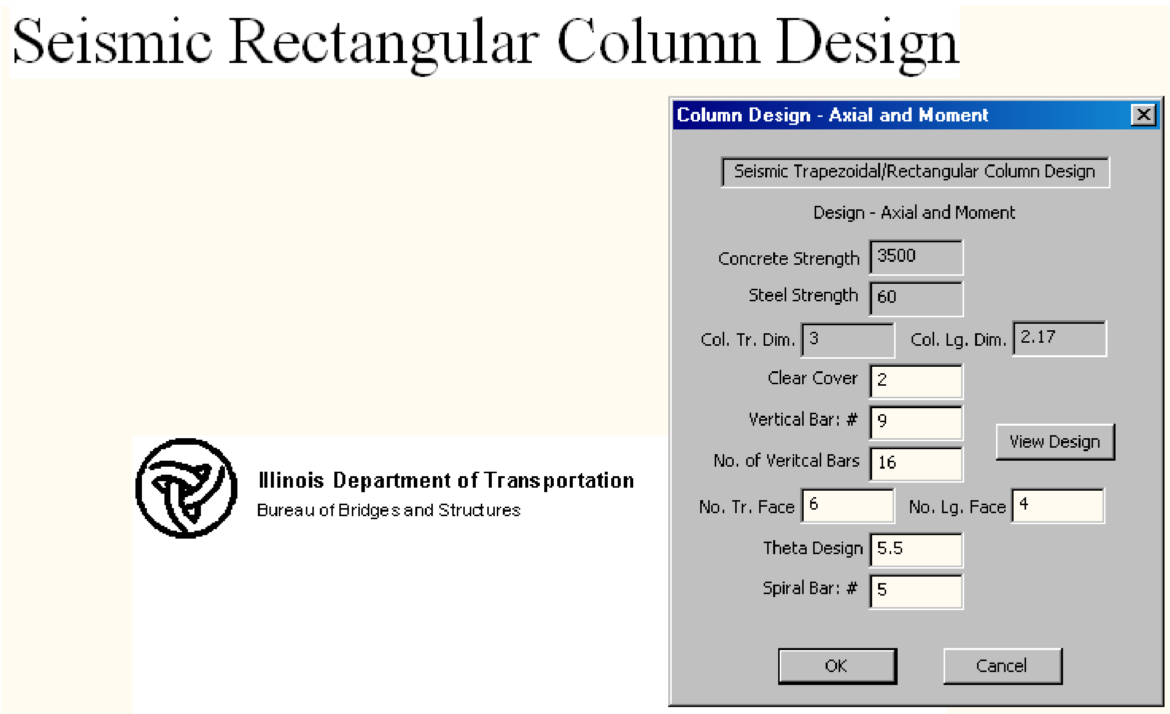
Bridge No.: 5 Force Based Pier Vertical Reinf Design and Displ Check

Description: 3-Span PPC-I Beam with Trapezoidal Pier Columns and Steel Piles at Piers and Abutments (Skew Simplified to 0 degrees) (ϕ = 1.0 for Design)

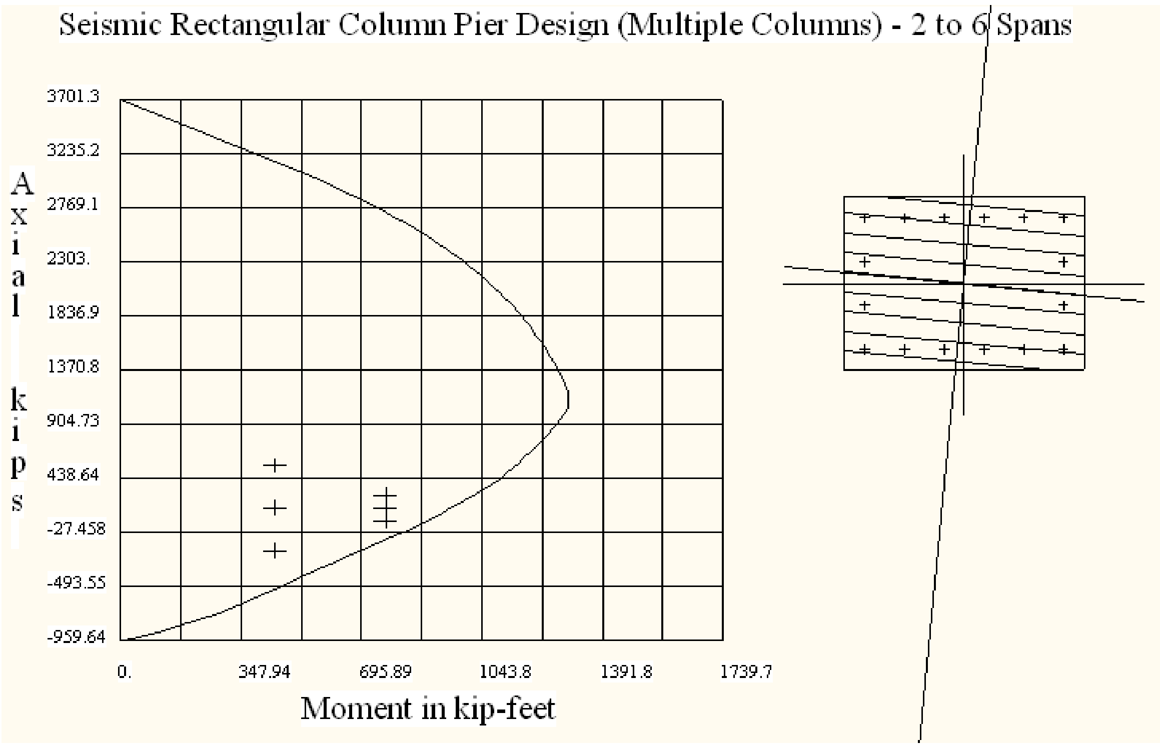
Col. Height	14.5	ft.
Assumed Columns are		
"Half Cracked" for Design	0.5	Ic

