RC-Pier

LRFD

3 Column Frame Pier Example

305 Wapello

218’ x 40’ PPCB Bridge
(Three Spans: 50.75’ – 116.5’ – 50.75’)
23 deg RA Skew
Frame Piers on Pile Footings
Integral Abutments

Office Example
08-26-2010

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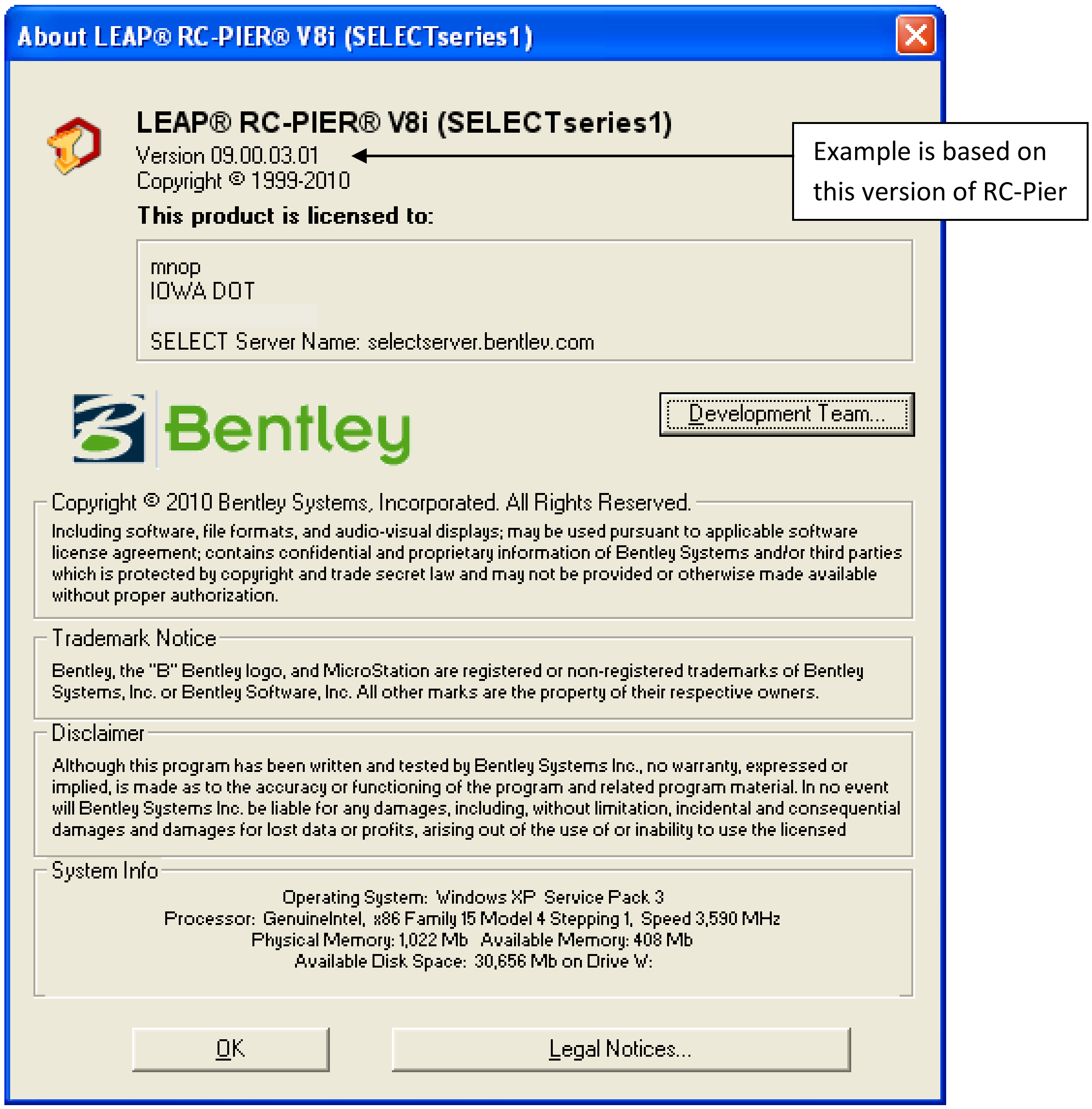
About LEAP® RC-PIER® V8i (SELECTseries1)
LEAP® RC-PIER® V8i (SELECTseries1)
Version 09.00.03.01
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mnop
IOWA DOT
SELECT Server Name: selectserver.bentley.com
Example is based on this version of RC-Pier
Bentley
Development Team...
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System Info
Operating System: Windows XP Service Pack 3
Processor: GenuineIntel, x86 Family 15 Model 4 Stepping 1, Speed 3,590 MHz
Physical Memory: 1,022 Mb Available Memory: 408 Mb
Available Disk Space: 30,656 Mb on Drive W:
OK
Legal Notices...

Notes and Issues

1.) The primary intent of this example is to illustrate the use of RC-Pier, not to show every aspect of a pier design.
2.) Pier No. 1 of the west bound bridge from 305 Wapello forms the basis of this example. In general pier dimensions and reinforcement will not deviate from the existing plans (which are not based on an LRFD design) unless necessary to better illustrate some aspect of RC-Pier or LRFD.
3.) Bridge Office preference is to establish column fixity at the base of the column for the cap and column design and then establish fixity at the base of the footing for the footing and pile design. This policy has a few implications as stated below:
    - Foundation springs due to pile flexibility may be incorporated into both models.
    - For the footing/pile analysis designers may extend the columns in RC-Pier to the bottom of the footing, but designers will not be required to increase the column inertia over the depth of the footing in order to model the footing's properties.
    - In general, the idea is that the applied loads will not have to be adjusted in RC-Pier due to the column height change between the two models. The only loads that would typically be affected by the change in model geometry are self-weight and wind on substructure. The self-weight of each column would go up by the amount of the extension, but this is generally minor; however, the designer may easily correct this if desired. The wind on substructure would be applied to the column somewhat incorrectly because the fractional point of application along the column would be off slightly. This too may easily be corrected by the designer if desired.
    - The designer should determine superstructure temperature loads based on pier fixity at the bottom of the column (with foundations springs if desired).
    - Some loads that may be significantly affected by the change in fixity are the pier's internal shrinkage and temperature change loads. This is particularly true for short piers where an increase in the fixity length from bottom of column to bottom of footing will significantly change the pier's flexibility.
4.) The Bridge Office typically bases wind loading forces on the requirements for "usual girder and slab bridges" from AASHTO LRFD Articles 3.8.1.2.2 and 3.8.1.3 when BDM requirements are met. Iowa allows the use of these provisions for span lengths up to 155' (this is meant to include bridges using Iowa's longest prestressed beam, BTE155) and for top of railing elevations not exceeding 100'. Substructure wind loading is then assumed to be 40 psf in the longitudinal and transverse directions simultaneously. In RC-Pier it is recommended the designer both apply and exclude the wind uplift force from all load combinations since it is conservative and requires less bookkeeping.
5.) There are various issues with RC-Pier version V8i (09.00.03.01). These issues are addressed as they come up in this example.
6.) The Iowa DOT Bridge Design Manual shall be consulted for the most up-to-date DOT policies.

General RC-Pier Layout Geometry

(Figures on this page and the next are taken from the RC-Pier User Manual)

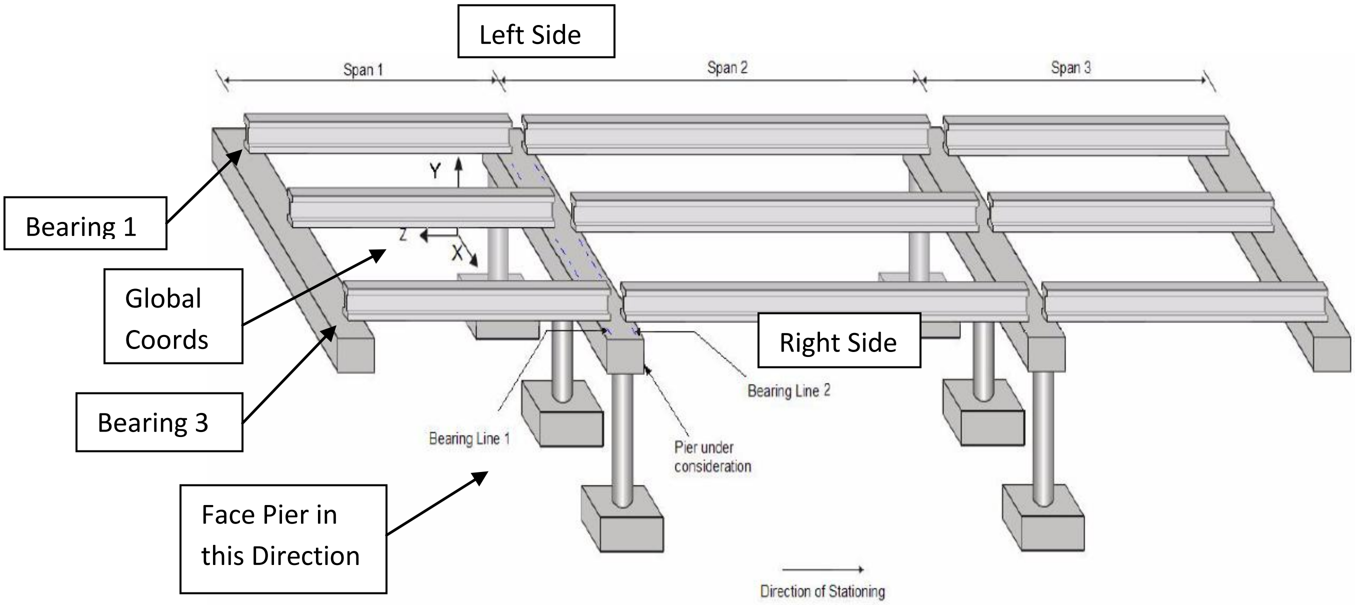


Figure TH-4 Bridge Pier Looking Upstation

Notes

1.) Recommend Upstation View over Downstation View.

2.) Generally Iowa uses only one bearing line in RC-Pier for typical steel and prestressed beam bridges.

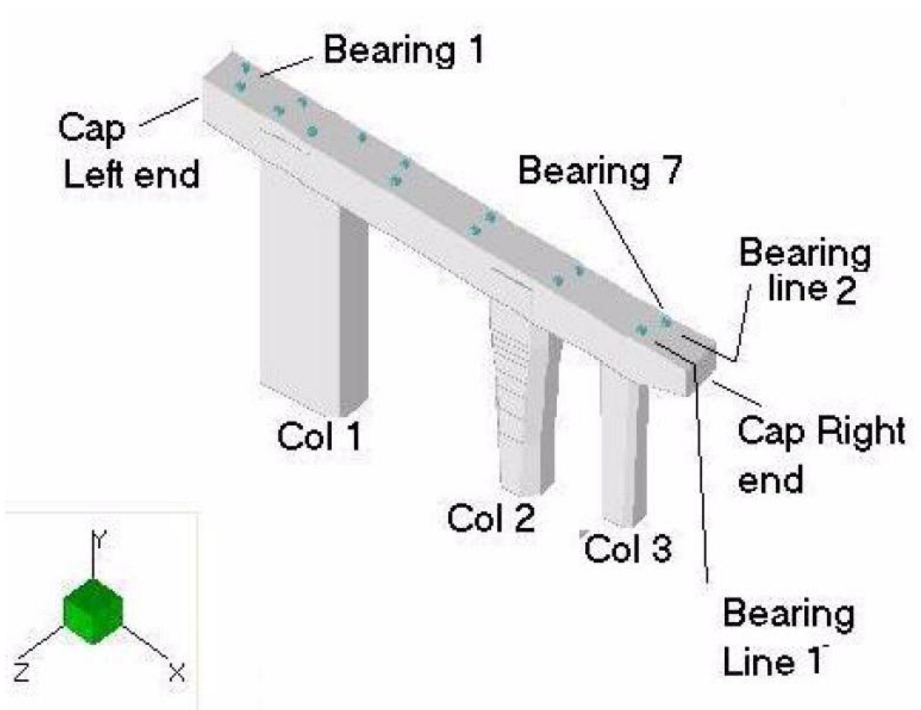


Figure TH-5 Bridge Pier Details in Upstation View

Skew Angle (in degrees)

The skew angle is defined as the angle between the normal to the centerline of the bridge and the centerline of the pier cap (in positive X-direction). It is positive if measured in counterclockwise direction, as shown in Figure TH-9. Note that the skew angle is used only for auto load generation.

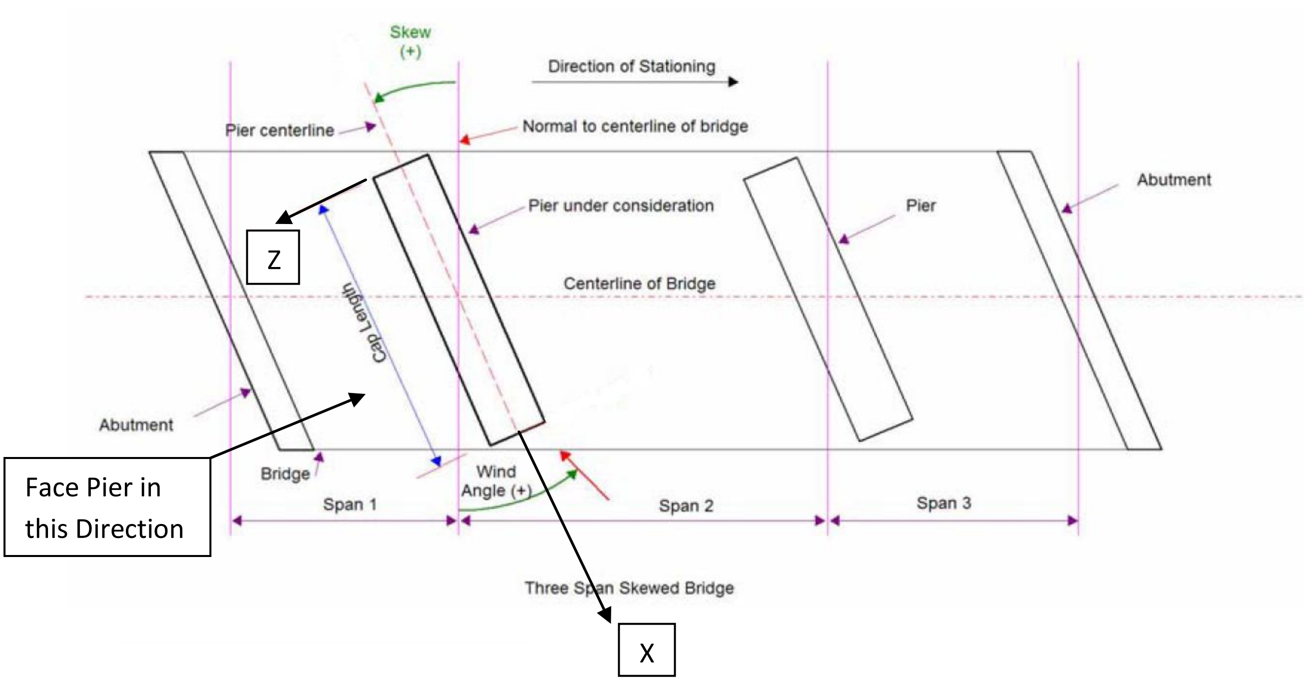


Figure TH-9 Bridge with Piers at Skew
(Modified Coordinate System)

<u>Notes</u>

1.) Redrew coordinate system to make it consistent with Upstation View.
2.) In RC-Pier the global coordinate system rotates with the skew.
3.) Right ahead skews are positive. Left ahead skews are negative.
4.) Face the pier looking in the negative Z-axis direction.

Dead Load: DC1, DC2, and DW

1.) RC-Pier can auto-generate these loads, but there are some drawbacks with doing it this way. Typically the DC1 loads are underestimated because the haunch, intermediate diaphragm, pier diaphragm, and the slab thickening on the overhang are not included. The distribution to the various beams is also based on tributary deck width which doesn't always correlate with Bridge Design Manual policy. So, in general, it is typically better to calculate these apart from RC-Pier and input them manually.
2.) The spreadsheet on the following pages can be used to generate loads for typical prestressed beam bridges. Hand calculations have also been provided as a check.

Pier Dead Load Beam Reactions for Interior and Exterior P/S Beams

Pier beam reactions consider only unfactored superstructure DC and DW loads.

Number of Spans	3	Can be 2 to 6 spans.

Beam Type	D	Beam type must be the same for each span.
Span 1 Beam Size	D50	
Span 2 Beam Size	D110	
Span 3 Beam Size	D50	
~~Span 4 Beam Size~~	---	
~~Span 5 Beam Size~~	---	
~~Span 6 Beam Size~~	---	

There is no D115 beam. The bridge being checked has the "old" LXD beams which are quite similar to the current D beams.

Pier Number of Interest	1	Can be 1, 2, 3, 4, or 5 depending on number of spans.

