AASHTO T-3 TRIAL DESIGN BRIDGE DESCRIPTION

State: *Illinois*

Trial Design Designation: *IL-1*

Bridge Name: *Typical Bridge in Illinois*

Superstructure Type: *Simply supported "I" girder with composite concrete deck*

Span Length(s): *56.8 ft. – 67.2 ft. – 56.8 ft. (total 176.3 ft.)*

Substructure Type: *Drop cap supported by 4 circular reinforced concrete columns*

Foundation: *Drilled shafts at bents and abutments*

Abutments: *Cap supported directly on drilled shafts*

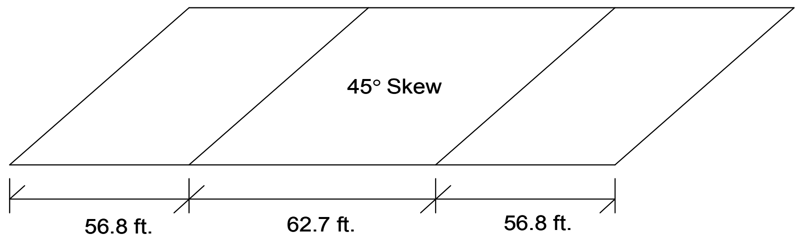
Seismic Design Category (SDC): *"D"*

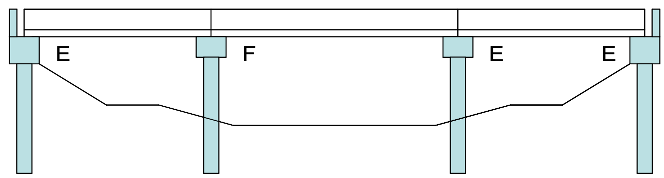
Seismic Design Strategy (Type 1, 2 or 3): *Type 1*

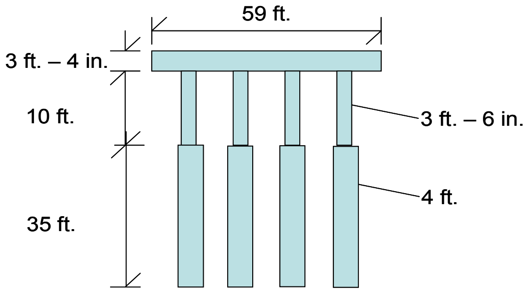
Design Spectral Acceleration at 1-second Period (S_{D1}): *0.849g*

Additional Description (Optional): *Includes comparisons of the Forced Based Approach (NCHRP 12-49) with the Displacement Based Approach (NCHRP 20-07).*

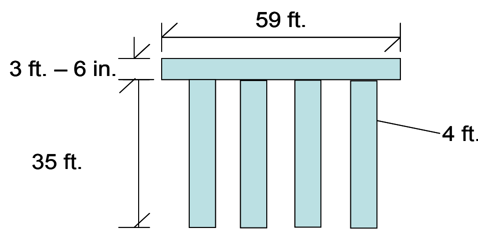
Bridge No.: 1 Pushover and Force Based Comparison
Description: 3-Span Wide Flange with Drilled Shaft Piers and Drilled Shaft Abutments
(Skew not simplfied, Cate D. Design, Site Class D





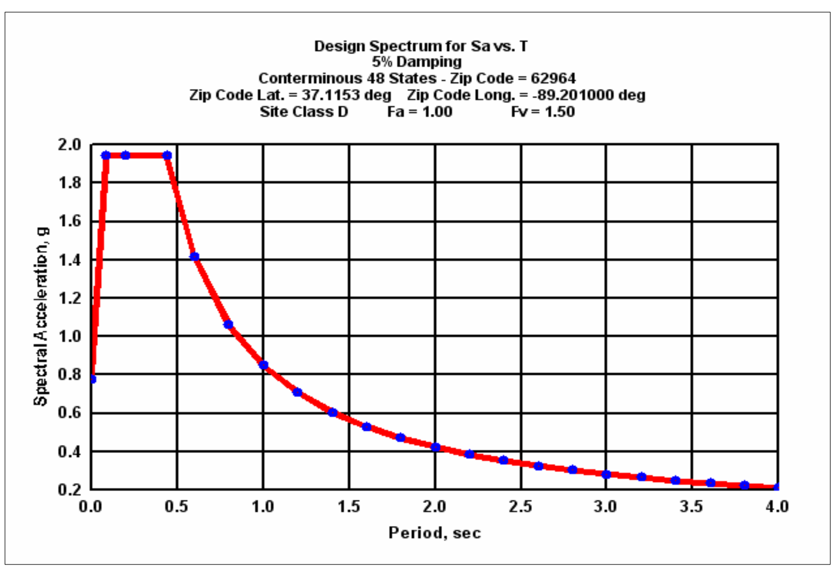


Pier Sketch



Abutment Sketch
(Showing abut. cap and shafts)