Ast	16.0	in^2
Ag	936.0	in^2
Ast/Ag	1.7	%

Computer Program Design Dialog Box (Longitudinal Dominant Load Case - Governs the Design)

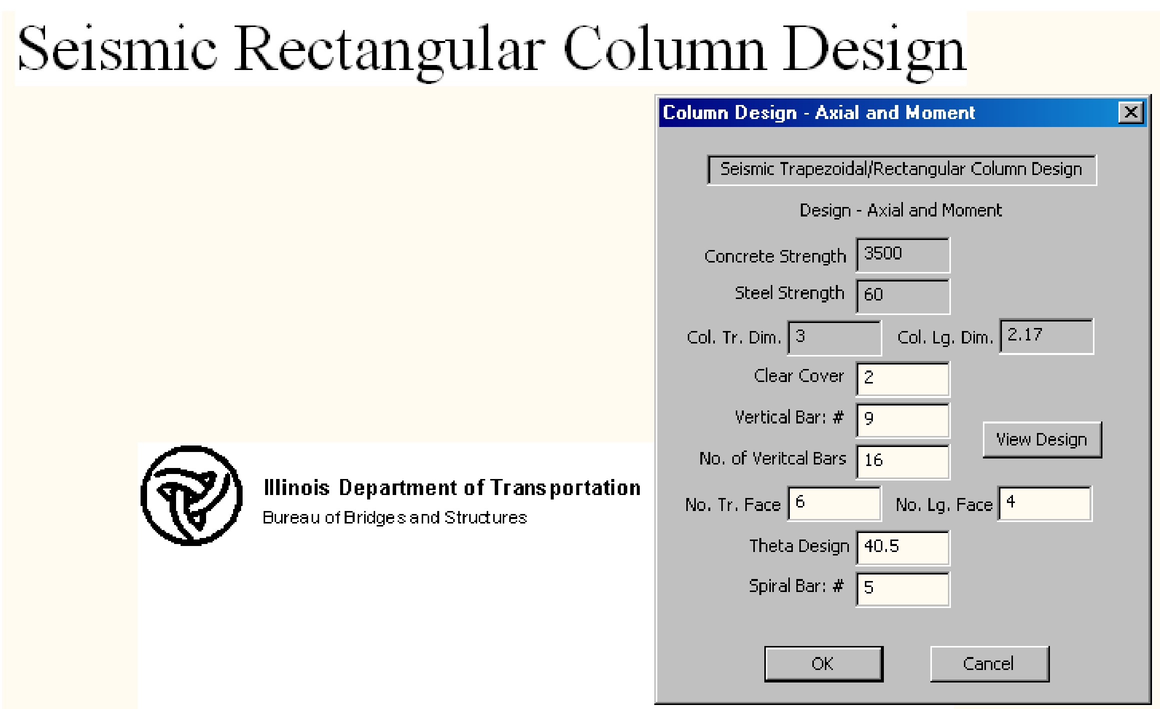


Computer Program Column Design Envelope - Longitudinal Dominant

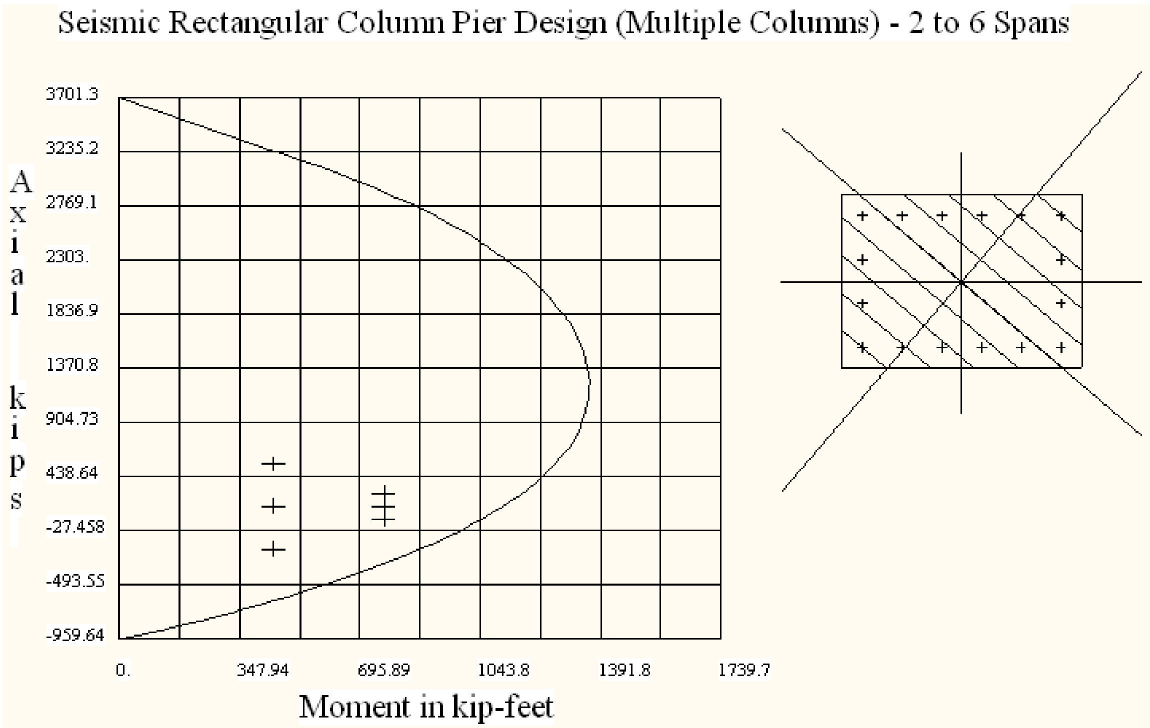


Col. Height	14.5 ft.		Ast	16.0 in²
Assumed Columns are			Ag	936.0 in²
"Half Cracked" for Desigı	0.5 Ic		Ast/Ag	1.7 %

Computer Program Design Dialog Box (Transverse Dominant Load Case)



Computer Program Column Design Envelope - Transverse Dominant



Displacement Check

Scratch Calculation Table (Transverse)

Imbsen Section 4.8					
Column Height (ft)	Column Width (ft)	H/100 (in)	x Fixed-Fixed	Delta Calc. Fixed - Fixed (in)	Delta Allow. Fixed - Fixed (in)
14.5	3.00	1.74	0.41	1.44	1.74

Scratch Calculation Table (Longitudinal)

Imbsen Section 4.8					
Column Height (ft)	Column Width (ft)	H/100 (in)	x Fixed-Pinned	Delta Calc. Fixed - Pinned (in)	Delta Allow. Fixed - Pinned (in)
14.5	2.17	1.74	0.15	5.54	5.54

Transverse Direction

	Imbsen Fig. 5.4				Imbsen Sec. 4.3.3		Uncombined	Req. Seismic