Default Override	Description	Units	Default	Input/Override	Value Used
☐ Default Override	Span 1 Beam Length, end to end	ft	51.000		51.000
☑ Default Override	Span 2 Beam Length, end to end	ft	~~111.000~~	116.000	116.000
☐ Default Override	Span 3 Beam Length, end to end	ft	51.000		51.000
☐ Default Override	~~Span 4 Beam Length, end to end~~	~~ft~~	~~0.000~~		~~0.000~~
☐ Default Override	~~Span 5 Beam Length, end to end~~	~~ft~~	~~0.000~~		~~0.000~~
☐ Default Override	~~Span 6 Beam Length, end to end~~	~~ft~~	~~0.000~~		~~0.000~~
☐ Default Override	Span 1 Beam Length, c.l. to c.l. bearing	ft	50.000		50.000
☑ Default Override	Span 2 Beam Length, c.l. to c.l. bearing	ft	~~110.000~~	115.000	115.000
☐ Default Override	Span 3 Beam Length, c.l. to c.l. bearing	ft	50.000		50.000
☐ Default Override	~~Span 4 Beam Length, c.l. to c.l. bearing~~	~~ft~~	~~0.000~~		~~0.000~~
☐ Default Override	~~Span 5 Beam Length, c.l. to c.l. bearing~~	~~ft~~	~~0.000~~		~~0.000~~
☐ Default Override	~~Span 6 Beam Length, c.l. to c.l. bearing~~	~~ft~~	~~0.000~~		~~0.000~~
☐ Default Override	Distance bt. Beam Ends on Pier 1	in	6.000		6.000
☐ Default Override	Distance bt. Beam Ends on Pier 2	in	6.000		6.000
☐ Default Override	~~Distance bt. Beam Ends on Pier 3~~	~~in~~	~~0.000~~		~~0.000~~
☐ Default Override	~~Distance bt. Beam Ends on Pier 4~~	~~in~~	~~0.000~~		~~0.000~~
☐ Default Override	~~Distance bt. Beam Ends on Pier 5~~	~~in~~	~~0.000~~		~~0.000~~

Override the default values since I have LXD115s in Span 2.

	Distance bt. Centerline Bearings on Pier 1	ft	1.500		1.500

	Roadway Width (Gutter to Gutter)	ft		40.000	40.000
	Number of Beams in Cross-Section			6.000	6.000

	FWS Load	ksf		0.020	0.020
	SBC Load - Includes Both Rails	klf		0.852	0.852

☐ Default Override	FWS Distribution Factor for Exterior Beam (*)		0.167		0.167
☐ Default Override	FWS Distribution Factor for Interior Beam (*)		0.167		0.167
☐ Default Override	SBC Distribution Factor for Exterior Beam (*)		0.167		0.167
☐ Default Override	SBC Distribution Factor for Interior Beam (*)		0.167		0.167

* Default is an equal distribution of the FWS and SBC loads among all the beams in the cross-section.

☐ Default Override	Pier 1 Reaction Due to 1.00 klf Distributed Load (**)	kips	102.323		102.323

** The pier rxn is used as a ratio to distribute FWS and SBC loads to the pier. It is based on span continuity and a 1.00 klf distr. load.

☐ Default Override	Exterior Beam Reaction for FWS	kips	13.643		13.643
☐ Default Override	Interior Beam Reaction for FWS	kips	13.643		13.643
☐ Default Override	Exterior Beam Reaction for SBC	kips	14.530		14.530
☐ Default Override	Interior Beam Reaction for SBC	kips	14.530		14.530

	Bearing Pad Height at Pier 1 (+)	in		1.000	1.000
	Beam Height at Pier 1	ft	4.500		4.500
	Average Haunch Thickness for Spans 1 and 2	in		1.000	1.000
	Max Haunch Thickness at Pier 1	in		1.750	1.750
	Slab Thickness	in		8.000	8.000
	Slab Cantilever Min. Thickness at Slab Edge	in		8.750	8.750
	Slab Cantilever Max. Thickness at Flange Edge	in		10.250	10.250

+ Bearing pad height is only used in the calculation of pier diaphragm weight for fixed piers.

Top Flange Width	in	20.000		20.000
Beam Area	in^2	638.750		638.750

Beam Spacing Perpendicular to Roadway (++)	ft		7.401	7.401
Slab Cantilever Length (++)	ft		3.083	3.083

++ Beam spacing, slab cantilever length, number of beams, and roadway width should all be consistent.

	Skew, Always Positive	deg		23.000	23.000
☐ Default Override	No. of Intermediate Diaphragms in Span 1		1		1.000
	Enter: 1 = Steel Diaphragm, 2 = Concrete Diaphragm			2	2.000
☐ Default Override	Weight of One Intermediate Diaphragm in Span 1	kips	3.149		3.149
☐ Default Override	No. of Intermediate Diaphragms in Span 2		1		1.000
	Enter: 1 = Steel Diaphragm, 2 = Concrete Diaphragm			2	2.000
☐ Default Override	Weight of One Intermediate Diaphragm in Span 2	kips	3.149		3.149

Perpendicular Thickness of Pier Diaphragm	ft		2.667	2.667
Perp. Extension of Pier Diaph. Past C.L. Ext. Beam (#)	ft		1.583	1.583
Enter: 1 = Fixed Pier, 2 = Expansion Pier			1	1.000

If the pier diaphragm is flush with the exterior side of the exterior beam then enter 0.

See hand calculations on following sheets for more information.

Pier 1		Unfactored Beam Reactions (includes both spans)	
Component	Load Type	Interior Beam kips	Exterior Beam kips
Beam	DC1	55.558	55.558
Slab	DC1	62.168	60.525
Haunch	DC1	1.740	1.740
Intermediate Diaphragms	DC1	3.149	1.574
Pier Diaphragm	DC1	13.529	8.799
SBC	DC2	14.530	14.530
Total (##)	DC Total	150.673	142.726
FWS	DW	13.643	13.643

Pier cap step weight is not included here.

Some designers include pier cap step weight in the beam reactions. That weight is not included here.

	Interior	Exterior
DC1 Pier Cap Step Weight	1.131 k	0.000 k
	150.673 k	142.726 k
Total DC	151.804 k	142.726 k
Total DW	13.643 k	13.643 k

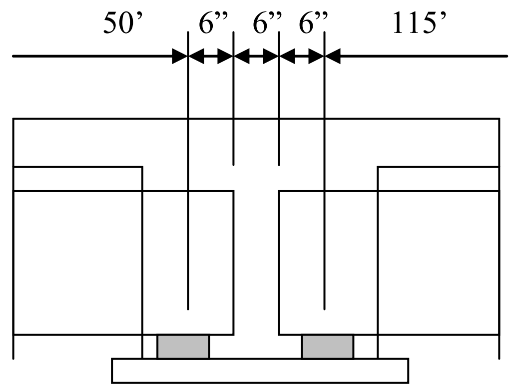
These loads will be used in RC-Pier for the general frame design. The loads for the pier cap overhang will be different.

Hand Calculations for Superstructure Beam Dead Load Reactions

1.) Beam - DC1

Interior & Exterior Beams

$(0.5)*(51\text{ ft} + 116\text{ ft})*[(638.75\text{ in}^2)/(144\text{ in}^2/\text{ft}^2)]*(0.150\text{ kcf}) = \underline{55.558\text{ k}}$



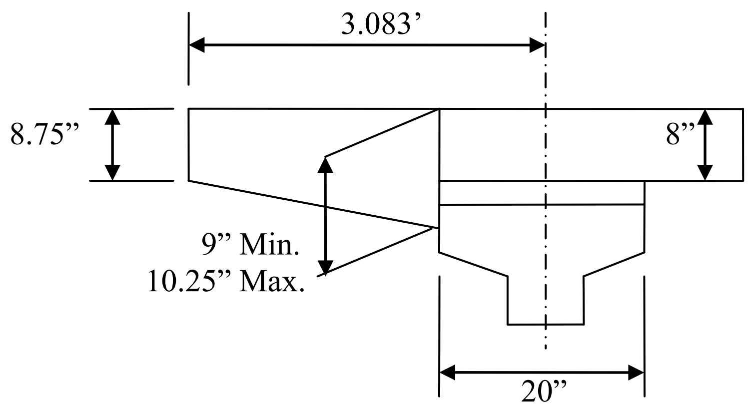
2.) Slab - DC1

Interior Beam

$(7.401')*[(8")/(12\text{ in}^2/\text{ft}^2)]*[(0.5)*(50' + 115') + 1.5']*(0.150\text{ kcf}) = \underline{62.168\text{ k}}$

Exterior Beam

$\{[(0.5)*(7.401') + 3.083']*[(8")/(12\text{ in/ft})] +$
$[3.083' – (0.5)*(20")/(12\text{ in/ft})]*[0.75" + (0.5)*(1.5")]/(12\text{ in/ft})\}*$
$[(0.5)*(50') + (0.5)*(115') + 1.5']*(0.150\text{ kcf}) = \underline{60.525\text{ k}}$



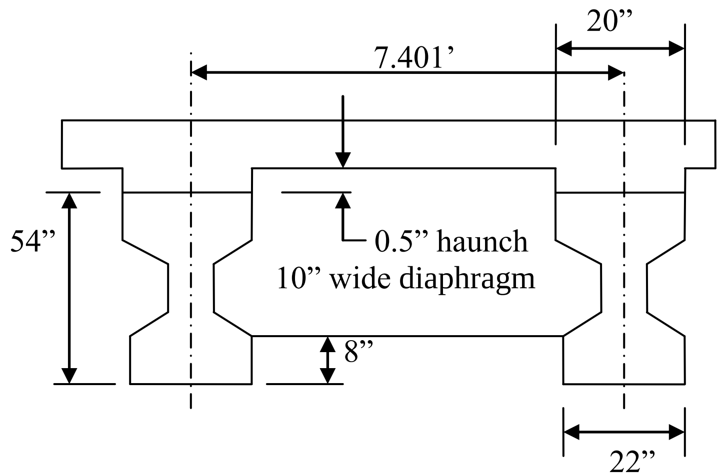
3.) Haunch – DC1

Interior & Exterior Beams

$[(1")*(20")/(144\text{ in}^2/\text{ft}^2)][(0.5)*(50') + (0.5)*(115') + 1']*(0.150\text{ kcf}) = \underline{1.740\text{ k}}$

4.) Intermediate Concrete Diaphragm – DC1

(One diaphragm per span)



Interior Beam

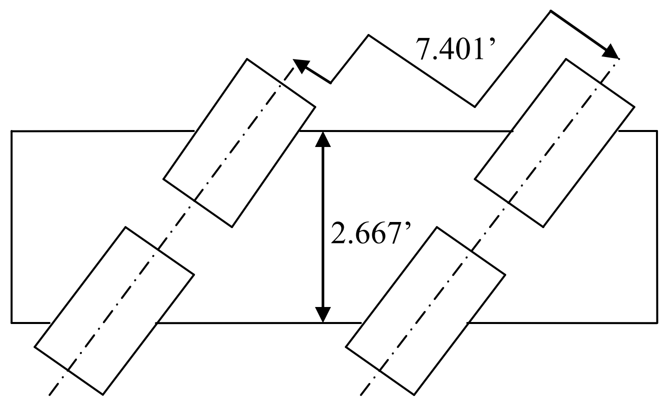
{(7.401’)*[(10”)*(54” + 0.5”)/(144 in^2/ft^2)] - [(638.75 in^2)/(144 in^2/ft^2)]*[(10”)/(12 in/ft)] - [(0.5”)*(20”)*(10”)/(1728 in^3/ft^3)] - [7.401’ - (22”)/(12 in/ft)]*(8”)*(10”)/(144 in^2/ft^2)}* (0.150 kcf) = <u>3.174 k</u>

Exterior Beam

(0.5)*(3.174 k) = <u>1.587 k</u>

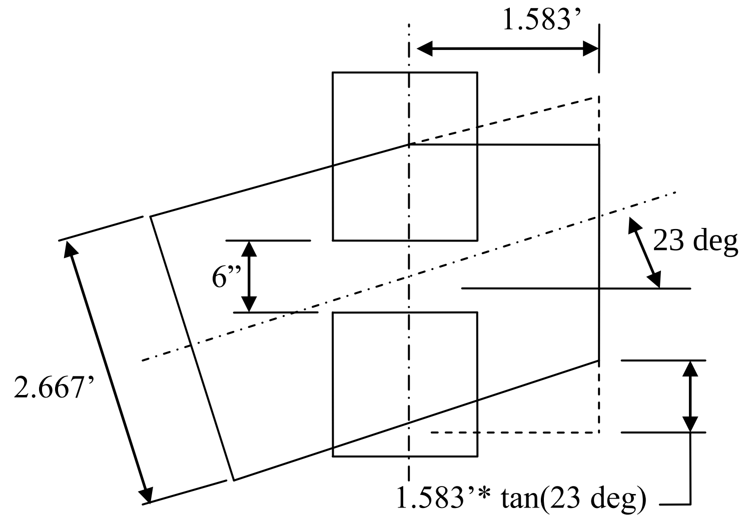
5.) Pier Diaphragm – DC1

The pier diaphragm load is assumed To act through the beams.



Interior Beam

{(7.401’)*[(2.667’)/(cos(23 deg))]*[4.5’ + (1” + 1.75”)/(12 in/ft)] – [(638.75 in^2 + (1.75”)*(20”))/(144 in^2/ft^2)]*[(2.667’)/(cos(23 deg)) – 0.5’]}* (0.150 kcf) = <u>13.529 k</u>



Exterior Beam

(0.5)*(13.529 k) + [4.5’ + (1” + 1.75”)/(12 in/ft)]*(1.583’)*[(2.667’)/(cos(23 deg)) – (0.5)*(1.583’)*(tan(23 deg))] – (0.5)*[(638.75 in^2 + (1.75”)*(20”))/(144 in^2/ft^2)]* [(2.667’)/(cos(23 deg)) – 0.5’]}*(0.150 kcf) = 8.799 k

5.) Pier Diaphragm – DC1

For simplicity we generally assume the pier step load acts through the interior beams.

Interior Only

Average Step Height = 3”
Pier Cap Width = 3.25’
Average Total Step Length Along Pier Cap = 45’ – 7.880’ = 37.12’

[(3”)/(12 in/ft)]*(3.25’)*(37.12’)*(0.150 kcf)/(4 Int. Beams) = 1.131 k

6.) SBC – DC2

Interior & Exterior Beams

Area of One SBC = 2.84 ft^2
Reaction from QConBridge Due to 1.00 k/ft Uniform ead Load = 102.323 k

(2 SBC)*(2.84 ft^2)*(0.150 kcf)*[(102.323 k)/(1.00 k/ft)]/(6 Beams) = 14.530 k

7.) FWS – DW

Interior & Exterior Beams

(0.020 ksf)*(40’)*[(102.323 k)/(1.00 k/ft)]/(6 Beams) = 13.643 k

Total Dead Load

DC Load

		Interior	Exterior
DC1	Beam	55.558 k	55.558 k
DC1	Slab	62.168 k	60.525 k
DC1	Haunch	1.740 k	1.740 k
DC1	Interm. Diaph.	3.149 k	1.574 k
DC1	Pier Diaph.	13.529 k	8.799 k
DC1	Pier Steps	1.131 k	0.000 k
DC2 →	SBC	14.530 k	14.530 k
	Total DC	151.805 k	142.726 k

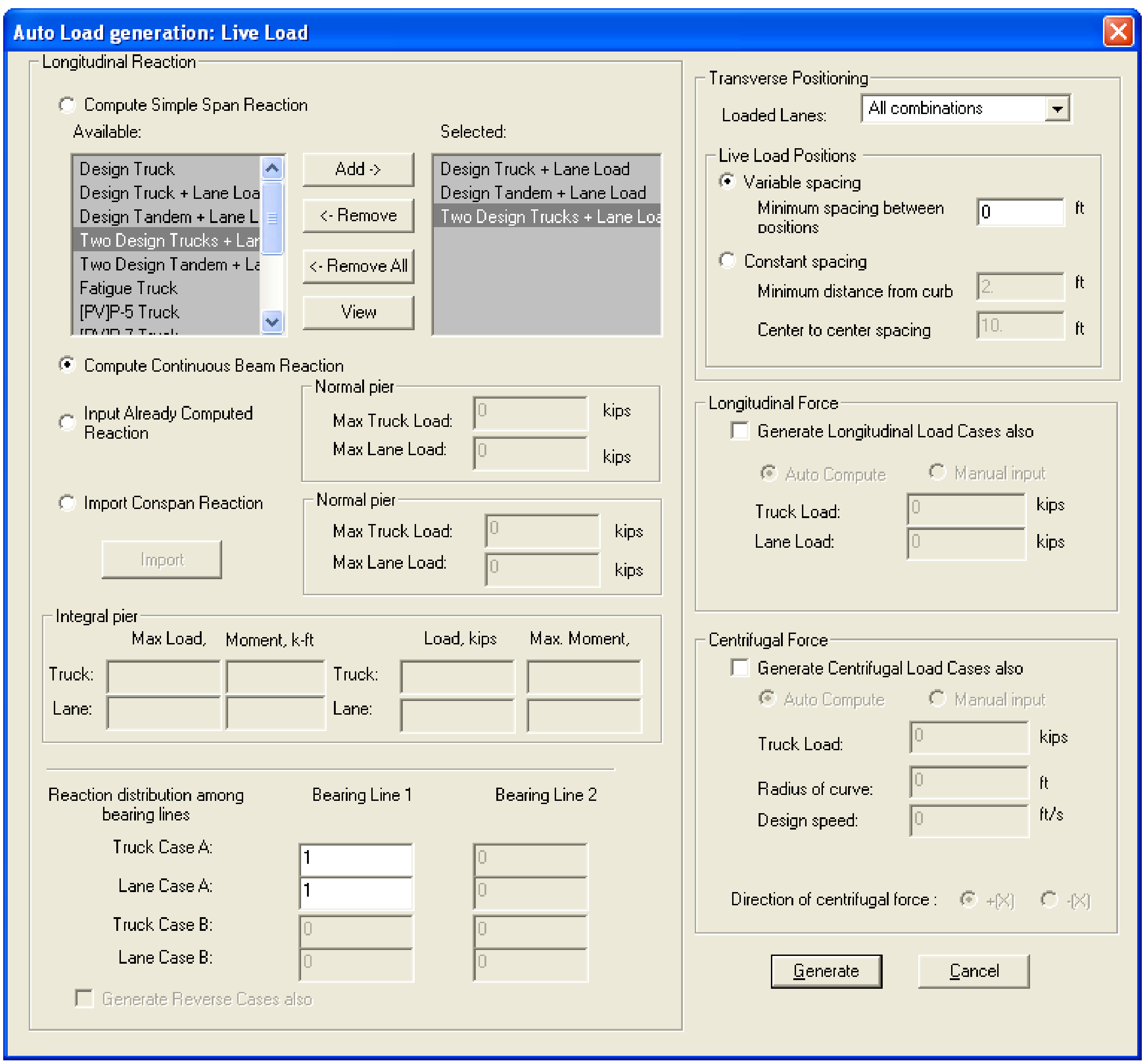
DW Load

	Interior	Exterior
FWS	13.643 k	13.643 k

Live Load: LL

There are a number of ways live load can be done in RC-Pier.