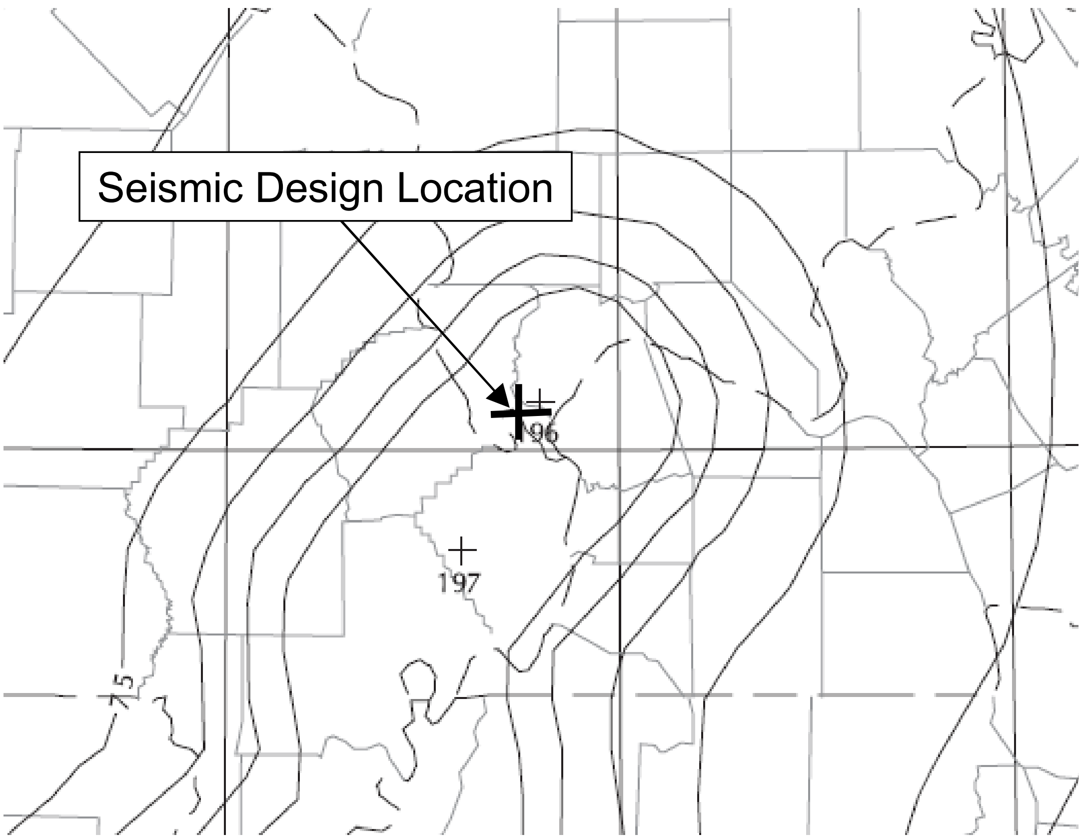
Design Response Specturm



SDC and Other Pertinent Design Spectrum Information

S_{D1} =	0.849 g	Seismic Design Category D (Imbsen Table 3.5-1)	
S_{DS} =	1.944 g	0.5g <	0.849g
End Plateau	0.437 Seconds	Seismic Zone 4 (Assumed for LRFD)	

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

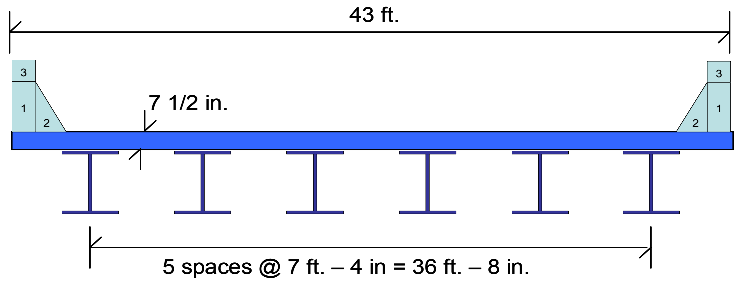


<u>Permissible Analysis Methods</u>: Essential, regular bridge with less than six spans.

Imbsen Table 4.1: Equivalent Static or Multimode Spectral
LRFD Table 4.7.4.3.1-1: Multimode Spectral

It has been IDOT's experience for it's "garden variety" structures that the first mode of vibration is dominant. For such structures, equivalent static methods are preferred over the multimode spectral method for their simplicity and ease of verifying results. The subject structure will be analyzed by computer using the uniform load method and including the effects of soil structure interaction.

<u>Simple Cross Section of Deck</u>



<u>Superstructure Properties and Moment of Inertia:</u>

$E =$	29000	ksi	f'c deck =	3500	psi
$I_x =$	63280	in.^{4}			
$I_y =$	3.70E+06	in.^{4}			
$J =$	24065	in.^{4}			

The superstructure properties have been calculated using the parallel axis theorem and transforming the area of the concrete deck. For simplicity, the stiffness added by the parapets has been neglected in computing the properties.

Substructure Properties and Moment of Inertias:

- ***Pier Columns***

LRFD / Force Based Approach

Diameter =	42	in.	
f'_c =	3.5	ksi	
EI_{eff} =	$0.5EI_g$	NCHRP 12-49 C5.3.3.2	
EJ_{eff} =	$0.2EJ_g$		
E =	$1820*\sqrt{(f'c)}$	LRFD C5.4.2.4	
E =	3405	ksi	
I_{eff} =	76373	$in.^4$	
J_{eff} =	61098	$in.^4$	
EI_{eff} =	2.60E+08	$k*in.^2$	
EJ_{eff} =	2.08E+08	$k*in.^2$	
A_g =	1385	$in.^2$	