Column Height (ft.)	Steel Ratio (Ast/Ag)	Fraction of Ig	Trans. Period (Sec.)	Trans. Deflection (in)	Short Period Ampl.	Trans Des. Defl. (in)	Trans Allow. Des. Defl. (in)	Ast/Ag Force Based Design
14.5	≅ 0.02	0.5	0.21	0.45	2.08	0.94	1.74	0.017

Longitudinal Direction

	Imbsen Fig. 5.4				Uncombined	Req. Seismic
Column Height (ft.)	Steel Ratio (Ast/Ag)	Fraction of Ig	Long. Period (Sec.)	Long. Deflection (in)	Longitudinal Allowable Deflection (in)	Ast/Ag Force Based Design
14.5	≅ 0.02	0.4	0.79	3.74	5.54	0.017

An Orthogonally Combined Delfection Check Will Be OK

Bridge No.: 5 Pier Shear Reinforcement Design
Descriptior 3-Span PPC-I Beam with Trapezoidal Pier Columns and Steel Piles at Piers and Abutments
(Skew Simplified to 0 degrees)

Pier Column Shear Reinforcement Design

For simplicity, use the elastic seismic design forces
Perform basic desgin in plastic hinging region only (reinf. for confinement)
For all columns, take the shear strength of the concrete as zero (0)
ϕ = 1.0 for Design Initially

Equations and Methods (LRFD and Imbsen - Both are Similar for Simple Design)

LRFD 5.10.11.4.1e Max. Spacing of Ties = 4 in.