1.) Use QConBridge to get the live load pier reaction. Move the live load(s) transversely back and forth across the deck width and determine the beam reactions for those arrangements that maximize force effects in the pier. Typically placement of live load for maximum force effects can be done intuitively. The spreadsheet on the following pages facilitates this method and consequently is used for this example.
2.) Another method is to use RC-Pier's auto-generation feature for determination of live loads. The program is capable of determining the pier live load reaction for a continuous bridge with a constant moment of inertia. (For this example I checked the live load reaction QConBridge came up with against RC-Pier's value and the two compared quite well.) The user can use RC-Pier's live load reaction, import it from Conspan, or enter their own. Once RC-Pier has this determined there are basically two ways to obtain the actual live load cases.
   a.) Variable spacing
   b.) Constant spacing



RC-Pier's manual explains how the two methods work. Essentially each method follows an algorithm for determining how many different live load positions are possible. The program then seeks to maximize forces for each member and keeps only the live load arrangements that do this. In general the variable spacing method is going to check more possibilities and thus produce more live load cases (especially with 0' as the "Minimum spacing between positions"). This in turn will increase the number of load combinations and computing time. For the settings shown above the variable spacing method comes up with 30 different live load cases. The constant spacing method results in 14 live load cases.

Note:
When live loads are auto-generated in RC-Pier the user can review some of the processes RC-Pier went through in order to determine the live load cases. There is a "LL details" button on the Loads tab screen that brings this information up. One shortcoming of RC-Pier is that the information for the truck positions in these details is based on a downstation view of the pier even when the user is working in the upstation view mode. I've contacted the developers and they are already aware of the inconsistency and it is on their list of things to fix.

Live Load QConBridge Runs are on the following pages.

Run 1:

This was done to determine what live load controls for the pier reaction. The dual truck train with lane controlled.

Max. LL + I = 185.558 k Dual Truck Train + Lane
Impact
Axle Load
Unfactored

Run 2:

This was done to determine the truck portion of the controlling live load since RC-Pier entry requires the truck and lane to be separated in order to track impact application for the various pier components.

Dual Truck Rxn = 94.589 k No Lane
No Impact
Axle Load
Unfactored

Run 3:

This was done to determine the lane portion of the controlling live load since RC-Pier entry requires the truck and lane to be separated in order to track application for the various pier components.

Dual Lane Rxn = 59.754 k No Truck
No Impact
Axle Load
Unfactored

Check: (94.589 k)*(1.33) + 59.754 k = 185.557 k

Note: The dual truck train + lane often controls the pier reaction. It needs to be remembered that the truck and lane weights are reduced to 90% and that this reduction is already included in the reactions above.

The next page details how the reactions for the truck and lane can be obtained separately in QConBridge.

QConBridge Version 1.3

Getting Truck Load and Lane Load Separately for HL-93 Loading.

This description is taken directly from the Washington DOT website:
http://www.wsdot.wa.gov/eesc/bridge/software/

Q2 How can I get the truck load and lane load results separately?

A2 The HL93 Live Load model consists of the truck and lane applied simultaneously, along with appropriate dynamic load allowance (impact) factors. This is how QConBridge approaches the problem, so there is no direct way to separate the truck and lane response.

However, there is a "trick" that you can use to "turn off" either the truck or lane load. The trick is to use a dynamic load allowance of -100% for the load component you want to turn off. Truck and lane responses are scaled by (1.0 + IM/100) where IM is the applicable dynamic load allowance factor. Using a factor of -100% the response is scaled by (1.0 + -100/100) = 0.0, which, in effect, "turns off" the response.

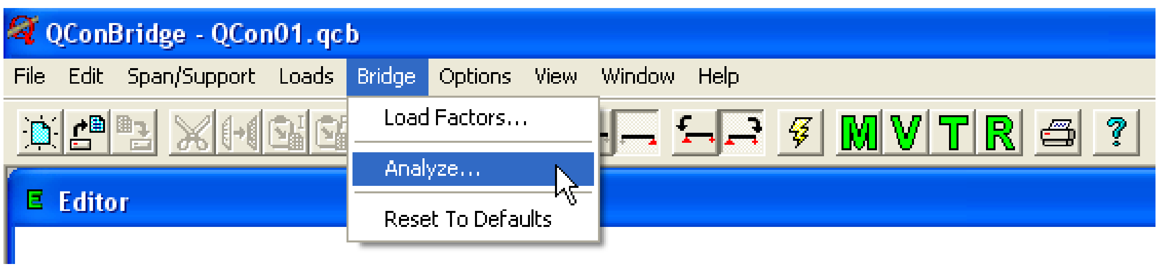
To modify the dynamic load allowance, select Loads | Dynamic Load Allowance... Enter a value of -100% for **either** Truck or Lane. Press the OK button and run the analysis.

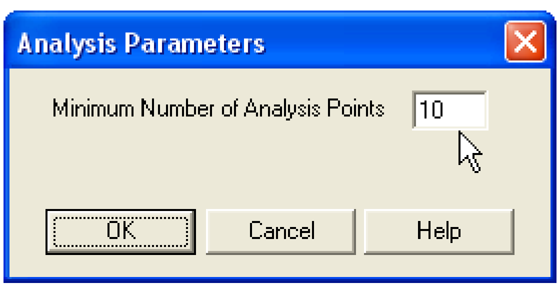
Thanks to Dr. Harry Cole from the Mississippi State University for sharing this tip. (Go Bulldogs)

QConBridge Version 1.3

Minimum Number of Analysis Points

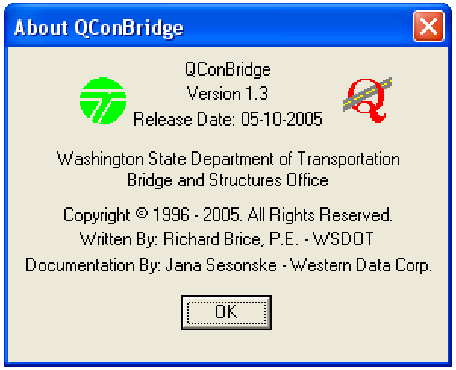
Use a minimum of 10 analysis points for any QConBridge run. QConBridge uses a finer influence line as more analysis points are used. Every axle on every truck is placed at every analysis point. If you decrease the number of points from 10 your results will likely be off by a significant percentage. In order to get reasonable results the minimum default value of 10 analysis points should be used. You can also use more than 10 analysis points, but this isn't typically necessary and as you increase the number of analysis points you increase the time of execution which can be substantial for bridges with a large number of spans.

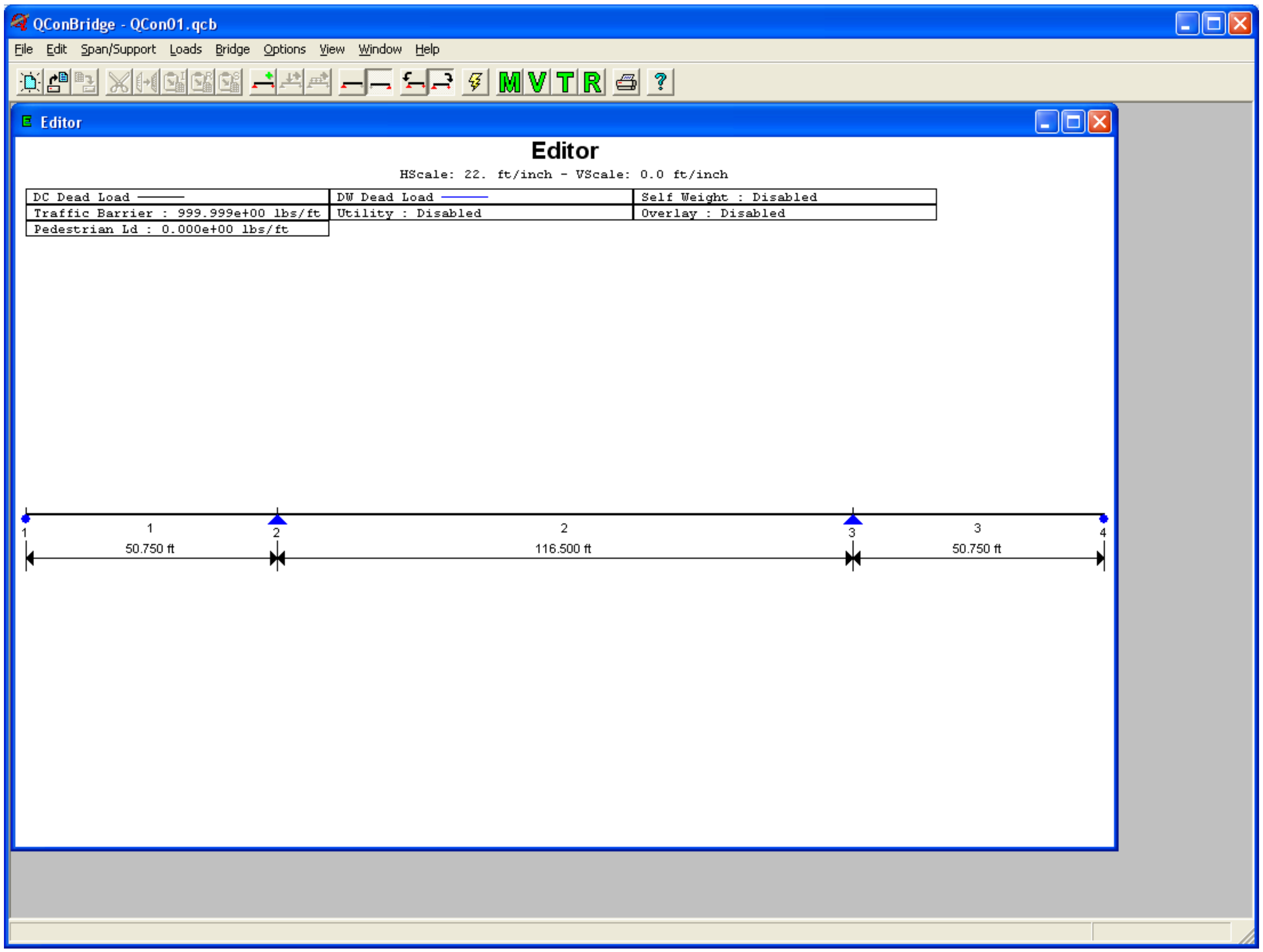


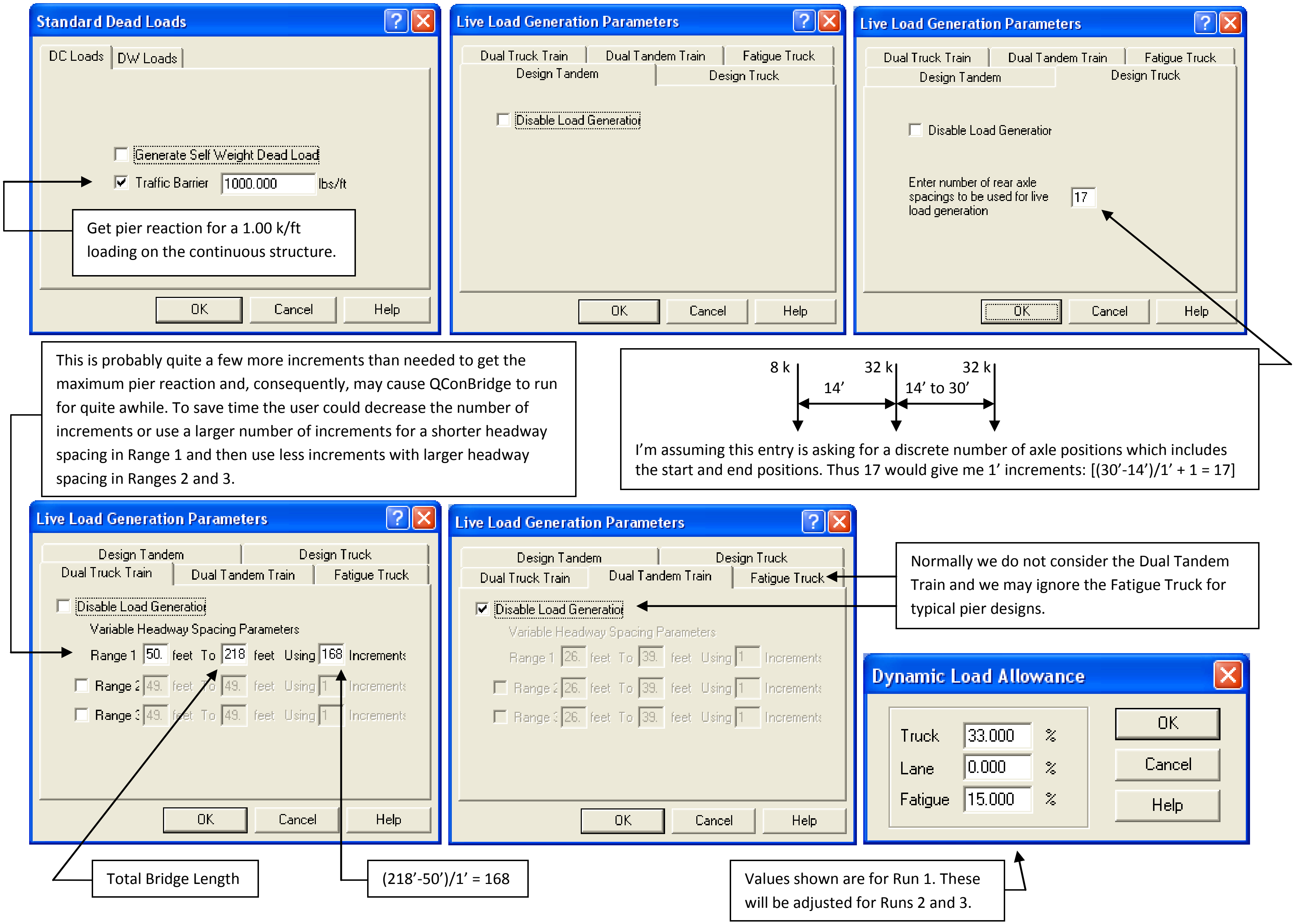


Load Factors

Typically we are only interested in unfactored LL pier reactions from QConBridge. However, it is worth noting that changing the load factors in QConBridge Version 1.3 does show the change in the load factors in the output's echo of the input, but the results for the limit states are not affected. The default load factors always seem to be used in the factored results.


About QConBridge
QConBridge
Version 1.3
Release Date: 05-10-2005
Washington State Department of Transportation
Bridge and Structures Office
Copyright © 1996 - 2005. All Rights Reserved.
Written By: Richard Brice, P.E. - WSDOT
Documentation By: Jana Sesonske - Western Data Corp.
OK


QConBridge - QCon01.qcb
File Edit Span/Support Loads Bridge Options View Window Help
M V T R
Editor
Editor
HScale: 22. ft/inch - VScale: 0.0 ft/inch
DC Dead Load
DW Dead Load
Self Weight : Disabled
Traffic Barrier : 999.999e+00 lbs/ft
Utility : Disabled
Overlay : Disabled
Pedestrian Ld : 0.000e+00 lbs/ft
1
2
3
50.750 ft
116.500 ft
50.750 ft
1
2
3
4


Standard Dead Loads
DC Loads
DW Loads
Generate Self Weight Dead Load
Traffic Barrier 1000.000 lbs/ft
Get pier reaction for a 1.00 k/ft loading on the continuous structure.
OK Cancel Help
Live Load Generation Parameters
Dual Truck Train
Dual Tandem Train
Fatigue Truck
Design Tandem
Design Truck
Disable Load Generatio
OK Cancel Help
Live Load Generation Parameters
Dual Truck Train
Dual Tandem Train
Fatigue Truck
Design Tandem
Design Truck
Disable Load Generatior
Enter number of rear axle spacings to be used for live load generation 17
OK Cancel Help
This is probably quite a few more increments than needed to get the maximum pier reaction and, consequently, may cause QConBridge to run for quite awhile. To save time the user could decrease the number of increments or use a larger number of increments for a shorter headway spacing in Range 1 and then use less increments with larger headway spacing in Ranges 2 and 3.
8 k
32 k
32 k
14'
14' to 30'
I'm assuming this entry is asking for a discrete number of axle positions which includes the start and end positions. Thus 17 would give me 1' increments: [(30'-14')/1' + 1 = 17]
Live Load Generation Parameters
Design Tandem
Design Truck
Dual Truck Train
Dual Tandem Train
Fatigue Truck
Disable Load Generatio
Variable Headway Spacing Parameters
Range 1 50. feet To 218 feet Using 168 Increments
Range 2 49. feet To 49. feet Using 1 Increments
Range 3 49. feet To 49. feet Using 1 Increments
OK Cancel Help
Live Load Generation Parameters
Design Tandem
Design Truck
Dual Truck Train
Dual Tandem Train
Fatigue Truck
Disable Load Generatio
Variable Headway Spacing Parameters
Range 1 26. feet To 39. feet Using 1 Increments
Range 2 26. feet To 39. feet Using 1 Increments
Range 3 26. feet To 39. feet Using 1 Increments
OK Cancel Help
Normally we do not consider the Dual Tandem Train and we may ignore the Fatigue Truck for typical pier designs.
Dynamic Load Allowance
Truck 33.000 %
Lane 0.000 %
Fatigue 15.000 %
OK
Cancel
Help
Total Bridge Length
(218'-50')/1' = 168
Values shown are for Run 1. These will be adjusted for Runs 2 and 3.