Imbsen / Displacement Based Approach

Diameter =	42	in.	
f'_c =	3.5	ksi	
Vert. Bar Size =	11		*Initial Estimate*
# Vert. Bars =	18	bars	
f'_{ce} =	$1.3*f'_c$	ksi	Imbsen 8.4.4
or	5	ksi	
E =	$1820*\sqrt{(f'_{ce})}$	ksi	LRFD C5.4.2.4
I_{eff}/I_g =	Fig. 5.4		Imbsen 5.6.2
J_{eff} =	$0.2J_g$		
f'_{ce} =	5	ksi	
E =	4070	ksi	
P =	159	kips	*Avg. axial dead load as det. from design.*
A_g =	1385	$in.^2$	
$P/(f'_{ce}*A_g)$ =	0.023		
A_{st} =	28.08	$in.^2$	
A_{st}/A_g =	0.02		
I_{eff}/I_g =	0.425		*Det. from Fig. 5.4*
I_{eff} =	64917	$in.^4$	
EI_{eff} =	2.64E+08	$k*in.^2$	
J_{eff} =	61098	$in.^4$	
EJ_{eff} =	2.49E+08	$k*in.^2$	

● Pier Shafts

LRFD / Force Based Approach

Diameter =	48	in.	
f'_c =	3.5	ksi	
EI_{eff} =	$0.5EI_g$		NCHRP 12-49 C5.3.3.2
EJ_{eff} =	$0.2EJ_g$		NCHRP 12-49 C5.3.3.2
E =	1820*√(f'c)		LRFD C5.4.2.4
E =	3405	ksi	
I_{eff} =	130288	$in.^4$	
J_{eff} =	104230	$in.^4$	
EI_{eff} =	4.44E+08	$k*in.^2$	
EJ_{eff} =	3.55E+08	$k*in.^2$	
A_g =	1810	$in.^2$	

Imbsen / Displacement Based Approach

Diameter =	48	in.	
f'_c =	3.5	ksi	
Vert. Bar Size =	11		*Initial Estimate*
# Vert. Bars =	18	bars	*Initial Estimate*
f'_{ce} =	$1.3*f'_c$	ksi	Imbsen 8.4.4
or	5	ksi	Imbsen 8.4.4
E =	$1820*\sqrt{(f'_{ce})}$	ksi	LRFD C5.4.2.4
I_{eff}/I_g =	Fig. 5.4		Imbsen 5.6.2
J_{eff} =	$0.2J_g$		
f'_{ce} =	5	ksi	
E =	4070	ksi	
P =	159	kips	*Avg. axial dead load as det. from design.*
A_g =	1810	$in.^2$	
$P/(f'_{ce}*A_g)$ =	0.018		
A_{st} =	28.08	$in.^2$	
A_{st}/A_g =	0.016		
I_{eff}/I_g =	0.38		*Det. from Fig. 5.4*
I_{eff} =	99019	$in.^4$	
EI_{eff} =	4.03E+08	$k*in.^2$	
J_{eff} =	104231	$in.^4$	
EJ_{eff} =	4.24E+08	$k*in.^2$	

● Pier Cap

Min. Height =	3.5	ft
Width =	3.5	ft
Area =	12.25	ft^2
$I_x = I_y = J$ =	10,000	ft^4

The section properties of the pier cap have been artficially designated based on past experience to ensure lateral force distribution to the columns and shafts in the elastic computer model.

- ***Abutment Shafts***

LRFD / Force Based Approach

Diameter =	42	in.	
f'_c =	3.5	ksi	
EI_{eff} =	$0.5EI_g$		NCHRP 12-49 C5.3.3.2
EJ_{eff} =	$0.2EJ_g$		
E =	$1820*\sqrt{(f'c)}$		LRFD C5.4.2.4
E =	3405	ksi	
I_{eff} =	76373	$in.^4$	
J_{eff} =	61098	$in.^4$	
EI_{eff} =	2.60E+08	$k*in.^2$	
EJ_{eff} =	2.08E+08	$k*in.^2$	
A_g =	1385	$in.^2$	

Imbsen / Displacement Based Approach

Diameter =	42	in.	
f'_c =	3.5	ksi	
Vert. Bar Size =	11		*Initial Estimate*
# Vert. Bars =	18	bars	
f'_{ce} =	$1.3*f'_c$	ksi	Imbsen 8.4.4
or	5	ksi	
E =	$1820*\sqrt{(f'_{ce})}$	ksi	LRFD C5.4.2.4
I_{eff}/I_g =	Fig. 5.4		Imbsen 5.6.2
J_{eff} =	$0.2J_g$		
f'_{ce} =	5	ksi	
E =	4070	ksi	
P =	145	kips	*Avg. axial dead load as det. from design.*
A_g =	1385	$in.^2$	
$P/(f'_{ce}*A_g)$ =	0.021		
A_{st} =	28.08	$in.^2$	
A_{st}/A_g =	0.02		
I_{eff}/I_g =	0.425		*Det. from Fig. 5.4*
I_{eff} =	64917	$in.^4$	
EI_{eff} =	2.64E+08	$k*in.^2$	
J_{eff} =	61098	$in.^4$	
EJ_{eff} =	2.49E+08	$k*in.^2$	

- ***Abutment Cap***

Min. Height =	5	ft
Width =	3.5	ft
Area =	17.5	ft^2
$I_x = I_y = J$ =	10,000	ft^4

The section properties of the pier cap have been artficially designated based on past experience to ensure lateral force distribution to the columns and shafts in the elastic computer model.

Note that it was anticipated that the two methods contained herein for calculating "effective" section properties would yield substantially different results. This was not the case however and the results from the two methods are largely similar. Therefore, only one elastic analysis will be performed using the results for the "force based" approach..