LRFD 5.10.11.4.1d Minimum Reinforcment #1

$$A_{sh} \geq 0.12 s h_c \frac{f'_c}{f_y}$$

f'c =	3500	psi
fy =	60000	psi
s =	4	in
hc =	32	in
Ash req. =	0.90	in^2

LRFD 5.10.11.4.1d Minimum Reinfocement #2

$$A_{sh} \geq 0.30 s h_c \frac{f'_c}{f_y}\left[\frac{A_g}{A_c} - 1\right]$$

f'c =	3500	psi
fy =	60000	psi
Ag =	936.0	in^2
Ac =	704.0	in^2
s =	4	in
hc =	32	in
Ash req. =	0.74	in^2

Imbsen 8.6.6 Minimum Reinfocement #2

$$\rho_w \geq 0.005 \text{ or } A_{sh} \geq 0.005 s h_c$$

ρ_w min =	0.0050	
Ash req.	0.72	in^2

LRFD 5.8.3.4.1 Simplified Shear Procedure for Non-Prestressed Sections

$$\phi V_s = \phi \frac{A_v f_y d_v}{s}$$

Av =	0.93	in^2	(#5 bars with a cross tie)
fy =	60	ksi	
dv =	18.72	in	(0.72h LRFD 5.8.2.9)
s =	4	in	
ϕVs =	261	kips	

Imbsen 8.6.3 Shear Strength of Steel

$$\phi V_s = \phi \frac{A_v f_{yh} D}{s}$$

Av =	0.93	in^2	(#5 bars with a cross tie)
fyh =	60	ksi	
D =	26	in	(Imbsen means full col dimension?)
s =	4	in	
ϕVs =	363	kips	

Summary of Shear Reinforcement Designs (LRFD and Imbsen)

Using #5 Ties at Max. Spacing of 4 in.

						ϕ = 1.0	ϕ = 0.85	ϕ = 1.0
	Imbsen Fig. 5.4		Trans.	Long.	Ld. Cse 1	Imbsen	Imbsen	LRFD
Column Height (ft.)	Steel Ratio (Ast/Ag)	Fraction of Ig	Elastic Shear Per Col. (kips)	Elastic Shear Per Col. (kips)	Governs (Trans. Dom.) (kips)	Strength #5's at 4 in. (kips)	Strength #5's at 4 in. (kips)	Strength #5's at 4 in. (kips)
14.5	≅ 0.02	0.5	176.8	142.5	181.9	363.0	308.6	261.0

gn Pg. 1

;

gn Pg. 2

gn Pg. 2

$\phi = 0.9$
LRFD Strength #5's at 4 in. (kips)
234.9

Bridge No.: 5 Seat Width Requirements

Descriptior 3-Span PPC-I Beam with Trapezoidal Pier Columns and Steel Piles at Piers and Abutments
(Skew Simplified to 0 degrees)

Seat Width Requirements

Compare Imbsen with NCHRP 12-49 and the Current LRFD Code
LRFD calibrated for 500 years and 12-49 calibrated to 1.0 Sec. Accel. with improved Soil Coef.
so it is "return period independent".

NCHRP 12-49

$$N = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H}\sqrt{1+\left(2\frac{B}{L}\right)^2}\right](1+1.25F_vS_1)$$ (metric)

L =	150	ft	or	45.72	meters
FvS1 =				0.487	g
H =	14.5	ft	or	4.42	meters
B =	42	ft	or	12.8	meters

Imbsen 4.12.2

$$N = (4 + \Delta_{ot} + 1.65\Delta_{eq}) \geq 12$$

Δot = 0.01L = 1.5 inches

Δeq = Long period frame seismic displacement

LRFD 4.7.4.4

$$N = 8 + 0.02L + 0.08H$$

L =	150	ft	%N for Cat. C = 150
H =	14.5	ft	

Summary of Seat Width Requirements (NCHRP 12-49, Imbsen and LRFD)

Column Height (ft.)	Imbsen Fig. 5.4 Steel Ratio (Ast/Ag)	Imbsen Fig. 5.4 Fraction of Ig	Long. Deflection (in)	Imbsen 4.12.2 Calc. Seat (in)	Imbsen 4.12.2 Req. Seat (in)	NCHRP 12-49 Req. Seat (in)	Current LRFD Req. Seat (in)
14.5	≅ 0.02	0.5	3.74	11.7	12.0	22.1	20.3