Washington State Department of Transporation
Bridge and Structures Office
QConBridge Version 1.0

QConBridge
Run 1 Output

Max LL+I Rxn = 185.558 k — Dual Truck Train + Lane Controls

Rxn Due to 1.00 k/ft Uniform Load = 102.323 k

Code: LRFD First Edition 1994

Span Data

Span 1 Length: 50.750 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	999.999e-03	1.000e+03	999.997e-03

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Span 2 Length: 116.500 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	999.999e-03	1.000e+03	999.997e-03

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Span 3 Length: 50.750 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	999.999e-03	1.000e+03	999.997e-03

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Support Data

Support 1 Roller

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Support  2  Pinned


Support  3  Pinned


Support  4  Roller


Loading Data
------------
DC Loads
   Self Weight Generation Disabled
   Traffic Barrier Load 999.999e+00 plf

DW Loads
   Utility Load Disabled
   Wearing Surface Load Disabled

Live Load Data
--------------
Live Load Generation Parameters
   Design Tandem    : Enabled
   Design Truck     : 17 rear axle spacing increments
   Dual Truck Train : Headway Spacing varies from 50.000 ft to 218.000 ft using 168 increments
   Dual Tandem Train: Disabled
   Fatigue Truck    : Disabled

Live Load Impact
   Truck Loads   33.000%
   Lane  Loads    0.000%
   Fatigue Truck 15.000%

   Pedestrian Live Load    0.000e+00 plf

Load Factors
------------
   Strength I   DC min 0.900 DC max 1.250 DW min 0.650 DW max 1.500 LL     1.750
   Service I    DC     1.000 DW     1.000 LL     1.000
   Service II   DC     1.000 DW     1.000 LL     1.300
   Service III  DC     1.000 DW     1.000 LL     0.800
   Fatigue      DC     0.000 DW     0.000 LL     0.750

Analysis Results
----------------
DC Dead Load
```

Impact

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	6.676e+03	0.000e+00
1	1	1.601e+03	21.006e+03
1	2	-3.473e+03	16.258e+03
1	3	-8.548e+03	-14.246e+03
1	4	-13.623e+03	-70.506e+03
1	5	-18.698e+03	-152.521e+03
1	6	-23.773e+03	-260.292e+03
1	7	-28.848e+03	-393.819e+03
1	8	-33.923e+03	-553.102e+03
1	9	-38.998e+03	-738.140e+03
1	10	-44.073e+03	-948.933e+03
2	0	58.249e+03	-948.933e+03
2	1	46.599e+03	-338.182e+03
2	2	34.949e+03	136.846e+03
2	3	23.300e+03	476.152e+03

2	4	11.650e+03	679.735e+03
2	5	0.000e+00	747.597e+03
2	6	-11.650e+03	679.735e+03
2	7	-23.300e+03	476.152e+03
2	8	-34.949e+03	136.846e+03
2	9	-46.599e+03	-338.182e+03
2	10	-58.249e+03	-948.933e+03
3	0	44.073e+03	-948.933e+03
3	1	38.998e+03	-738.140e+03
3	2	33.923e+03	-553.102e+03
3	3	28.848e+03	-393.819e+03
3	4	23.773e+03	-260.292e+03
3	5	18.698e+03	-152.521e+03
3	6	13.623e+03	-70.506e+03
3	7	8.548e+03	-14.246e+03
3	8	3.473e+03	16.258e+03
3	9	-1.601e+03	21.006e+03
3	10	-6.676e+03	0.000e+00

DC Dead Load

Pier	Fx(lbs)	Fy(lbs)	Mz(ft-lbs)
1	0.000e+00	6.676e+03	0.000e+00
2	0.000e+00	102.323e+03	0.000e+00
3	0.000e+00	102.323e+03	0.000e+00
4	0.000e+00	6.676e+03	0.000e+00

DW Dead Load

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00

DW Dead Load

Pier	Fx(lbs)	Fy(lbs)	Mz(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00

Live Load Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-33.742e+03	91.969e+03	0.000e+00	0.000e+00
1	1	-33.931e+03	77.991e+03	-171.242e+03	403.054e+03
1	2	-34.498e+03	64.670e+03	-342.484e+03	681.560e+03
1	3	-35.436e+03	52.108e+03	-513.726e+03	841.402e+03
1	4	-41.204e+03	40.395e+03	-684.969e+03	918.412e+03
1	5	-51.245e+03	29.975e+03	-856.211e+03	910.419e+03
1	6	-63.085e+03	22.023e+03	-1.027e+06	856.117e+03
1	7	-75.515e+03	15.012e+03	-1.198e+06	708.328e+03
1	8	-87.725e+03	8.801e+03	-1.369e+06	483.475e+03
1	9	-99.581e+03	3.650e+03	-1.549e+06	215.847e+03
1	10	-110.962e+03	2.398e+03	-1.821e+06	121.735e+03
2	0	-4.045e+03	127.249e+03	-1.821e+06	121.735e+03
2	1	-4.403e+03	114.626e+03	-894.241e+03	300.657e+03
2	2	-10.555e+03	98.188e+03	-274.486e+03	813.139e+03
2	3	-20.321e+03	81.106e+03	-212.094e+03	1.412e+06
2	4	-33.116e+03	64.115e+03	-174.428e+03	1.803e+06
2	5	-47.909e+03	47.909e+03	-136.762e+03	1.921e+06
2	6	-64.115e+03	33.116e+03	-174.428e+03	1.803e+06
2	7	-81.106e+03	20.321e+03	-212.094e+03	1.412e+06
2	8	-98.188e+03	10.555e+03	-274.486e+03	813.139e+03
2	9	-114.626e+03	4.403e+03	-894.241e+03	300.657e+03
2	10	-129.644e+03	4.045e+03	-1.821e+06	121.735e+03
3	0	-2.398e+03	109.783e+03	-1.821e+06	121.735e+03
3	1	-3.650e+03	99.581e+03	-1.549e+06	215.847e+03
3	2	-8.801e+03	87.725e+03	-1.369e+06	483.475e+03
3	3	-15.012e+03	75.515e+03	-1.198e+06	708.328e+03
3	4	-22.023e+03	63.085e+03	-1.027e+06	856.117e+03
3	5	-29.975e+03	51.245e+03	-856.211e+03	910.419e+03
3	6	-40.395e+03	41.204e+03	-684.969e+03	918.412e+03
3	7	-52.108e+03	35.436e+03	-513.726e+03	841.402e+03
3	8	-64.670e+03	34.498e+03	-342.484e+03	681.560e+03
3	9	-77.991e+03	33.931e+03	-171.242e+03	403.054e+03
3	10	-91.969e+03	33.742e+03	0.000e+00	0.000e+00

Unfactored Max LL+I Rxn based on all live loads: Dual Truck Train + Lane Controls

Live Load Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-33.742e+03	91.969e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-6.443e+03	185.558e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-6.443e+03	185.558e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-33.742e+03	91.969e+03	0.000e+00	0.000e+00

Design Tandem + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-26.836e+03	78.060e+03	0.000e+00	0.000e+00
1	1	-27.026e+03	67.340e+03	-136.196e+03	348.996e+03
1	2	-27.592e+03	57.086e+03	-272.392e+03	604.578e+03
1	3	-32.534e+03	47.377e+03	-408.588e+03	769.378e+03
1	4	-41.204e+03	38.257e+03	-544.785e+03	847.101e+03
1	5	-49.984e+03	29.788e+03	-680.981e+03	848.410e+03
1	6	-58.787e+03	22.023e+03	-817.177e+03	787.317e+03
1	7	-67.526e+03	15.012e+03	-953.374e+03	654.252e+03
1	8	-76.111e+03	8.801e+03	-1.089e+06	457.805e+03

1	9	-84.449e+03	3.433e+03	-1.233e+06	215.847e+03
1	10	-92.442e+03	1.986e+03	-1.432e+06	100.824e+03
2	0	-3.350e+03	100.348e+03	-1.432e+06	100.824e+03
2	1	-4.403e+03	91.534e+03	-612.246e+03	300.657e+03
2	2	-10.555e+03	78.899e+03	-230.636e+03	747.598e+03
2	3	-18.909e+03	65.832e+03	-176.339e+03	1.222e+06
2	4	-28.946e+03	52.830e+03	-146.768e+03	1.537e+06
2	5	-40.386e+03	40.386e+03	-117.197e+03	1.641e+06
2	6	-52.830e+03	28.946e+03	-146.768e+03	1.537e+06
2	7	-65.832e+03	18.909e+03	-176.339e+03	1.222e+06
2	8	-78.899e+03	10.555e+03	-230.636e+03	747.598e+03
2	9	-91.534e+03	4.403e+03	-612.246e+03	300.657e+03
2	10	-103.027e+03	3.350e+03	-1.432e+06	100.824e+03
3	0	-1.986e+03	91.403e+03	-1.432e+06	100.824e+03
3	1	-3.433e+03	84.449e+03	-1.233e+06	215.847e+03
3	2	-8.801e+03	76.111e+03	-1.089e+06	457.805e+03
3	3	-15.012e+03	67.526e+03	-953.374e+03	654.252e+03
3	4	-22.023e+03	58.787e+03	-817.177e+03	787.317e+03
3	5	-29.788e+03	49.984e+03	-680.981e+03	848.410e+03
3	6	-38.257e+03	41.204e+03	-544.785e+03	847.101e+03
3	7	-47.377e+03	32.534e+03	-408.588e+03	769.378e+03
3	8	-57.086e+03	27.592e+03	-272.392e+03	604.578e+03
3	9	-67.340e+03	27.026e+03	-136.196e+03	348.996e+03
3	10	-78.060e+03	26.836e+03	0.000e+00	0.000e+00

Design Tandem + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-26.836e+03	78.060e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-5.337e+03	136.287e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-5.337e+03	136.287e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-26.836e+03	78.060e+03	0.000e+00	0.000e+00

Design Truck + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-33.742e+03	91.969e+03	0.000e+00	0.000e+00
1	1	-33.931e+03	77.991e+03	-171.242e+03	403.054e+03
1	2	-34.498e+03	64.670e+03	-342.484e+03	681.560e+03
1	3	-35.436e+03	52.108e+03	-513.726e+03	841.402e+03
1	4	-39.947e+03	40.395e+03	-684.969e+03	918.412e+03
1	5	-51.245e+03	29.975e+03	-856.211e+03	910.419e+03
1	6	-63.085e+03	20.790e+03	-1.027e+06	856.117e+03
1	7	-75.515e+03	12.587e+03	-1.198e+06	708.328e+03
1	8	-87.725e+03	7.467e+03	-1.369e+06	483.475e+03
1	9	-99.581e+03	3.650e+03	-1.549e+06	187.752e+03
1	10	-110.962e+03	2.398e+03	-1.782e+06	121.735e+03
2	0	-4.045e+03	127.249e+03	-1.782e+06	121.735e+03
2	1	-4.273e+03	114.626e+03	-755.848e+03	248.299e+03
2	2	-10.363e+03	98.188e+03	-274.486e+03	813.139e+03
2	3	-20.321e+03	81.106e+03	-212.094e+03	1.412e+06
2	4	-33.116e+03	64.115e+03	-174.428e+03	1.803e+06
2	5	-47.909e+03	47.909e+03	-136.762e+03	1.921e+06
2	6	-64.115e+03	33.116e+03	-174.428e+03	1.803e+06
2	7	-81.106e+03	20.321e+03	-212.094e+03	1.412e+06
2	8	-98.188e+03	10.363e+03	-274.486e+03	813.139e+03
2	9	-114.626e+03	4.273e+03	-755.848e+03	248.299e+03
2	10	-129.644e+03	4.045e+03	-1.782e+06	121.735e+03
3	0	-2.398e+03	109.783e+03	-1.782e+06	121.735e+03
3	1	-3.650e+03	99.581e+03	-1.549e+06	187.752e+03
3	2	-7.467e+03	87.725e+03	-1.369e+06	483.475e+03
3	3	-12.587e+03	75.515e+03	-1.198e+06	708.328e+03
3	4	-20.790e+03	63.085e+03	-1.027e+06	856.117e+03
3	5	-29.975e+03	51.245e+03	-856.211e+03	910.419e+03

3	6	-40.395e+03	39.947e+03	-684.969e+03	918.412e+03
3	7	-52.108e+03	35.436e+03	-513.726e+03	841.402e+03
3	8	-64.670e+03	34.498e+03	-342.484e+03	681.560e+03
3	9	-77.991e+03	33.931e+03	-171.242e+03	403.054e+03
3	10	-91.969e+03	33.742e+03	0.000e+00	0.000e+00

Design Truck + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-33.742e+03	91.969e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-6.443e+03	167.658e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-6.443e+03	167.658e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-33.742e+03	91.969e+03	0.000e+00	0.000e+00

Dual Truck Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	-478.212e+03	0.000e+00
1	4	0.000e+00	0.000e+00	-637.616e+03	0.000e+00
1	5	0.000e+00	0.000e+00	-797.020e+03	0.000e+00
1	6	0.000e+00	0.000e+00	-956.424e+03	0.000e+00
1	7	0.000e+00	0.000e+00	-1.115e+06	0.000e+00
1	8	0.000e+00	0.000e+00	-1.275e+06	0.000e+00
1	9	0.000e+00	0.000e+00	-1.441e+06	0.000e+00
1	10	0.000e+00	0.000e+00	-1.821e+06	0.000e+00
2	0	0.000e+00	0.000e+00	-1.821e+06	0.000e+00
2	1	0.000e+00	0.000e+00	-894.241e+03	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	-894.241e+03	0.000e+00
2	10	0.000e+00	0.000e+00	-1.821e+06	0.000e+00
3	0	0.000e+00	0.000e+00	-1.821e+06	0.000e+00
3	1	0.000e+00	0.000e+00	-1.441e+06	0.000e+00
3	2	0.000e+00	0.000e+00	-1.275e+06	0.000e+00
3	3	0.000e+00	0.000e+00	-1.115e+06	0.000e+00
3	4	0.000e+00	0.000e+00	-956.424e+03	0.000e+00
3	5	0.000e+00	0.000e+00	-797.020e+03	0.000e+00
3	6	0.000e+00	0.000e+00	-637.616e+03	0.000e+00
3	7	0.000e+00	0.000e+00	-478.212e+03	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Truck Train + Lane Controls

Dual Truck Train + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-5.799e+03	185.558e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-5.799e+03	185.558e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tandem Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00

1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tandem Train + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Truck Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Truck Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Strength I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-53.039e+03	169.292e+03	0.000e+00	0.000e+00