Passive Soil Resistance at Abutment

● ***Longitudinal Direction*** (NCHRP 12-49 7.5, Imbsen 5.2.3.3)

Backwall Height =	2.25	ft (Approx. clear dist. between bottom of approach slab and top of cap)
Abutment Cap Height =	3.5	ft
Length Along Abutment =	56.5	ft (Approx. clear dist. between the inside of the wing walls)

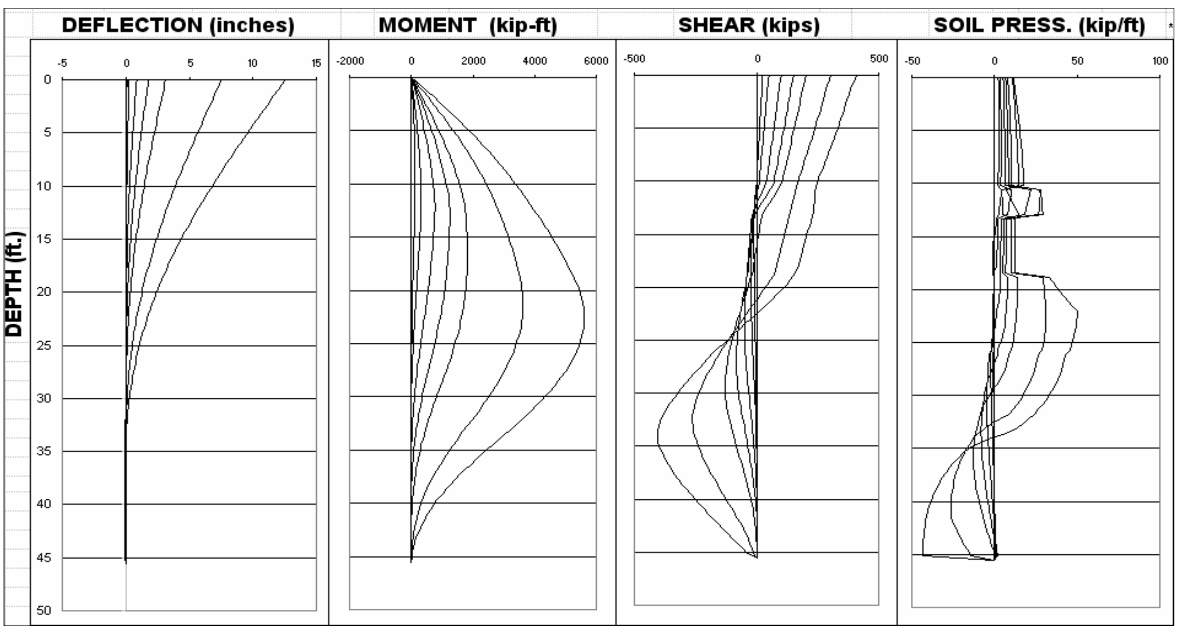
p_p =	2*H/3*L	k/ft	
=	217	k/ft	
P_p =	p_p*H	k	
=	1248	k	
K_{eff1} =	P_p/(0.2*H)	k/ft	<= "*Gap element" will be used in computer model to account for*
	10852	k/ft	*expansion joint.*

● ***Transverse Direction***

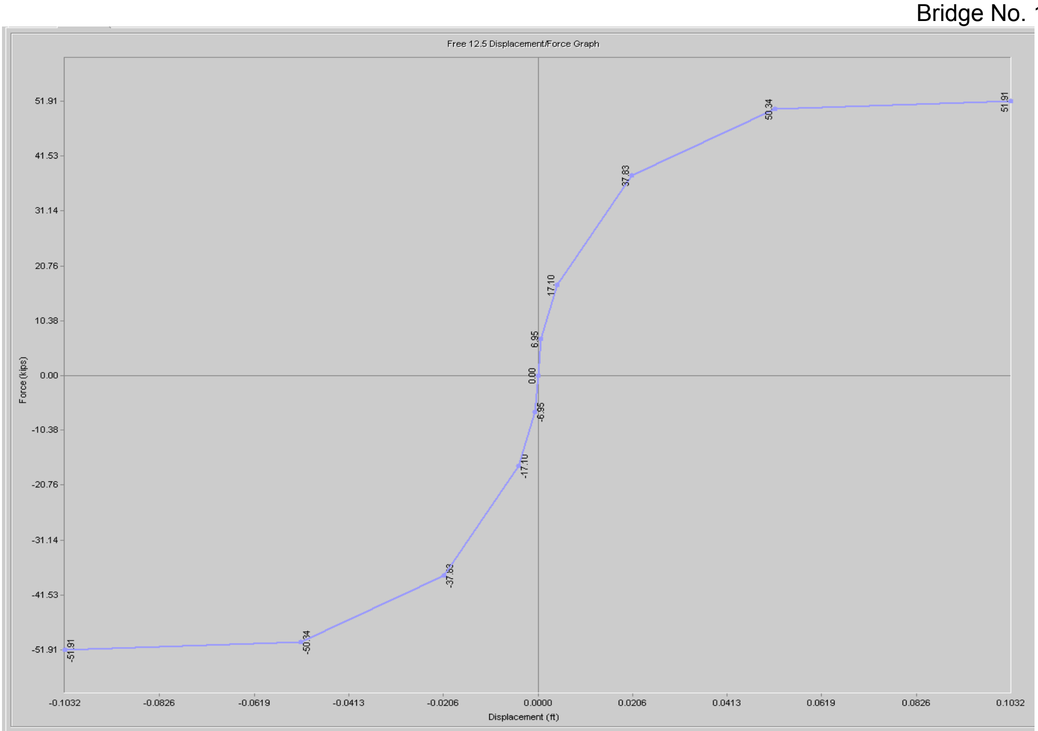
It is assumed for the design that the wingwalls may yield. Therefore, passive resistance of the soil in the transverse direction is neglected and it is assumed that the drilled shafts will provide all of the resistance in the transverse direction.