1	1	-57.938e+03	138.488e+03	-280.767e+03	731.604e+03
1	2	-64.713e+03	110.047e+03	-584.715e+03	1.213e+06
1	3	-72.699e+03	83.495e+03	-916.829e+03	1.459e+06
1	4	-89.136e+03	58.431e+03	-1.286e+06	1.543e+06
1	5	-113.053e+03	35.628e+03	-1.689e+06	1.455e+06
1	6	-140.115e+03	17.145e+03	-2.123e+06	1.263e+06
1	7	-168.212e+03	308.398e+00	-2.589e+06	885.137e+03
1	8	-195.924e+03	-15.128e+03	-3.088e+06	348.289e+03
1	9	-223.015e+03	-28.709e+03	-3.633e+06	-286.593e+03
1	10	-249.276e+03	-35.468e+03	-4.373e+06	-641.004e+03
2	0	45.345e+03	295.498e+03	-4.373e+06	-641.004e+03
2	1	34.233e+03	258.846e+03	-1.987e+06	221.785e+03
2	2	12.982e+03	215.517e+03	-357.190e+03	1.594e+06
2	3	-14.592e+03	171.061e+03	57.370e+03	3.067e+06
2	4	-47.468e+03	126.764e+03	306.512e+03	4.005e+06
2	5	-83.841e+03	83.841e+03	433.503e+03	4.296e+06
2	6	-126.764e+03	47.468e+03	306.512e+03	4.005e+06
2	7	-171.061e+03	14.592e+03	57.370e+03	3.067e+06
2	8	-215.517e+03	-12.982e+03	-357.190e+03	1.594e+06
2	9	-258.846e+03	-34.233e+03	-1.987e+06	221.785e+03
2	10	-299.689e+03	-45.345e+03	-4.373e+06	-641.004e+03
3	0	35.468e+03	247.212e+03	-4.373e+06	-641.004e+03
3	1	28.709e+03	223.015e+03	-3.633e+06	-286.593e+03
3	2	15.128e+03	195.924e+03	-3.088e+06	348.289e+03
3	3	-308.398e+00	168.212e+03	-2.589e+06	885.137e+03
3	4	-17.145e+03	140.115e+03	-2.123e+06	1.263e+06
3	5	-35.628e+03	113.053e+03	-1.689e+06	1.455e+06
3	6	-58.431e+03	89.136e+03	-1.286e+06	1.543e+06
3	7	-83.495e+03	72.699e+03	-916.829e+03	1.459e+06
3	8	-110.047e+03	64.713e+03	-584.715e+03	1.213e+06
3	9	-138.488e+03	57.938e+03	-280.767e+03	731.604e+03
3	10	-169.292e+03	53.039e+03	0.000e+00	0.000e+00

Strength I Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-53.039e+03	169.292e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	80.814e+03	452.630e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	80.814e+03	452.630e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-53.039e+03	169.292e+03	0.000e+00	0.000e+00

Service I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-27.065e+03	98.646e+03	0.000e+00	0.000e+00
1	1	-32.329e+03	79.593e+03	-150.235e+03	424.061e+03
1	2	-37.971e+03	61.197e+03	-326.226e+03	697.819e+03
1	3	-43.985e+03	43.559e+03	-527.972e+03	827.156e+03
1	4	-54.827e+03	26.772e+03	-755.475e+03	847.906e+03
1	5	-69.944e+03	11.277e+03	-1.008e+06	757.897e+03
1	6	-86.858e+03	-1.749e+03	-1.287e+06	595.824e+03
1	7	-104.363e+03	-13.835e+03	-1.592e+06	314.509e+03
1	8	-121.649e+03	-25.121e+03	-1.923e+06	-69.626e+03
1	9	-138.580e+03	-35.347e+03	-2.287e+06	-522.292e+03
1	10	-155.035e+03	-41.674e+03	-2.770e+06	-827.198e+03
2	0	54.204e+03	185.499e+03	-2.770e+06	-827.198e+03
2	1	42.196e+03	161.226e+03	-1.232e+06	-37.525e+03
2	2	24.394e+03	133.138e+03	-137.640e+03	949.985e+03
2	3	2.978e+03	104.406e+03	264.057e+03	1.889e+06
2	4	-21.466e+03	75.765e+03	505.307e+03	2.482e+06
2	5	-47.909e+03	47.909e+03	610.835e+03	2.668e+06
2	6	-75.765e+03	21.466e+03	505.307e+03	2.482e+06
2	7	-104.406e+03	-2.978e+03	264.057e+03	1.889e+06
2	8	-133.138e+03	-24.394e+03	-137.640e+03	949.985e+03
2	9	-161.226e+03	-42.196e+03	-1.232e+06	-37.525e+03
2	10	-187.894e+03	-54.204e+03	-2.770e+06	-827.198e+03
3	0	41.674e+03	153.856e+03	-2.770e+06	-827.198e+03
3	1	35.347e+03	138.580e+03	-2.287e+06	-522.292e+03
3	2	25.121e+03	121.649e+03	-1.923e+06	-69.626e+03
3	3	13.835e+03	104.363e+03	-1.592e+06	314.509e+03
3	4	1.749e+03	86.858e+03	-1.287e+06	595.824e+03
3	5	-11.277e+03	69.944e+03	-1.008e+06	757.897e+03
3	6	-26.772e+03	54.827e+03	-755.475e+03	847.906e+03
3	7	-43.559e+03	43.985e+03	-527.972e+03	827.156e+03
3	8	-61.197e+03	37.971e+03	-326.226e+03	697.819e+03
3	9	-79.593e+03	32.329e+03	-150.235e+03	424.061e+03
3	10	-98.646e+03	27.065e+03	0.000e+00	0.000e+00

Service I Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-27.065e+03	98.646e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	95.879e+03	287.881e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	95.879e+03	287.881e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-27.065e+03	98.646e+03	0.000e+00	0.000e+00

Service II Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-37.188e+03	126.237e+03	0.000e+00	0.000e+00
1	1	-42.509e+03	102.991e+03	-201.608e+03	544.978e+03
1	2	-48.320e+03	80.598e+03	-428.971e+03	902.287e+03
1	3	-54.616e+03	59.192e+03	-682.091e+03	1.079e+06
1	4	-67.189e+03	38.891e+03	-960.965e+03	1.123e+06
1	5	-85.317e+03	20.269e+03	-1.265e+06	1.031e+06
1	6	-105.784e+03	4.857e+03	-1.595e+06	852.659e+03
1	7	-127.018e+03	-9.332e+03	-1.952e+06	527.007e+03
1	8	-147.966e+03	-22.481e+03	-2.334e+06	75.415e+03
1	9	-168.454e+03	-34.252e+03	-2.751e+06	-457.538e+03
1	10	-188.324e+03	-40.954e+03	-3.317e+06	-790.678e+03
2	0	52.991e+03	223.674e+03	-3.317e+06	-790.678e+03
2	1	40.875e+03	195.614e+03	-1.500e+06	52.671e+03
2	2	21.227e+03	162.594e+03	-219.986e+03	1.193e+06
2	3	-3.117e+03	128.738e+03	200.428e+03	2.312e+06
2	4	-31.401e+03	95.000e+03	452.978e+03	3.023e+06

2	5	-62.282e+03	62.282e+03	569.806e+03	3.245e+06
2	6	-95.000e+03	31.401e+03	452.978e+03	3.023e+06
2	7	-128.738e+03	3.117e+03	200.428e+03	2.312e+06
2	8	-162.594e+03	-21.227e+03	-219.986e+03	1.193e+06
2	9	-195.614e+03	-40.875e+03	-1.500e+06	52.671e+03
2	10	-226.787e+03	-52.991e+03	-3.317e+06	-790.678e+03
3	0	40.954e+03	186.791e+03	-3.317e+06	-790.678e+03
3	1	34.252e+03	168.454e+03	-2.751e+06	-457.538e+03
3	2	22.481e+03	147.966e+03	-2.334e+06	75.415e+03
3	3	9.332e+03	127.018e+03	-1.952e+06	527.007e+03
3	4	-4.857e+03	105.784e+03	-1.595e+06	852.659e+03
3	5	-20.269e+03	85.317e+03	-1.265e+06	1.031e+06
3	6	-38.891e+03	67.189e+03	-960.965e+03	1.123e+06
3	7	-59.192e+03	54.616e+03	-682.091e+03	1.079e+06
3	8	-80.598e+03	48.320e+03	-428.971e+03	902.287e+03
3	9	-102.991e+03	42.509e+03	-201.608e+03	544.978e+03
3	10	-126.237e+03	37.188e+03	0.000e+00	0.000e+00

Service II Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-37.188e+03	126.237e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	93.946e+03	343.548e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	93.946e+03	343.548e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-37.188e+03	126.237e+03	0.000e+00	0.000e+00

Service III Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-20.317e+03	80.252e+03	0.000e+00	0.000e+00
1	1	-25.543e+03	63.995e+03	-115.986e+03	343.450e+03
1	2	-31.071e+03	48.263e+03	-257.729e+03	561.506e+03
1	3	-36.897e+03	33.138e+03	-425.227e+03	658.875e+03
1	4	-46.586e+03	18.693e+03	-618.481e+03	664.224e+03
1	5	-59.694e+03	5.281e+03	-837.490e+03	575.813e+03
1	6	-74.241e+03	-6.154e+03	-1.082e+06	424.601e+03
1	7	-89.260e+03	-16.838e+03	-1.352e+06	172.843e+03
1	8	-104.103e+03	-26.882e+03	-1.649e+06	-166.321e+03
1	9	-118.663e+03	-36.077e+03	-1.977e+06	-565.462e+03
1	10	-132.843e+03	-42.154e+03	-2.406e+06	-851.545e+03
2	0	55.013e+03	160.049e+03	-2.406e+06	-851.545e+03
2	1	43.077e+03	138.301e+03	-1.053e+06	-97.656e+03
2	2	26.505e+03	113.500e+03	-82.743e+03	787.357e+03
2	3	7.042e+03	88.185e+03	306.476e+03	1.606e+06
2	4	-14.843e+03	62.942e+03	540.193e+03	2.122e+06
2	5	-38.327e+03	38.327e+03	638.187e+03	2.284e+06
2	6	-62.942e+03	14.843e+03	540.193e+03	2.122e+06
2	7	-88.185e+03	-7.042e+03	306.476e+03	1.606e+06
2	8	-113.500e+03	-26.505e+03	-82.743e+03	787.357e+03
2	9	-138.301e+03	-43.077e+03	-1.053e+06	-97.656e+03
2	10	-161.965e+03	-55.013e+03	-2.406e+06	-851.545e+03
3	0	42.154e+03	131.899e+03	-2.406e+06	-851.545e+03
3	1	36.077e+03	118.663e+03	-1.977e+06	-565.462e+03
3	2	26.882e+03	104.103e+03	-1.649e+06	-166.321e+03
3	3	16.838e+03	89.260e+03	-1.352e+06	172.843e+03
3	4	6.154e+03	74.241e+03	-1.082e+06	424.601e+03
3	5	-5.281e+03	59.694e+03	-837.490e+03	575.813e+03
3	6	-18.693e+03	46.586e+03	-618.481e+03	664.224e+03
3	7	-33.138e+03	36.897e+03	-425.227e+03	658.875e+03
3	8	-48.263e+03	31.071e+03	-257.729e+03	561.506e+03
3	9	-63.995e+03	25.543e+03	-115.986e+03	343.450e+03
3	10	-80.252e+03	20.317e+03	0.000e+00	0.000e+00

Service III Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-20.317e+03	80.252e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	97.168e+03	250.769e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	97.168e+03	250.769e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-20.317e+03	80.252e+03	0.000e+00	0.000e+00

Fatigue Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

QConBridge
Run 2 Output

Dual Truck Axle Rxn = 94.589 k

No Impact
No Lane

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Washington State Department of Transporation
Bridge and Structures Office
QConBridge Version 1.0


Code: LRFD First Edition 1994

Span Data
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Span  1  Length:     50.750 ft

Section Properties
Location      Ax           Iz          Mod. E      Unit Wgt
  (ft)      (in^2)       (in^4)        (psi)       (pcf)
   0.000  1.000e+00    999.999e-03  1.000e+03    999.997e-03

Live Load Distribution Factors
Location  Str/Serv Limit States  Fatigue Limit State
  (ft)         gM        gV           gM        gV
   0.000       1.000     1.000       1.000     1.000

Strength Limit State Factors: Ductility 1.00  Redundancy 1.00  Importance 1.00
Service  Limit State Factors: Ductility 1.00  Redundancy 1.00  Importance 1.00


Span  2  Length:    116.500 ft

Section Properties
Location      Ax           Iz          Mod. E      Unit Wgt
  (ft)      (in^2)       (in^4)        (psi)       (pcf)
   0.000  1.000e+00    999.999e-03  1.000e+03    999.997e-03

Live Load Distribution Factors
Location  Str/Serv Limit States  Fatigue Limit State
  (ft)         gM        gV           gM        gV
   0.000       1.000     1.000       1.000     1.000

Strength Limit State Factors: Ductility 1.00  Redundancy 1.00  Importance 1.00
Service  Limit State Factors: Ductility 1.00  Redundancy 1.00  Importance 1.00


Span  3  Length:     50.750 ft

Section Properties
Location      Ax           Iz          Mod. E      Unit Wgt
  (ft)      (in^2)       (in^4)        (psi)       (pcf)
   0.000  1.000e+00    999.999e-03  1.000e+03    999.997e-03

Live Load Distribution Factors
Location  Str/Serv Limit States  Fatigue Limit State
  (ft)         gM        gV           gM        gV
   0.000       1.000     1.000       1.000     1.000

Strength Limit State Factors: Ductility 1.00  Redundancy 1.00  Importance 1.00
Service  Limit State Factors: Ductility 1.00  Redundancy 1.00  Importance 1.00


Support Data
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Support  1  Roller
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Support  2  Pinned


Support  3  Pinned


Support  4  Roller


Loading Data
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DC Loads
   Self Weight Generation Disabled
   Traffic Barrier Load 999.999e+00 plf

DW Loads
   Utility Load Disabled
   Wearing Surface Load Disabled

Live Load Data
--------------
Live Load Generation Parameters
   Design Tandem    : Enabled
   Design Truck     : 17 rear axle spacing increments
   Dual Truck Train : Headway Spacing varies from 50.000 ft to 218.000 ft using 168 increments
   Dual Tandem Train: Disabled
   Fatigue Truck    : Disabled

Live Load Impact
   Truck Loads    0.000%
   Lane  Loads   -100.000%
   Fatigue Truck 15.000%
```

Impact

```
   Pedestrian Live Load    0.000e+00 plf

Load Factors
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   Strength I   DC min 0.900 DC max 1.250 DW min 0.650 DW max 1.500 LL     1.750
   Service I    DC     1.000 DW     1.000 LL     1.000
   Service II   DC     1.000 DW     1.000 LL     1.300
   Service III  DC     1.000 DW     1.000 LL     0.800
   Fatigue      DC     0.000 DW     0.000 LL     0.750

Analysis Results
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DC Dead Load
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Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	6.676e+03	0.000e+00
1	1	1.601e+03	21.006e+03
1	2	-3.473e+03	16.258e+03
1	3	-8.548e+03	-14.246e+03
1	4	-13.623e+03	-70.506e+03
1	5	-18.698e+03	-152.521e+03
1	6	-23.773e+03	-260.292e+03
1	7	-28.848e+03	-393.819e+03
1	8	-33.923e+03	-553.102e+03
1	9	-38.998e+03	-738.140e+03
1	10	-44.073e+03	-948.933e+03
2	0	58.249e+03	-948.933e+03
2	1	46.599e+03	-338.182e+03
2	2	34.949e+03	136.846e+03
2	3	23.300e+03	476.152e+03

2	4	11.650e+03	679.735e+03
2	5	0.000e+00	747.597e+03
2	6	-11.650e+03	679.735e+03
2	7	-23.300e+03	476.152e+03
2	8	-34.949e+03	136.846e+03
2	9	-46.599e+03	-338.182e+03
2	10	-58.249e+03	-948.933e+03
3	0	44.073e+03	-948.933e+03
3	1	38.998e+03	-738.140e+03
3	2	33.923e+03	-553.102e+03
3	3	28.848e+03	-393.819e+03
3	4	23.773e+03	-260.292e+03
3	5	18.698e+03	-152.521e+03
3	6	13.623e+03	-70.506e+03
3	7	8.548e+03	-14.246e+03
3	8	3.473e+03	16.258e+03
3	9	-1.601e+03	21.006e+03
3	10	-6.676e+03	0.000e+00

DC Dead Load

Pier	Fx(lbs)	Fy(lbs)	Mz(ft-lbs)
1	0.000e+00	6.676e+03	0.000e+00
2	0.000e+00	102.323e+03	0.000e+00
3	0.000e+00	102.323e+03	0.000e+00
4	0.000e+00	6.676e+03	0.000e+00

DW Dead Load

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00

DW Dead Load

Pier	Fx(lbs)	Fy(lbs)	Mz(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00

Live Load Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-17.167e+03	57.669e+03	0.000e+00	0.000e+00
1	1	-17.167e+03	49.449e+03	-87.126e+03	250.954e+03
1	2	-17.167e+03	41.438e+03	-174.253e+03	420.604e+03
1	3	-17.167e+03	33.719e+03	-261.379e+03	513.372e+03
1	4	-20.525e+03	26.365e+03	-348.506e+03	556.203e+03
1	5	-26.831e+03	19.718e+03	-435.632e+03	547.460e+03
1	6	-34.236e+03	14.675e+03	-522.759e+03	516.240e+03
1	7	-41.846e+03	10.098e+03	-609.885e+03	427.070e+03
1	8	-49.068e+03	5.901e+03	-697.012e+03	292.296e+03
1	9	-55.819e+03	2.297e+03	-784.138e+03	131.764e+03
1	10	-62.030e+03	1.441e+03	-947.507e+03	73.156e+03
2	0	-2.430e+03	67.155e+03	-947.507e+03	73.156e+03
2	1	-2.528e+03	63.075e+03	-497.935e+03	190.269e+03
2	2	-6.573e+03	55.717e+03	-153.408e+03	488.826e+03
2	3	-12.789e+03	47.328e+03	-125.087e+03	795.763e+03
2	4	-20.653e+03	38.378e+03	-96.767e+03	991.611e+03
2	5	-29.335e+03	29.335e+03	-68.446e+03	1.047e+06
2	6	-38.378e+03	20.653e+03	-96.767e+03	991.611e+03
2	7	-47.328e+03	12.789e+03	-125.087e+03	795.763e+03
2	8	-55.717e+03	6.573e+03	-153.408e+03	488.826e+03
2	9	-63.075e+03	2.528e+03	-497.935e+03	190.269e+03
2	10	-68.956e+03	2.430e+03	-947.507e+03	73.156e+03
3	0	-1.441e+03	61.143e+03	-947.507e+03	73.156e+03
3	1	-2.297e+03	55.819e+03	-784.138e+03	131.764e+03
3	2	-5.901e+03	49.068e+03	-697.012e+03	292.296e+03
3	3	-10.098e+03	41.846e+03	-609.885e+03	427.070e+03
3	4	-14.675e+03	34.236e+03	-522.759e+03	516.240e+03
3	5	-19.718e+03	26.831e+03	-435.632e+03	547.460e+03
3	6	-26.365e+03	20.525e+03	-348.506e+03	556.203e+03
3	7	-33.719e+03	17.167e+03	-261.379e+03	513.372e+03
3	8	-41.438e+03	17.167e+03	-174.253e+03	420.604e+03
3	9	-49.449e+03	17.167e+03	-87.126e+03	250.954e+03
3	10	-57.669e+03	17.167e+03	0.000e+00	0.000e+00

Not interested in the overall envelope.

Live Load Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-17.167e+03	57.669e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-3.872e+03	94.589e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-3.872e+03	94.589e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-17.167e+03	57.669e+03	0.000e+00	0.000e+00

Design Tandem + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-11.975e+03	47.211e+03	0.000e+00	0.000e+00
1	1	-11.975e+03	41.440e+03	-60.776e+03	210.309e+03
1	2	-11.975e+03	35.736e+03	-121.552e+03	362.722e+03
1	3	-14.985e+03	30.162e+03	-182.328e+03	459.219e+03
1	4	-20.525e+03	24.757e+03	-243.104e+03	502.585e+03
1	5	-25.883e+03	19.578e+03	-303.880e+03	500.837e+03
1	6	-31.005e+03	14.675e+03	-364.657e+03	464.511e+03
1	7	-35.840e+03	10.098e+03	-425.433e+03	386.411e+03
1	8	-40.335e+03	5.901e+03	-486.209e+03	272.996e+03

1	9	-44.441e+03	2.133e+03	-546.985e+03	131.764e+03
1	10	-48.105e+03	1.131e+03	-607.761e+03	57.433e+03
2	0	-1.908e+03	46.929e+03	-607.761e+03	57.433e+03
2	1	-2.528e+03	45.713e+03	-266.528e+03	190.269e+03
2	2	-6.573e+03	41.214e+03	-120.438e+03	439.547e+03
2	3	-11.727e+03	35.844e+03	-98.204e+03	652.643e+03
2	4	-17.518e+03	29.893e+03	-75.970e+03	792.032e+03
2	5	-23.678e+03	23.678e+03	-53.736e+03	837.417e+03
2	6	-29.893e+03	17.518e+03	-75.970e+03	792.032e+03
2	7	-35.844e+03	11.727e+03	-98.204e+03	652.643e+03
2	8	-41.214e+03	6.573e+03	-120.438e+03	439.547e+03
2	9	-45.713e+03	2.528e+03	-266.528e+03	190.269e+03
2	10	-48.943e+03	1.908e+03	-607.761e+03	57.433e+03
3	0	-1.131e+03	47.324e+03	-607.761e+03	57.433e+03
3	1	-2.133e+03	44.441e+03	-546.985e+03	131.764e+03
3	2	-5.901e+03	40.335e+03	-486.209e+03	272.996e+03
3	3	-10.098e+03	35.840e+03	-425.433e+03	386.411e+03
3	4	-14.675e+03	31.005e+03	-364.657e+03	464.511e+03
3	5	-19.578e+03	25.883e+03	-303.880e+03	500.837e+03
3	6	-24.757e+03	20.525e+03	-243.104e+03	502.585e+03
3	7	-30.162e+03	14.985e+03	-182.328e+03	459.219e+03
3	8	-35.736e+03	11.975e+03	-121.552e+03	362.722e+03
3	9	-41.440e+03	11.975e+03	-60.776e+03	210.309e+03
3	10	-47.211e+03	11.975e+03	0.000e+00	0.000e+00

Design Tandem + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-11.975e+03	47.211e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-3.040e+03	52.551e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-3.040e+03	52.551e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-11.975e+03	47.211e+03	0.000e+00	0.000e+00

Design Truck + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-17.167e+03	57.669e+03	0.000e+00	0.000e+00
1	1	-17.167e+03	49.449e+03	-87.126e+03	250.954e+03
1	2	-17.167e+03	41.438e+03	-174.253e+03	420.604e+03
1	3	-17.167e+03	33.719e+03	-261.379e+03	513.372e+03
1	4	-19.579e+03	26.365e+03	-348.506e+03	556.203e+03
1	5	-26.831e+03	19.718e+03	-435.632e+03	547.460e+03
1	6	-34.236e+03	13.747e+03	-522.759e+03	516.240e+03
1	7	-41.846e+03	8.275e+03	-609.885e+03	427.070e+03
1	8	-49.068e+03	4.898e+03	-697.012e+03	292.296e+03
1	9	-55.819e+03	2.297e+03	-784.138e+03	110.641e+03
1	10	-62.030e+03	1.441e+03	-871.265e+03	73.156e+03
2	0	-2.430e+03	67.155e+03	-871.265e+03	73.156e+03
2	1	-2.430e+03	63.075e+03	-374.500e+03	150.903e+03
2	2	-6.429e+03	55.717e+03	-153.408e+03	488.826e+03
2	3	-12.789e+03	47.328e+03	-125.087e+03	795.763e+03
2	4	-20.653e+03	38.378e+03	-96.767e+03	991.611e+03
2	5	-29.335e+03	29.335e+03	-68.446e+03	1.047e+06
2	6	-38.378e+03	20.653e+03	-96.767e+03	991.611e+03
2	7	-47.328e+03	12.789e+03	-125.087e+03	795.763e+03
2	8	-55.717e+03	6.429e+03	-153.408e+03	488.826e+03
2	9	-63.075e+03	2.430e+03	-374.500e+03	150.903e+03
2	10	-68.956e+03	2.430e+03	-871.265e+03	73.156e+03
3	0	-1.441e+03	61.143e+03	-871.265e+03	73.156e+03
3	1	-2.297e+03	55.819e+03	-784.138e+03	110.641e+03
3	2	-4.898e+03	49.068e+03	-697.012e+03	292.296e+03
3	3	-8.275e+03	41.846e+03	-609.885e+03	427.070e+03
3	4	-13.747e+03	34.236e+03	-522.759e+03	516.240e+03
3	5	-19.718e+03	26.831e+03	-435.632e+03	547.460e+03

3	6	-26.365e+03	19.579e+03	-348.506e+03	556.203e+03
3	7	-33.719e+03	17.167e+03	-261.379e+03	513.372e+03
3	8	-41.438e+03	17.167e+03	-174.253e+03	420.604e+03
3	9	-49.449e+03	17.167e+03	-87.126e+03	250.954e+03
3	10	-57.669e+03	17.167e+03	0.000e+00	0.000e+00

Design Truck + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-17.167e+03	57.669e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-3.872e+03	76.138e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-3.872e+03	76.138e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-17.167e+03	57.669e+03	0.000e+00	0.000e+00

Dual Truck Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	-247.164e+03	0.000e+00
1	4	0.000e+00	0.000e+00	-329.553e+03	0.000e+00
1	5	0.000e+00	0.000e+00	-411.941e+03	0.000e+00
1	6	0.000e+00	0.000e+00	-494.329e+03	0.000e+00
1	7	0.000e+00	0.000e+00	-576.718e+03	0.000e+00
1	8	0.000e+00	0.000e+00	-659.106e+03	0.000e+00
1	9	0.000e+00	0.000e+00	-741.494e+03	0.000e+00
1	10	0.000e+00	0.000e+00	-947.507e+03	0.000e+00
2	0	0.000e+00	0.000e+00	-947.507e+03	0.000e+00
2	1	0.000e+00	0.000e+00	-497.935e+03	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	-497.935e+03	0.000e+00
2	10	0.000e+00	0.000e+00	-947.507e+03	0.000e+00
3	0	0.000e+00	0.000e+00	-947.507e+03	0.000e+00
3	1	0.000e+00	0.000e+00	-741.494e+03	0.000e+00
3	2	0.000e+00	0.000e+00	-659.106e+03	0.000e+00
3	3	0.000e+00	0.000e+00	-576.718e+03	0.000e+00
3	4	0.000e+00	0.000e+00	-494.329e+03	0.000e+00
3	5	0.000e+00	0.000e+00	-411.941e+03	0.000e+00
3	6	0.000e+00	0.000e+00	-329.553e+03	0.000e+00
3	7	0.000e+00	0.000e+00	-247.164e+03	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tuck Axle Rxn – No Impact and No Lane

Dual Truck Train + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-3.485e+03	94.589e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-3.485e+03	94.589e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tandem Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00

1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tandem Train + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Truck Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Truck Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Strength I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-24.034e+03	109.267e+03	0.000e+00	0.000e+00
1	1	-28.602e+03	88.538e+03	-133.565e+03	465.429e+03
1	2	-34.385e+03	69.392e+03	-290.310e+03	756.380e+03
1	3	-40.728e+03	51.314e+03	-475.221e+03	885.581e+03
1	4	-52.948e+03	33.878e+03	-698.018e+03	909.900e+03
1	5	-70.328e+03	17.679e+03	-953.009e+03	820.786e+03
1	6	-89.631e+03	4.285e+03	-1.240e+06	669.157e+03
1	7	-109.292e+03	-8.290e+03	-1.559e+06	392.935e+03
1	8	-128.273e+03	-20.203e+03	-1.911e+06	13.727e+03
1	9	-146.431e+03	-31.078e+03	-2.294e+06	-433.737e+03
1	10	-163.645e+03	-37.143e+03	-2.844e+06	-726.016e+03
2	0	48.170e+03	190.335e+03	-2.844e+06	-726.016e+03
2	1	37.514e+03	168.632e+03	-1.294e+06	28.608e+03
2	2	19.950e+03	141.192e+03	-145.303e+03	1.026e+06
2	3	-1.411e+03	111.950e+03	209.633e+03	1.987e+06
2	4	-25.657e+03	81.725e+03	442.419e+03	2.584e+06
2	5	-51.337e+03	51.337e+03	553.055e+03	2.768e+06
2	6	-81.725e+03	25.657e+03	442.419e+03	2.584e+06
2	7	-111.950e+03	1.411e+03	209.633e+03	1.987e+06
2	8	-141.192e+03	-19.950e+03	-145.303e+03	1.026e+06
2	9	-168.632e+03	-37.514e+03	-1.294e+06	28.608e+03
2	10	-193.486e+03	-48.170e+03	-2.844e+06	-726.016e+03
3	0	37.143e+03	162.093e+03	-2.844e+06	-726.016e+03
3	1	31.078e+03	146.431e+03	-2.294e+06	-433.737e+03
3	2	20.203e+03	128.273e+03	-1.911e+06	13.727e+03
3	3	8.290e+03	109.292e+03	-1.559e+06	392.935e+03
3	4	-4.285e+03	89.631e+03	-1.240e+06	669.157e+03
3	5	-17.679e+03	70.328e+03	-953.009e+03	820.786e+03
3	6	-33.878e+03	52.948e+03	-698.018e+03	909.900e+03
3	7	-51.314e+03	40.728e+03	-475.221e+03	885.581e+03
3	8	-69.392e+03	34.385e+03	-290.310e+03	756.380e+03
3	9	-88.538e+03	28.602e+03	-133.565e+03	465.429e+03
3	10	-109.267e+03	24.034e+03	0.000e+00	0.000e+00

Strength I Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-24.034e+03	109.267e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	85.314e+03	293.434e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	85.314e+03	293.434e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-24.034e+03	109.267e+03	0.000e+00	0.000e+00

Service I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-10.490e+03	64.346e+03	0.000e+00	0.000e+00
1	1	-15.565e+03	51.051e+03	-66.119e+03	271.961e+03
1	2	-20.640e+03	37.965e+03	-157.994e+03	436.862e+03
1	3	-25.715e+03	25.170e+03	-275.625e+03	499.126e+03
1	4	-34.148e+03	12.742e+03	-419.012e+03	485.697e+03
1	5	-45.530e+03	1.020e+03	-588.154e+03	394.939e+03
1	6	-58.010e+03	-9.098e+03	-783.052e+03	255.947e+03
1	7	-70.695e+03	-18.749e+03	-1.003e+06	33.250e+03
1	8	-82.991e+03	-28.021e+03	-1.250e+06	-260.805e+03
1	9	-94.817e+03	-36.700e+03	-1.522e+06	-606.375e+03
1	10	-106.103e+03	-42.631e+03	-1.896e+06	-875.777e+03
2	0	55.819e+03	125.405e+03	-1.896e+06	-875.777e+03
2	1	44.071e+03	109.675e+03	-836.118e+03	-147.912e+03
2	2	28.376e+03	90.667e+03	-16.562e+03	625.671e+03
2	3	10.510e+03	70.628e+03	351.064e+03	1.271e+06
2	4	-9.003e+03	50.028e+03	582.968e+03	1.671e+06
2	5	-29.335e+03	29.335e+03	679.150e+03	1.795e+06
2	6	-50.028e+03	9.003e+03	582.968e+03	1.671e+06
2	7	-70.628e+03	-10.510e+03	351.064e+03	1.271e+06
2	8	-90.667e+03	-28.376e+03	-16.562e+03	625.671e+03
2	9	-109.675e+03	-44.071e+03	-836.118e+03	-147.912e+03
2	10	-127.206e+03	-55.819e+03	-1.896e+06	-875.777e+03
3	0	42.631e+03	105.217e+03	-1.896e+06	-875.777e+03
3	1	36.700e+03	94.817e+03	-1.522e+06	-606.375e+03
3	2	28.021e+03	82.991e+03	-1.250e+06	-260.805e+03
3	3	18.749e+03	70.695e+03	-1.003e+06	33.250e+03
3	4	9.098e+03	58.010e+03	-783.052e+03	255.947e+03
3	5	-1.020e+03	45.530e+03	-588.154e+03	394.939e+03
3	6	-12.742e+03	34.148e+03	-419.012e+03	485.697e+03
3	7	-25.170e+03	25.715e+03	-275.625e+03	499.126e+03
3	8	-37.965e+03	20.640e+03	-157.994e+03	436.862e+03
3	9	-51.051e+03	15.565e+03	-66.119e+03	271.961e+03
3	10	-64.346e+03	10.490e+03	0.000e+00	0.000e+00

Service I Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-10.490e+03	64.346e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	98.450e+03	196.912e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	98.450e+03	196.912e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-10.490e+03	64.346e+03	0.000e+00	0.000e+00

Service II Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-15.641e+03	81.647e+03	0.000e+00	0.000e+00
1	1	-20.716e+03	65.885e+03	-92.257e+03	347.248e+03
1	2	-25.791e+03	50.397e+03	-210.270e+03	563.043e+03
1	3	-30.866e+03	35.286e+03	-354.039e+03	653.138e+03
1	4	-40.306e+03	20.652e+03	-523.564e+03	652.558e+03
1	5	-53.579e+03	6.936e+03	-718.844e+03	559.177e+03
1	6	-68.281e+03	-4.695e+03	-939.879e+03	410.820e+03
1	7	-83.249e+03	-15.719e+03	-1.186e+06	161.371e+03
1	8	-97.712e+03	-26.251e+03	-1.459e+06	-173.116e+03
1	9	-111.563e+03	-36.011e+03	-1.757e+06	-566.845e+03
1	10	-124.712e+03	-42.199e+03	-2.180e+06	-853.830e+03
2	0	55.089e+03	145.552e+03	-2.180e+06	-853.830e+03
2	1	43.312e+03	128.598e+03	-985.498e+03	-90.831e+03
2	2	26.403e+03	107.382e+03	-62.584e+03	772.319e+03
2	3	6.673e+03	84.827e+03	313.538e+03	1.510e+06
2	4	-15.199e+03	61.542e+03	553.938e+03	1.968e+06

2	5	-38.136e+03	38.136e+03	658.616e+03	2.109e+06
2	6	-61.542e+03	15.199e+03	553.938e+03	1.968e+06
2	7	-84.827e+03	-6.673e+03	313.538e+03	1.510e+06
2	8	-107.382e+03	-26.403e+03	-62.584e+03	772.319e+03
2	9	-128.598e+03	-43.312e+03	-985.498e+03	-90.831e+03
2	10	-147.893e+03	-55.089e+03	-2.180e+06	-853.830e+03
3	0	42.199e+03	123.560e+03	-2.180e+06	-853.830e+03
3	1	36.011e+03	111.563e+03	-1.757e+06	-566.845e+03
3	2	26.251e+03	97.712e+03	-1.459e+06	-173.116e+03
3	3	15.719e+03	83.249e+03	-1.186e+06	161.371e+03
3	4	4.695e+03	68.281e+03	-939.879e+03	410.820e+03
3	5	-6.936e+03	53.579e+03	-718.844e+03	559.177e+03
3	6	-20.652e+03	40.306e+03	-523.564e+03	652.558e+03
3	7	-35.286e+03	30.866e+03	-354.039e+03	653.138e+03
3	8	-50.397e+03	25.791e+03	-210.270e+03	563.043e+03
3	9	-65.885e+03	20.716e+03	-92.257e+03	347.248e+03
3	10	-81.647e+03	15.641e+03	0.000e+00	0.000e+00

Service II Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-15.641e+03	81.647e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	97.289e+03	225.288e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	97.289e+03	225.288e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-15.641e+03	81.647e+03	0.000e+00	0.000e+00

Service III Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-7.057e+03	52.812e+03	0.000e+00	0.000e+00
1	1	-12.132e+03	41.161e+03	-48.694e+03	221.770e+03
1	2	-17.207e+03	29.677e+03	-123.144e+03	352.741e+03
1	3	-22.282e+03	18.427e+03	-223.349e+03	396.452e+03
1	4	-30.043e+03	7.469e+03	-349.310e+03	374.456e+03
1	5	-40.163e+03	-2.923e+03	-501.027e+03	285.446e+03
1	6	-51.162e+03	-12.033e+03	-678.500e+03	152.699e+03
1	7	-62.325e+03	-20.769e+03	-881.728e+03	-52.163e+03
1	8	-73.177e+03	-29.202e+03	-1.110e+06	-319.264e+03
1	9	-83.653e+03	-37.160e+03	-1.365e+06	-632.728e+03
1	10	-93.697e+03	-42.920e+03	-1.706e+06	-890.408e+03
2	0	56.305e+03	111.974e+03	-1.706e+06	-890.408e+03
2	1	44.577e+03	97.060e+03	-736.531e+03	-185.966e+03
2	2	29.690e+03	79.523e+03	14.119e+03	527.906e+03
2	3	13.068e+03	61.163e+03	376.081e+03	1.112e+06
2	4	-4.872e+03	42.353e+03	602.322e+03	1.473e+06
2	5	-23.468e+03	23.468e+03	692.839e+03	1.585e+06
2	6	-42.353e+03	4.872e+03	602.322e+03	1.473e+06
2	7	-61.163e+03	-13.068e+03	376.081e+03	1.112e+06
2	8	-79.523e+03	-29.690e+03	14.119e+03	527.906e+03
2	9	-97.060e+03	-44.577e+03	-736.531e+03	-185.966e+03
2	10	-113.415e+03	-56.305e+03	-1.706e+06	-890.408e+03
3	0	42.920e+03	92.988e+03	-1.706e+06	-890.408e+03
3	1	37.160e+03	83.653e+03	-1.365e+06	-632.728e+03
3	2	29.202e+03	73.177e+03	-1.110e+06	-319.264e+03
3	3	20.769e+03	62.325e+03	-881.728e+03	-52.163e+03
3	4	12.033e+03	51.162e+03	-678.500e+03	152.699e+03
3	5	2.923e+03	40.163e+03	-501.027e+03	285.446e+03
3	6	-7.469e+03	30.043e+03	-349.310e+03	374.456e+03
3	7	-18.427e+03	22.282e+03	-223.349e+03	396.452e+03
3	8	-29.677e+03	17.207e+03	-123.144e+03	352.741e+03
3	9	-41.161e+03	12.132e+03	-48.694e+03	221.770e+03
3	10	-52.812e+03	7.057e+03	0.000e+00	0.000e+00

Service III Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-7.057e+03	52.812e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	99.225e+03	177.994e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	99.225e+03	177.994e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-7.057e+03	52.812e+03	0.000e+00	0.000e+00

Fatigue Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Washington State Department of Transporation
Bridge and Structures Office
QConBridge Version 1.0

QConBridge
Run 3 Output

Dual Lane Axle Rxn = 59.754 k

No Impact
No Truck

Code: LRFD First Edition 1994

Span Data

Span 1 Length: 50.750 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	999.999e-03	1.000e+03	999.997e-03

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Span 2 Length: 116.500 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	999.999e-03	1.000e+03	999.997e-03

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Span 3 Length: 50.750 ft

Section Properties

Location (ft)	Ax (in^2)	Iz (in^4)	Mod. E (psi)	Unit Wgt (pcf)
0.000	1.000e+00	999.999e-03	1.000e+03	999.997e-03

Live Load Distribution Factors

Location	Str/Serv Limit States		Fatigue Limit State	
(ft)	gM	gV	gM	gV
0.000	1.000	1.000	1.000	1.000

Strength Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00
Service Limit State Factors: Ductility 1.00 Redundancy 1.00 Importance 1.00

Support Data

Support 1 Roller

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Support  2  Pinned


Support  3  Pinned


Support  4  Roller


Loading Data
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DC Loads
   Self Weight Generation Disabled
   Traffic Barrier Load 999.999e+00 plf

DW Loads
   Utility Load Disabled
   Wearing Surface Load Disabled

Live Load Data
--------------
Live Load Generation Parameters
   Design Tandem    : Enabled
   Design Truck     : 17 rear axle spacing increments
   Dual Truck Train : Headway Spacing varies from 50.000 ft to 218.000 ft using 168 increments
   Dual Tandem Train: Disabled
   Fatigue Truck    : Disabled

Live Load Impact
   Truck Loads   -100.000%
   Lane  Loads    0.000%
   Fatigue Truck 15.000%

   Pedestrian Live Load    0.000e+00 plf
```