Soil Structure Interaction for Drilled Shafts

The drilled shafts were analyzed using COM624 and an arbitrary range of lateral loads. A sample output of the COM624 analysis is indicated below. This data has been interpolated to determine a series of nonlinear springs at regular intervals along the length of the drilled shafts. A boring log from an existing stucture containing soil properties consistent with that defined for Site Class D was chosen for the COM624 analysis.

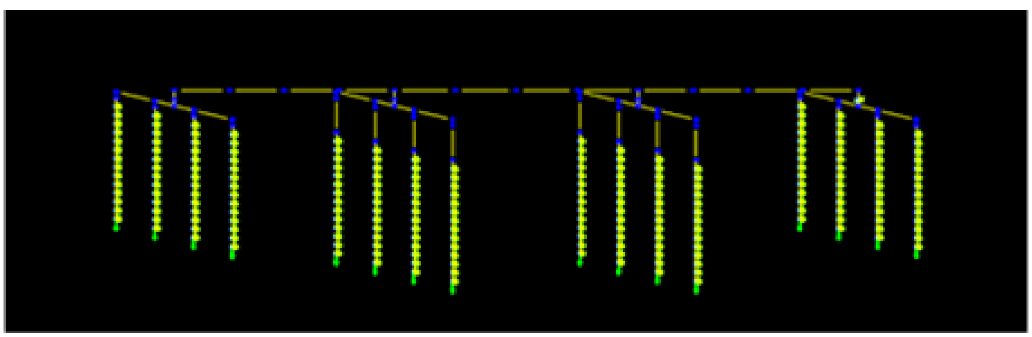


Sample output from COM624 analysis.



Sample soil spring at a given layer used in the analysis (force vs. displacement) as derived from COM624.

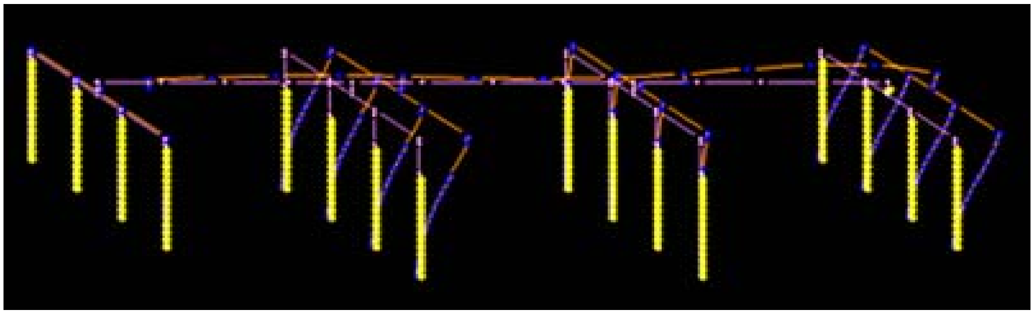
Elastic Seismic Analysis



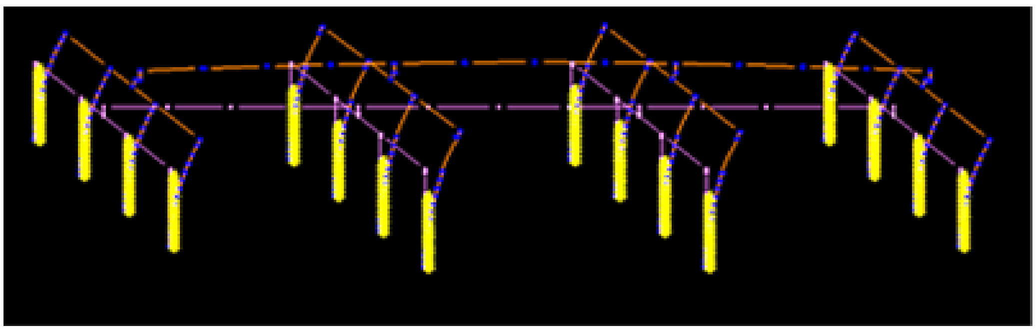
General elevation view of analysis model.

Longitudinal Analysis			Transverse Analysis		
W =	1544	k	W =	1544	k
L =	176.3	ft	L =	176.3	ft
Estimated C_s =	1.66		Estimated C_s =	1.42	
P_o =	14.5	k/ft	P_o =	12.40	k/ft
Δ_L =	4.4		Δ_T =	5.0	
K =	583	k/in.	K =	438	k/in.
T =	0.52	sec.	T =	0.6	sec.
Actual Cs =	1.63	**O.K., w/in 10% Est. Cs**	Actual Cs =	1.42	**O.K., w/in 10% Est. Cs**

Note W includes the dead load weight of the entire superstructure and one half of the weight of the pier components above the drilled shafts.
Deflections shown are relative to the longitudinal centerline of the superstructure.



Deflected shape of structure for longitudinal seismic analysis.



Deflected shape of structure for transverse seismic analysis.