Impact

```
Load Factors
------------
   Strength I   DC min 0.900 DC max 1.250 DW min 0.650 DW max 1.500 LL     1.750
   Service I    DC     1.000 DW     1.000 LL     1.000
   Service II   DC     1.000 DW     1.000 LL     1.300
   Service III  DC     1.000 DW     1.000 LL     0.800
   Fatigue      DC     0.000 DW     0.000 LL     0.750

Analysis Results
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DC Dead Load
```

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	6.676e+03	0.000e+00
1	1	1.601e+03	21.006e+03
1	2	-3.473e+03	16.258e+03
1	3	-8.548e+03	-14.246e+03
1	4	-13.623e+03	-70.506e+03
1	5	-18.698e+03	-152.521e+03
1	6	-23.773e+03	-260.292e+03
1	7	-28.848e+03	-393.819e+03
1	8	-33.923e+03	-553.102e+03
1	9	-38.998e+03	-738.140e+03
1	10	-44.073e+03	-948.933e+03
2	0	58.249e+03	-948.933e+03
2	1	46.599e+03	-338.182e+03
2	2	34.949e+03	136.846e+03
2	3	23.300e+03	476.152e+03

2	4	11.650e+03	679.735e+03
2	5	0.000e+00	747.597e+03
2	6	-11.650e+03	679.735e+03
2	7	-23.300e+03	476.152e+03
2	8	-34.949e+03	136.846e+03
2	9	-46.599e+03	-338.182e+03
2	10	-58.249e+03	-948.933e+03
3	0	44.073e+03	-948.933e+03
3	1	38.998e+03	-738.140e+03
3	2	33.923e+03	-553.102e+03
3	3	28.848e+03	-393.819e+03
3	4	23.773e+03	-260.292e+03
3	5	18.698e+03	-152.521e+03
3	6	13.623e+03	-70.506e+03
3	7	8.548e+03	-14.246e+03
3	8	3.473e+03	16.258e+03
3	9	-1.601e+03	21.006e+03
3	10	-6.676e+03	0.000e+00

DC Dead Load

Pier	Fx(lbs)	Fy(lbs)	Mz(ft-lbs)
1	0.000e+00	6.676e+03	0.000e+00
2	0.000e+00	102.323e+03	0.000e+00
3	0.000e+00	102.323e+03	0.000e+00
4	0.000e+00	6.676e+03	0.000e+00

DW Dead Load

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00

DW Dead Load

Pier	Fx(lbs)	Fy(lbs)	Mz(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00

Live Load Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-10.909e+03	15.269e+03	0.000e+00	0.000e+00
1	1	-11.098e+03	12.224e+03	-55.363e+03	69.284e+03
1	2	-11.664e+03	9.556e+03	-110.728e+03	122.157e+03
1	3	-12.603e+03	7.261e+03	-166.092e+03	158.616e+03
1	4	-13.905e+03	5.329e+03	-221.456e+03	178.662e+03
1	5	-15.559e+03	3.749e+03	-276.820e+03	182.296e+03
1	6	-17.550e+03	2.505e+03	-332.184e+03	169.517e+03
1	7	-19.859e+03	1.580e+03	-387.548e+03	140.325e+03
1	8	-22.464e+03	952.436e+00	-442.912e+03	94.720e+03
1	9	-25.341e+03	595.481e+00	-506.173e+03	40.600e+03
1	10	-28.462e+03	481.522e+00	-623.805e+03	24.437e+03
2	0	-812.039e+00	37.931e+03	-623.805e+03	24.437e+03
2	1	-1.040e+03	30.736e+03	-257.763e+03	47.598e+03
2	2	-1.812e+03	24.084e+03	-70.453e+03	163.001e+03
2	3	-3.311e+03	18.159e+03	-45.728e+03	354.499e+03
2	4	-5.647e+03	13.071e+03	-45.728e+03	484.234e+03
2	5	-8.893e+03	8.893e+03	-45.728e+03	527.478e+03
2	6	-13.071e+03	5.647e+03	-45.728e+03	484.234e+03
2	7	-18.159e+03	3.311e+03	-45.728e+03	354.499e+03
2	8	-24.084e+03	1.812e+03	-70.453e+03	163.001e+03
2	9	-30.736e+03	1.040e+03	-257.763e+03	47.598e+03
2	10	-37.931e+03	812.039e+00	-623.805e+03	24.437e+03
3	0	-481.522e+00	28.462e+03	-623.805e+03	24.437e+03
3	1	-595.481e+00	25.341e+03	-506.173e+03	40.600e+03
3	2	-952.436e+00	22.464e+03	-442.912e+03	94.720e+03
3	3	-1.580e+03	19.859e+03	-387.548e+03	140.325e+03
3	4	-2.505e+03	17.550e+03	-332.184e+03	169.517e+03
3	5	-3.749e+03	15.559e+03	-276.820e+03	182.296e+03
3	6	-5.329e+03	13.905e+03	-221.456e+03	178.662e+03
3	7	-7.261e+03	12.603e+03	-166.092e+03	158.616e+03
3	8	-9.556e+03	11.664e+03	-110.728e+03	122.157e+03
3	9	-12.224e+03	11.098e+03	-55.363e+03	69.284e+03
3	10	-15.269e+03	10.909e+03	0.000e+00	0.000e+00

Not interested in the overall envelope.

Live Load Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-10.909e+03	15.269e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-1.293e+03	66.394e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-1.293e+03	66.394e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-10.909e+03	15.269e+03	0.000e+00	0.000e+00

Design Tandem + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-10.909e+03	15.269e+03	0.000e+00	0.000e+00
1	1	-11.098e+03	12.224e+03	-55.363e+03	69.284e+03
1	2	-11.664e+03	9.556e+03	-110.728e+03	122.157e+03
1	3	-12.603e+03	7.261e+03	-166.092e+03	158.616e+03
1	4	-13.905e+03	5.329e+03	-221.456e+03	178.662e+03
1	5	-15.559e+03	3.749e+03	-276.820e+03	182.296e+03
1	6	-17.550e+03	2.505e+03	-332.184e+03	169.517e+03
1	7	-19.859e+03	1.580e+03	-387.548e+03	140.325e+03
1	8	-22.464e+03	952.436e+00	-442.912e+03	94.720e+03

1	9	-25.341e+03	595.481e+00	-506.173e+03	40.600e+03
1	10	-28.462e+03	481.522e+00	-623.805e+03	24.437e+03
2	0	-812.039e+00	37.931e+03	-623.805e+03	24.437e+03
2	1	-1.040e+03	30.736e+03	-257.763e+03	47.598e+03
2	2	-1.812e+03	24.084e+03	-70.453e+03	163.001e+03
2	3	-3.311e+03	18.159e+03	-45.728e+03	354.499e+03
2	4	-5.647e+03	13.071e+03	-45.728e+03	484.234e+03
2	5	-8.893e+03	8.893e+03	-45.728e+03	527.478e+03
2	6	-13.071e+03	5.647e+03	-45.728e+03	484.234e+03
2	7	-18.159e+03	3.311e+03	-45.728e+03	354.499e+03
2	8	-24.084e+03	1.812e+03	-70.453e+03	163.001e+03
2	9	-30.736e+03	1.040e+03	-257.763e+03	47.598e+03
2	10	-37.931e+03	812.039e+00	-623.805e+03	24.437e+03
3	0	-481.522e+00	28.462e+03	-623.805e+03	24.437e+03
3	1	-595.481e+00	25.341e+03	-506.173e+03	40.600e+03
3	2	-952.436e+00	22.464e+03	-442.912e+03	94.720e+03
3	3	-1.580e+03	19.859e+03	-387.548e+03	140.325e+03
3	4	-2.505e+03	17.550e+03	-332.184e+03	169.517e+03
3	5	-3.749e+03	15.559e+03	-276.820e+03	182.296e+03
3	6	-5.329e+03	13.905e+03	-221.456e+03	178.662e+03
3	7	-7.261e+03	12.603e+03	-166.092e+03	158.616e+03
3	8	-9.556e+03	11.664e+03	-110.728e+03	122.157e+03
3	9	-12.224e+03	11.098e+03	-55.363e+03	69.284e+03
3	10	-15.269e+03	10.909e+03	0.000e+00	0.000e+00

Design Tandem + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-10.909e+03	15.269e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-1.293e+03	66.394e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-1.293e+03	66.394e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-10.909e+03	15.269e+03	0.000e+00	0.000e+00

Design Truck + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-10.909e+03	15.269e+03	0.000e+00	0.000e+00
1	1	-11.098e+03	12.224e+03	-55.363e+03	69.284e+03
1	2	-11.664e+03	9.556e+03	-110.728e+03	122.157e+03
1	3	-12.603e+03	7.261e+03	-166.092e+03	158.616e+03
1	4	-13.905e+03	5.329e+03	-221.456e+03	178.662e+03
1	5	-15.559e+03	3.749e+03	-276.820e+03	182.296e+03
1	6	-17.550e+03	2.505e+03	-332.184e+03	169.517e+03
1	7	-19.859e+03	1.580e+03	-387.548e+03	140.325e+03
1	8	-22.464e+03	952.436e+00	-442.912e+03	94.720e+03
1	9	-25.341e+03	595.481e+00	-506.173e+03	40.600e+03
1	10	-28.462e+03	481.522e+00	-623.805e+03	24.437e+03
2	0	-812.039e+00	37.931e+03	-623.805e+03	24.437e+03
2	1	-1.040e+03	30.736e+03	-257.763e+03	47.598e+03
2	2	-1.812e+03	24.084e+03	-70.453e+03	163.001e+03
2	3	-3.311e+03	18.159e+03	-45.728e+03	354.499e+03
2	4	-5.647e+03	13.071e+03	-45.728e+03	484.234e+03
2	5	-8.893e+03	8.893e+03	-45.728e+03	527.478e+03
2	6	-13.071e+03	5.647e+03	-45.728e+03	484.234e+03
2	7	-18.159e+03	3.311e+03	-45.728e+03	354.499e+03
2	8	-24.084e+03	1.812e+03	-70.453e+03	163.001e+03
2	9	-30.736e+03	1.040e+03	-257.763e+03	47.598e+03
2	10	-37.931e+03	812.039e+00	-623.805e+03	24.437e+03
3	0	-481.522e+00	28.462e+03	-623.805e+03	24.437e+03
3	1	-595.481e+00	25.341e+03	-506.173e+03	40.600e+03
3	2	-952.436e+00	22.464e+03	-442.912e+03	94.720e+03
3	3	-1.580e+03	19.859e+03	-387.548e+03	140.325e+03
3	4	-2.505e+03	17.550e+03	-332.184e+03	169.517e+03
3	5	-3.749e+03	15.559e+03	-276.820e+03	182.296e+03

3	6	-5.329e+03	13.905e+03	-221.456e+03	178.662e+03
3	7	-7.261e+03	12.603e+03	-166.092e+03	158.616e+03
3	8	-9.556e+03	11.664e+03	-110.728e+03	122.157e+03
3	9	-12.224e+03	11.098e+03	-55.363e+03	69.284e+03
3	10	-15.269e+03	10.909e+03	0.000e+00	0.000e+00

Design Truck + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-10.909e+03	15.269e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-1.293e+03	66.394e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-1.293e+03	66.394e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-10.909e+03	15.269e+03	0.000e+00	0.000e+00

Dual Truck Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	-149.482e+03	0.000e+00
1	4	0.000e+00	0.000e+00	-199.310e+03	0.000e+00
1	5	0.000e+00	0.000e+00	-249.138e+03	0.000e+00
1	6	0.000e+00	0.000e+00	-298.965e+03	0.000e+00
1	7	0.000e+00	0.000e+00	-348.793e+03	0.000e+00
1	8	0.000e+00	0.000e+00	-398.620e+03	0.000e+00
1	9	0.000e+00	0.000e+00	-455.556e+03	0.000e+00
1	10	0.000e+00	0.000e+00	-561.424e+03	0.000e+00
2	0	0.000e+00	0.000e+00	-561.424e+03	0.000e+00
2	1	0.000e+00	0.000e+00	-231.987e+03	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	-231.987e+03	0.000e+00
2	10	0.000e+00	0.000e+00	-561.424e+03	0.000e+00
3	0	0.000e+00	0.000e+00	-561.424e+03	0.000e+00
3	1	0.000e+00	0.000e+00	-455.556e+03	0.000e+00
3	2	0.000e+00	0.000e+00	-398.620e+03	0.000e+00
3	3	0.000e+00	0.000e+00	-348.793e+03	0.000e+00
3	4	0.000e+00	0.000e+00	-298.965e+03	0.000e+00
3	5	0.000e+00	0.000e+00	-249.138e+03	0.000e+00
3	6	0.000e+00	0.000e+00	-199.310e+03	0.000e+00
3	7	0.000e+00	0.000e+00	-149.482e+03	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tuck Axle Rxn – No Impact and No Lane

Dual Truck Train + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	-1.164e+03	59.754e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	-1.164e+03	59.754e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tandem Train + Lane Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00

1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Dual Tandem Train + Lane Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Truck Envelopes (Per Lane)

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Truck Envelopes (Per Lane)

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Strength I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-13.081e+03	35.067e+03	0.000e+00	0.000e+00
1	1	-17.980e+03	23.395e+03	-77.980e+03	147.507e+03
1	2	-24.755e+03	13.598e+03	-179.141e+03	234.097e+03
1	3	-32.741e+03	5.014e+03	-308.468e+03	264.757e+03
1	4	-41.363e+03	-2.934e+03	-475.680e+03	249.204e+03
1	5	-50.601e+03	-10.267e+03	-675.087e+03	181.749e+03
1	6	-60.429e+03	-17.010e+03	-906.688e+03	62.391e+03
1	7	-70.813e+03	-23.196e+03	-1.170e+06	-108.868e+03
1	8	-81.717e+03	-28.864e+03	-1.466e+06	-332.031e+03
1	9	-93.096e+03	-34.056e+03	-1.808e+06	-593.275e+03
1	10	-104.900e+03	-38.823e+03	-2.277e+06	-811.275e+03
2	0	51.003e+03	139.193e+03	-2.277e+06	-811.275e+03
2	1	40.119e+03	112.038e+03	-873.814e+03	-221.067e+03
2	2	28.283e+03	85.835e+03	-132.041e+00	456.309e+03
2	3	15.175e+03	60.903e+03	348.512e+03	1.215e+06
2	4	601.250e+00	37.438e+03	531.738e+03	1.697e+06
2	5	-15.563e+03	15.563e+03	592.813e+03	1.857e+06
2	6	-37.438e+03	-601.250e+00	531.738e+03	1.697e+06
2	7	-60.903e+03	-15.175e+03	348.512e+03	1.215e+06
2	8	-85.835e+03	-28.283e+03	-132.041e+00	456.309e+03
2	9	-112.038e+03	-40.119e+03	-873.814e+03	-221.067e+03
2	10	-139.193e+03	-51.003e+03	-2.277e+06	-811.275e+03
3	0	38.823e+03	104.900e+03	-2.277e+06	-811.275e+03
3	1	34.056e+03	93.096e+03	-1.808e+06	-593.275e+03
3	2	28.864e+03	81.717e+03	-1.466e+06	-332.031e+03
3	3	23.196e+03	70.813e+03	-1.170e+06	-108.868e+03
3	4	17.010e+03	60.429e+03	-906.688e+03	62.391e+03
3	5	10.267e+03	50.601e+03	-675.087e+03	181.749e+03
3	6	2.934e+03	41.363e+03	-475.680e+03	249.204e+03
3	7	-5.014e+03	32.741e+03	-308.468e+03	264.757e+03
3	8	-13.598e+03	24.755e+03	-179.141e+03	234.097e+03
3	9	-23.395e+03	17.980e+03	-77.980e+03	147.507e+03
3	10	-35.067e+03	13.081e+03	0.000e+00	0.000e+00