- ***Resultant Forces / Displacements***

		Elastic Seismic Forces										
		Longitudinal EQ					Transverse EQ					Axial DL
		Long. Shear (kips)	Long. Moment (kip*ft)	Trans. Shear (kips)	Trans. Moment (kip*ft)	Axial (kips)	Long. Shear (kips)	Long. Moment (kip*ft)	Trans. Shear (kips)	Trans. Moment (kip*ft)	Axial (kips)	(kips)
Pier 1	Top Col	80	186	155	2174	+/-250	86	622	80	980	+/-125	160
	Bot. Col	80	830	155	621	+/-250	86	1360	80	180	+/-125	160
	Shaft	80	1348	155	868	+/-250	86	1862	80	530	+/-125	160
Abut. 2	Shaft	140	106	232	2427	+/-280	112	691	113	945	+/-140	140

		Orthogonal Seismic Force Combinations										
		100% Longitudinal EQ + 30% Transverse EQ					30% Longitudinal EQ + 100% Transverse EQ					Axial DL
		Long. Shear (kips)	Long. Moment (kip*ft)	Trans. Shear (kips)	Trans. Moment (kip*ft)	Axial (kips)	Long. Shear (kips)	Long. Moment (kip*ft)	Trans. Shear (kips)	Trans. Moment (kip*ft)	Axial (kips)	(kips)
Pier 1	Top Col	106	373	179	2468	+/-288	110	678	127	1632	+/-200	160
	Bot. Col	106	1238	179	675	+/-288	110	1609	127	366	+/-200	160
	Shaft	106	1907	179	1027	+/-288	110	2266	127	790	+/-200	160
Abut. 2	Shaft	174	313	266	2711	+/-322	154	723	183	1673	+/-224	140

		Modified Seismic Design Forces (R_{column}=3.5, R_{shaft}=1.0)										
		100% Longitudinal EQ + 30% Transverse EQ					30% Longitudinal EQ + 100% Transverse EQ					Axial DL
		Long. Shear	Long. Moment	Trans. Shear	Trans. Moment	Axial	Long. Shear	Long. Moment	Trans. Shear	Trans. Moment	Axial	
		(kips)	(kip*ft)	(kips)	(kip*ft)	(kips)	(kips)	(kip*ft)	(kips)	(kip*ft)	(kips)	(kips)
Pier 1	Top Col	106	106	179	705	+/-288	110	194	127	466	+/-200	160
	Bot. Col	106	354	179	193	+/-288	110	460	127	105	+/-200	160
	Shaft	106	1907	179	1027	+/-288	110	2266	127	790	+/-200	160
Abut. 2	Shaft	174	313	266	2711	+/-322	154	723	183	1673	+/-224	140

		SRSS Combined Seismic Design Forces (R_{column}=3.5, R_{shaft}=1.0)					
		100% Long. EQ + 30% Trans. EQ + DL			30% Long. EQ + 100% Trans. EQ + DL		
		Shear	Moment	Axial Load	Shear	Moment	Axial Load
		(kips)	(kip*ft)	(kips)	(kip*ft)	(kips)	(kips)
Pier 1	Top Col	208	713	+448/-128	168	505	+360/-40
	Bot. Col	208	403	+448/-128	168	471	+360/-40
	Shaft	208	2166	+448/-128	168	2400	+360/-40
Abut. 2	Shaft	318	2729	+461/-182	239	1823	+364/-84

	Elastic Seismic Displacement Demand			
	Longitudinal EQ		Transverse EQ	
	Longitudinal	Transverse	Longitudinal	Transverse
	(in.)	(in.)	(in.)	(in.)
Pier 1	3.44	2.67	5.5	0.92
Abut. 2	3.52	2.08	4.48	0.54

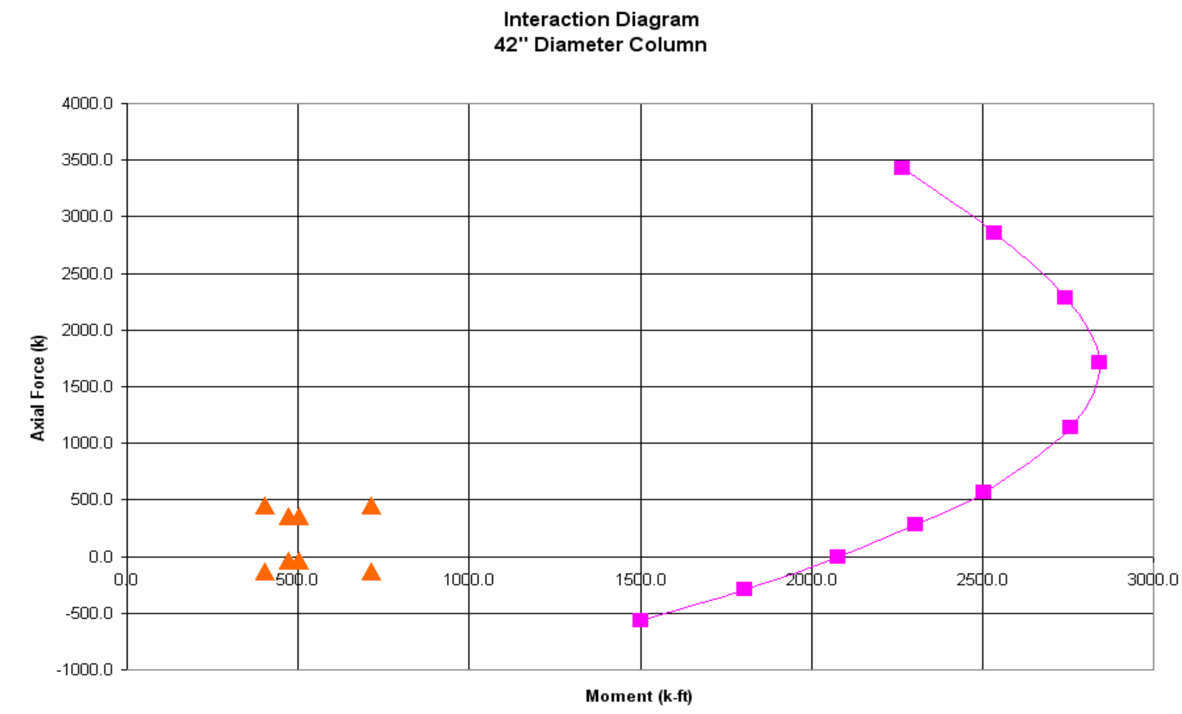
	Orthogonal Elastic Seismic Displacement Demand Combination			
	100% Long. EQ + 30% Trans. EQ		30% Long. EQ + 100% Trans. EQ	
	Longitudinal	Transverse	Longitudinal	Transverse
	(in.)	(in.)	(in.)	(in.)
Pier 1	5.09	2.95	6.53	1.72
Abut. 2	4.86	2.24	5.54	1.16

The results tabulated above have been extracted from the analysis data to represent the critical force combinations on the subject sections. Contrary to the other examples, the above results have not been magnified for P-delta effects as it is concluded that such secondary effects are typically insignficant when compared to the variability in magnitude and ground motion that can be expected to occur with an extreme seismic event for Category D.

<u>Pier Column Design</u> (Force Based)

- ***Flexure***

Column Type:	Circular	
Diameter =	42	in.
f'_c =	3.5	ksi
f_y =	60	ksi
Vert. Bar Size =	11	
# Vert. Bars =	18	
Clear Cover =	2	in.
Assumed Lat. Rein. Size =	5	
$\phi_{flexure}$ =	1.0	NCHRP 12-49 7.8.2.2



Data points for interaction diagram generated using WinRECOL by IMBSEN Software Systems.

Flexure okay by inspection of interaction diagram.

- ***Shear***

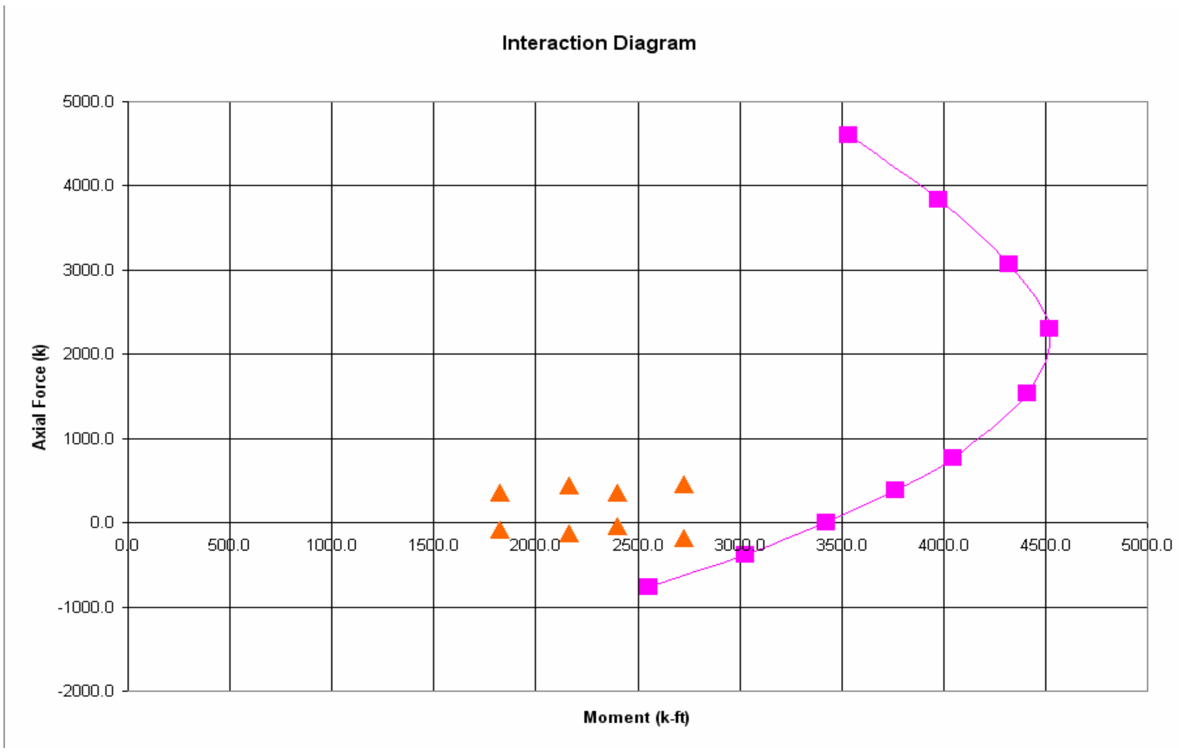
$\rho_{s\ min}$ =	0.007		LRFD 5.10.11.4.1c
Max. allowable pitch =	4	in.	LRFD 5.10.11.4.1e
Trans. Bar Size =	5		
Actual Pitch =	4	in.	
$\rho_{s\ prov}$ =	0.007	O.K.	
ϕ_{shear} =	0.9		LRFD 5.5.4.2.1
V_u =	$\phi(V_c + V_s)$		
V_c =	0	k	*<= Neglect shear contribution of concrete since columns can go into tension.*
V_s =	280	k	LRFD 5.8.3.3
V_u =	252	k	
$V_{elastic}$ =	208	k	*Shear Rein. O.K.*

The shear forces corresponding to plastic hinging have not been computed as the elastic shears are able to be accomodated (as concluded from previous examples).

Pier/Abutment Shaft Design (Force Based)

- ***Flexure***

Column Type:	Circular	
Diameter =	48	in.
f'_c =	3.5	ksi
f_y =	60	ksi
Vert. Bar Size =	14	
# Vert. Bars =	18	
Clear Cover =	2	in.
Assumed Lat. Rein. Size =	6	
$\phi_{flexure}$ =	1.0	NCHRP 12-49 7.8.2.2



Data points for interaction diagram generated using WinRECOL by IMBSEN Software Systems.

Flexure okay by inspection of interaction diagram.

- ***Shear***

$\rho_{s\ min}$ =	0.007		LRFD 5.10.11.4.1c
Max. allowable pitch =	4	in.	LRFD 5.10.11.4.1e
Trans. Bar Size =	6		
Actual Pitch =	4	in.	
$\rho_{s\ prov}$ =	0.008	O.K.	
ϕ_{shear} =	0.9		LRFD 5.5.4.2.1

$V_u = \phi(V_c + V_s)$

$V_c =$ 0 k *<= Neglect shear contribution of concrete since columns can go into tension.*

$V_s =$ 467 k LRFD 5.8.3.3

$V_u =$ 420 k

$V_{elastic} =$ 318 k *Shear Rein. O.K.*

<u>Pier Design</u> (Displacement Based - Pier 1 Only)

- ***Displacement Magnification*** (Imbsen 4.3.3)

$M_w =$ 7.5 *Obtained from USGS website*

$S_s =$ 1.944

$0.4{*}S_s =$ 0.778

$T^* =$ 0.85 Imbsen Table 4.3.3

$T_L =$ 0.52 <T* *Magnify displacements*

$T_T =$ 0.6 <T* *Magnify displacements*

-------> Calculate R for 100% Longitudinal EQ + 30% Transverse EQ

$V_L =$ 716 k
$V_T =$ 383 k
Demand as determined from elastic analysis

$V_{L\ Plastic} =$ 771 k
$V_{T\ Plastic} =$ 728 k
As determined from pushover analysis.

$R_L =$ 0.93 k
$R_T =$ 0.53 k
R<1, Say R_d = 1

-------> Calculate R for 30% Longitudinal EQ + 100% Transverse EQ

$V_L =$ 506 k
$V_T =$ 406 k
Demand as determined from elastic analysis

$V_{L\ Plastic} =$ 771 k
$V_{T\ Plastic} =$ 728 k
As determined from pushover analysis.

$R_L =$ 0.66 k
$R_T =$ 0.56 k
R<1, Say R_d = 1

- ***Member Displacement Check***

$\Delta_{y\ Long} =$ 20.5 in.
$\Delta_{y\ Trans} =$ 3.5 in.
As determined from pushover analysis.

<u>-- 100% Long EQ + 30% Trans EQ</u>

$\Delta_{Long} =$ 5.09 in.
$\Delta_{Trans} =$ 2.95 in. $< \Delta_{y\ Trans}$

<u>-- 30% Long EQ + 100% Trans EQ</u>

Δ_{Long} = 6.53 in. < $\Delta_{y\ Long}$
Δ_{Trans} = 1.72 in.

Summary:

This structure represents the proportionality of a typical bridge in Illinois. Pier columns and drilled shafts are often sized such that the design loads can be accommodated with approximately 2% reinforcement as this provides a reasonable configuration for promoting constructability. By modeling nonlinear soil springs along the length of the drilled shafts to "soften up" the response of the structure and relying upon the passive resistance of the soil behind the abutment in the longitudinal direction of the bridge, it is apparent that the design loads imposed by the larger ground accelerations inherent a 1000 year return period can be reasonably acommodated as illustrated with the modified force based design approach.

Computations for effective section properties using the approach from NCHRP 20-07 and by simply multiplying the gross section properties by one-half yielded very similar results. Although the discrepancy can be expected to vary depending upon the amount of reinforcement present in the column, it's not considered an issue when considering such an emperical design load. Applying a wholistic factor is much simpler than the procedure presented in NCHRP 20-07.

For the displacement based approach, only the pier elements have been considered as these are the components that the code is expected to have the largest impact on. The shear design has not been repeated as it is largely similar to the LRFD code given that the columns and shafts may go into tension and any concrete contribution has been ignored. Upon applying the displacement based approach, it appears that there are several items deserving of more attention. The periods of the structure are such that 20-07 Article 4.3.3 indicates the need to consider displacement magnification. However upon performing the pushover analysis and determing the plastic capacities, the R values are less than one, indicating that the magnification is not necessary. It appears that using the emperical R values provided for SDC C and B could be an unnecessary penalty.

Another area of concern is the lack of direction provided for comparing displacement demands and capacities. For the subject structure, a pushover analysis was conducted in the weak (longitudinal) and strong (transverse) direction of the pier to determine capacities. The capacities are compared to the orthogonally combined displacement demands. The skew and orthogonal combination results in displacements in each of the principal directions of the piers. The rationale for checking displacment capacities is unclear given that in the longitudinal direction bending at the bottom of the column controls the pushover analysis where bending at the top of the column controls for the pushover analysis in the transverse direction.

The displacement based approach can be a more exacting analysis given that many parameters are known with some certainty. However, such an analysis does require emperical estimation of anticipated material properties at the time of a seismic event (i.e., actual yield strength of reinforcement and the increase in concrete strength over time). Given the assumptions which need to be made for most pushover analyses, it is difficult argue that it is any more reliable than a force based method. For example, a comparison of test data from concrete samples taken in Illinois reveals compressive strengths of at least 6500 psi compared to the design strength of 3500 psi which is impractical to take into account for design purposes. The force based method is likely to introduce a conservativism in the design that is not necessarily imprudent or unwise for new structures given the variablity in eartquake magnitude that can be expected with a significant seismic event. Displacement based analysis appears more suited to the rehabilitation of existing structures where the cost of retrofitting often makes it desirable to reduce such conservatism. For new