Strength I Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-13.081e+03	35.067e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	89.827e+03	244.093e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	89.827e+03	244.093e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-13.081e+03	35.067e+03	0.000e+00	0.000e+00

Service I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-4.232e+03	21.946e+03	0.000e+00	0.000e+00
1	1	-9.496e+03	13.826e+03	-34.357e+03	90.291e+03
1	2	-15.138e+03	6.083e+03	-94.469e+03	138.415e+03
1	3	-21.151e+03	-1.286e+03	-180.338e+03	144.370e+03
1	4	-27.528e+03	-8.293e+03	-291.962e+03	108.156e+03
1	5	-34.257e+03	-14.948e+03	-429.341e+03	29.774e+03
1	6	-41.323e+03	-21.267e+03	-592.476e+03	-90.775e+03
1	7	-48.707e+03	-27.267e+03	-781.367e+03	-253.494e+03
1	8	-56.388e+03	-32.970e+03	-996.014e+03	-458.381e+03
1	9	-64.340e+03	-38.402e+03	-1.244e+06	-697.540e+03
1	10	-72.535e+03	-43.591e+03	-1.572e+06	-924.496e+03
2	0	57.437e+03	96.181e+03	-1.572e+06	-924.496e+03
2	1	45.559e+03	77.336e+03	-595.946e+03	-290.584e+03
2	2	33.137e+03	59.034e+03	66.392e+03	299.847e+03
2	3	19.988e+03	41.459e+03	430.424e+03	830.651e+03
2	4	6.002e+03	24.721e+03	634.007e+03	1.163e+06
2	5	-8.893e+03	8.893e+03	701.869e+03	1.275e+06
2	6	-24.721e+03	-6.002e+03	634.007e+03	1.163e+06
2	7	-41.459e+03	-19.988e+03	430.424e+03	830.651e+03
2	8	-59.034e+03	-33.137e+03	66.392e+03	299.847e+03
2	9	-77.336e+03	-45.559e+03	-595.946e+03	-290.584e+03
2	10	-96.181e+03	-57.437e+03	-1.572e+06	-924.496e+03
3	0	43.591e+03	72.535e+03	-1.572e+06	-924.496e+03
3	1	38.402e+03	64.340e+03	-1.244e+06	-697.540e+03
3	2	32.970e+03	56.388e+03	-996.014e+03	-458.381e+03
3	3	27.267e+03	48.707e+03	-781.367e+03	-253.494e+03
3	4	21.267e+03	41.323e+03	-592.476e+03	-90.775e+03
3	5	14.948e+03	34.257e+03	-429.341e+03	29.774e+03
3	6	8.293e+03	27.528e+03	-291.962e+03	108.156e+03
3	7	1.286e+03	21.151e+03	-180.338e+03	144.370e+03
3	8	-6.083e+03	15.138e+03	-94.469e+03	138.415e+03
3	9	-13.826e+03	9.496e+03	-34.357e+03	90.291e+03
3	10	-21.946e+03	4.232e+03	0.000e+00	0.000e+00

Service I Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-4.232e+03	21.946e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	101.029e+03	168.717e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	101.029e+03	168.717e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-4.232e+03	21.946e+03	0.000e+00	0.000e+00

Service II Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-7.505e+03	26.526e+03	0.000e+00	0.000e+00
1	1	-12.826e+03	17.493e+03	-50.966e+03	111.077e+03
1	2	-18.637e+03	8.950e+03	-127.688e+03	175.062e+03
1	3	-24.933e+03	891.916e+00	-230.165e+03	191.955e+03
1	4	-31.700e+03	-6.694e+03	-358.398e+03	161.755e+03
1	5	-38.925e+03	-13.824e+03	-512.387e+03	84.463e+03
1	6	-46.588e+03	-20.515e+03	-692.132e+03	-39.920e+03
1	7	-54.665e+03	-26.793e+03	-897.632e+03	-211.397e+03
1	8	-63.127e+03	-32.685e+03	-1.128e+06	-429.965e+03
1	9	-71.942e+03	-38.224e+03	-1.396e+06	-685.359e+03
1	10	-81.073e+03	-43.447e+03	-1.759e+06	-917.165e+03
2	0	57.194e+03	107.561e+03	-1.759e+06	-917.165e+03
2	1	45.247e+03	86.557e+03	-673.275e+03	-276.304e+03
2	2	32.593e+03	66.259e+03	45.256e+03	348.747e+03
2	3	18.995e+03	46.907e+03	416.705e+03	937.001e+03
2	4	4.307e+03	28.643e+03	620.289e+03	1.309e+06

2	5	-11.561e+03	11.561e+03	688.150e+03	1.433e+06
2	6	-28.643e+03	-4.307e+03	620.289e+03	1.309e+06
2	7	-46.907e+03	-18.995e+03	416.705e+03	937.001e+03
2	8	-66.259e+03	-32.593e+03	45.256e+03	348.747e+03
2	9	-86.557e+03	-45.247e+03	-673.275e+03	-276.304e+03
2	10	-107.561e+03	-57.194e+03	-1.759e+06	-917.165e+03
3	0	43.447e+03	81.073e+03	-1.759e+06	-917.165e+03
3	1	38.224e+03	71.942e+03	-1.396e+06	-685.359e+03
3	2	32.685e+03	63.127e+03	-1.128e+06	-429.965e+03
3	3	26.793e+03	54.665e+03	-897.632e+03	-211.397e+03
3	4	20.515e+03	46.588e+03	-692.132e+03	-39.920e+03
3	5	13.824e+03	38.925e+03	-512.387e+03	84.463e+03
3	6	6.694e+03	31.700e+03	-358.398e+03	161.755e+03
3	7	-891.916e+00	24.933e+03	-230.165e+03	191.955e+03
3	8	-8.950e+03	18.637e+03	-127.688e+03	175.062e+03
3	9	-17.493e+03	12.826e+03	-50.966e+03	111.077e+03
3	10	-26.526e+03	7.505e+03	0.000e+00	0.000e+00

Service II Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-7.505e+03	26.526e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	100.641e+03	188.635e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	100.641e+03	188.635e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-7.505e+03	26.526e+03	0.000e+00	0.000e+00

Service III Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	-2.050e+03	18.892e+03	0.000e+00	0.000e+00
1	1	-7.277e+03	11.381e+03	-23.284e+03	76.434e+03
1	2	-12.805e+03	4.172e+03	-72.324e+03	113.983e+03
1	3	-18.631e+03	-2.738e+03	-147.119e+03	112.646e+03
1	4	-24.747e+03	-9.359e+03	-247.670e+03	72.424e+03
1	5	-31.145e+03	-15.698e+03	-373.977e+03	-6.684e+03
1	6	-37.813e+03	-21.768e+03	-526.040e+03	-124.679e+03
1	7	-44.735e+03	-27.583e+03	-703.858e+03	-281.559e+03
1	8	-51.895e+03	-33.161e+03	-907.431e+03	-477.325e+03
1	9	-59.271e+03	-38.521e+03	-1.143e+06	-705.660e+03
1	10	-66.842e+03	-43.687e+03	-1.447e+06	-929.384e+03
2	0	57.600e+03	88.595e+03	-1.447e+06	-929.384e+03
2	1	45.767e+03	71.189e+03	-544.393e+03	-300.104e+03
2	2	33.500e+03	54.217e+03	80.483e+03	267.247e+03
2	3	20.650e+03	37.827e+03	439.569e+03	759.751e+03
2	4	7.131e+03	22.107e+03	643.153e+03	1.067e+06
2	5	-7.114e+03	7.114e+03	711.014e+03	1.169e+06
2	6	-22.107e+03	-7.131e+03	643.153e+03	1.067e+06
2	7	-37.827e+03	-20.650e+03	439.569e+03	759.751e+03
2	8	-54.217e+03	-33.500e+03	80.483e+03	267.247e+03
2	9	-71.189e+03	-45.767e+03	-544.393e+03	-300.104e+03
2	10	-88.595e+03	-57.600e+03	-1.447e+06	-929.384e+03
3	0	43.687e+03	66.842e+03	-1.447e+06	-929.384e+03
3	1	38.521e+03	59.271e+03	-1.143e+06	-705.660e+03
3	2	33.161e+03	51.895e+03	-907.431e+03	-477.325e+03
3	3	27.583e+03	44.735e+03	-703.858e+03	-281.559e+03
3	4	21.768e+03	37.813e+03	-526.040e+03	-124.679e+03
3	5	15.698e+03	31.145e+03	-373.977e+03	-6.684e+03
3	6	9.359e+03	24.747e+03	-247.670e+03	72.424e+03
3	7	2.738e+03	18.631e+03	-147.119e+03	112.646e+03
3	8	-4.172e+03	12.805e+03	-72.324e+03	113.983e+03
3	9	-11.381e+03	7.277e+03	-23.284e+03	76.434e+03
3	10	-18.892e+03	2.050e+03	0.000e+00	0.000e+00

Service III Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	-2.050e+03	18.892e+03	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	101.288e+03	155.438e+03	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	101.288e+03	155.438e+03	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	-2.050e+03	18.892e+03	0.000e+00	0.000e+00

Fatigue Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
1	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
1	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	1	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	2	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	3	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	4	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	5	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	6	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	7	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	8	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	9	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	10	0.000e+00	0.000e+00	0.000e+00	0.000e+00

Fatigue Limit State Envelopes

Pier	FxMin(lbs)	FxMax(lbs)	FyMin(lbs)	FyMax(lbs)	MzMin(ft-lbs)	MzMax(ft-lbs)
1	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
2	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
3	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00
4	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00	0.000e+00

LRFD Pier Live Load Distribution

Developed on 12/09/2006
Last Modified on 8/26/2010

DOT refers to the Iowa Department of Transportation.
OBS refers to the Iowa DOT Office of Bridges and Structures

The OBS will only support those persons using this software in connection with Iowa DOT related business.

Please report any spreadsheet errors to the Iowa DOT OBS.

This spreadsheet was developed to aid the design of typical Iowa DOT piers. The Iowa DOT Bridge Design Manual (BDM) should be consulted for the most up-to-date policies.

Description:
This spreadsheet allows the user to determine live load beam reactions to a pier cap for varying transverse live load positions and varying numbers of lanes loaded. The distribution of live load through the slab to the beams is based on the assumption that the slab between beams is simply supported. Office practice is to distribute the live load to the exterior beam with the assumption that all of the live load on the slab overhang is transferred to the exterior beam. [However, provision is made in the spreadsheet to distribute live load on the slab overhang to the exterior and first interior beam based on slab continuity over the simple exterior beam support.] This spreadsheet is designed to facilitate input into the LEAP® RC-PIER® (RC-Pier) program.

RC-Pier Import Feature:
The loads generated by this spreadsheet can be exported to a text file that can be imported directly into RC-Pier.

Steps:
1.) The user must determine the HL-93 live load axle pier reaction from a software such as QConBridge™. The truck and lane axle reactions must be determined separately. Impact should be removed from the live load truck reation. Multiple Presence Factors should be excluded as well.
2.) The user should fill in the cell input entries on the **Geometry** tab. Cell input entries are typically shown in bold blue text as: **Input**
Cells with calculated output are typically shown in bold red text as: **Output**
3.) The **Placement** tab allows the user to place lanes of live load in different positions along the slab. The resulting beam reactions are based on unit load influence values of 1.000 kip for the lane and truck axle live loads (see the **Geometry** tab). Each load case can be stored, recalled, and/or deleted using the buttons on the **Placement** tab. (The calculated beam reactions include the Multiple Presence Factors.)
4.) The **PierResults** tab allows the user to scale the unit load influence beam reactions by the actual truck and lane axle reactions for up to four piers (i.e. four different live load pier reactions). These live load reactions can be exported to a text file which can be imported directly into RC-Pier.
5.) The **LoadGraphs** tab allows the user to print a copy of the graphs showing the live load arrangements for each load case.

Limitations:
1.) Up to 10 beam lines can be entered.
2.) Up to 8 lanes of live load can be present in a load case.
3.) Up to 20 load cases can be stored.
4.) Beam reactions, based on the same unit load influence values, can be stored for up to 4 piers.

Notes:
1.) The user must manually place the live loads in order to create a suitable envelope of loads for a pier design. On the **Geometry** tab the user has the option to enter some of the pier geometry. Entering this geometry allows the pier to be graphed on the **Placement** tab. The intent is to help the user to visualize how the live loads should be placed in order to generate the live load envelopes needed for pier design.
2.) On the **Placement** tab, you must enter both the traffic location and the distributed lane location in order for the graph to plot the new load and for the beam reactions to be recalculated for the new load.
3.) A truck axle load is placed on the slab as two concentrated wheel loads spaced 6' apart. The lane axle load is placed as a 10' wide uniformly distributed load. The truck load is always centered about the lane load.
4.) If you change something on the **Geometry** tab you should recall each saved load case and resave to overwrite the previous one. The spreadsheet does not automatically update the beam reactions for changes.
5.) The **unitLoadResults**, **calcGraph**, and **calcBeamRxn** tabs contain most of the calculations needed for the spreadsheet to work.

Specify Bridge Geometry and Live Load

Note:	Roadway dimensions and beam spacing should be taken as perpendicular to roadway.

Note:	Blue text is for user input Red text is typ. calculated

Out to Out Slab Width (vOOS)	43.160	ft
Roadway Width (vRW)	40.000	ft
Left Curb Width (vLCW): Slab Edge to Gutterline	1.580	ft

Beam Height (vBH)	4.500	ft
Number of Beams (vNB)	6	
Left Slab Edge to Beam 1 (vBM01)	3.080	ft
Beam 1 to Beam 2 (vBM12)	7.400	ft
Beam 2 to Beam 3 (vBM23)	7.400	ft
Beam 3 to Beam 4 (vBM34)	7.400	ft
Beam 4 to Beam 5 (vBM45)	7.400	ft
Beam 5 to Beam 6 (vBM56)	7.400	ft
Beam 6 to Beam 7 (vBM67)		ft
Beam 7 to Beam 8 (vBM78)		ft
Beam 8 to Beam 9 (vBM89)		ft
Beam 9 to Beam 10 (vBM910)		ft

#

Graph Pier: 1 = Yes, 0 = No (vGP)	1	
Note: Fill out information below if you want to graph the pier.		
Skew, always positive (vSKW)	23.000	deg

Pier dimensions should be based on distances along the skewed cap.		
Cap Length (vCL)	45.000	ft
Left Cap Edge to Beam 1 (vLCEB1)	2.402	ft
Cap Height (vCH)	4.000	ft
Round Column: 1 = Yes, 0 = No (vRCOL)	1	
Column Diameter or Width (vCW)	2.500	ft
Number of Columns (vNC)	3	
Left Cap Edge to Column 1 (vCOL01)	6.500	ft
Column 1 to Column 2 (vCOL12)	16.000	ft
Column 2 to Column 3 (vCOL23)	16.000	ft
Column 3 to Column 4 (vCOL34)		ft
Column 4 to Column 5 (vCOL45)		ft

Overhang: 1 = Continuous, 0 = Hinged (vOHG)	0	

<-- Office practice is to use Hinged. RC-Pier uses Continuous for auto-generation of LL.

Traffic Lane Width for Placement (vTLWP)	12.000	ft

<-- Office practice is to use 12'. RC-Pier uses 10'.

Number of Lanes	MPF
1 Lane (vMPF1)	1.20
2 Lanes (vMPF2)	1.00
3 Lanes (vMPF3)	0.85
> 3 Lanes (vMPF4)	0.65

*

Unit Truck Axle Reaction (vTAR)	1.000	k
Unit Lane Axle Reaction (vLAR)	1.000	k

<-- Unit loads for truck and lane.

Traffic Lane Width for Max # of Lanes	12.000	ft
Max. Number of Possible Lanes (vNPL)	3	
Transv. Wheel Spacing (vWS)	6.000	ft
Transv. Lane Distribution Width (vLDW)	10.000	ft

* The truck and lane axle reactions will be treated as influence values first. Thus piers with similar geometry, but different live load reactions may be scaled from the same set of influence values for different live load positions.

Graphing the pier allows the user to better visualize the column locations with respect to the beam locations.

Place Traffic Lane Loads

Roadway Width	40.00000	ft

Lane #	Traffic Lane Location - ft (based on left gutterline) Min	Max	Actual
Lane 1	0.00000	28.00000	2.00000
Lane 2	14.00000	28.00000	14.00000
Lane 3	26.00000	28.00000	26.00000
Lane 4	Can't fill	Can't fill	
Lane 5			
Lane 6			
Lane 7			
Lane 8			
Number of Lanes Occupied			3

Number of Load Cases Stored (20 Max.) =	13

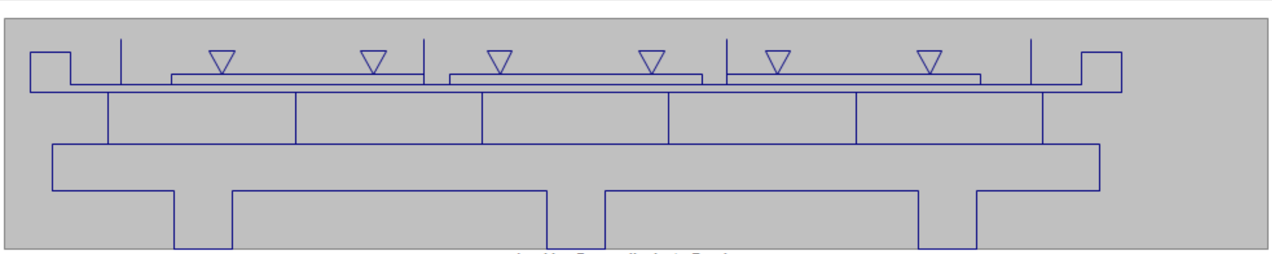
Store As Case # | 13 | Delete Case #

Recall Case #

Uniform Lane Load	0.100	k/ft
Conc Truck Load One wheel Line	0.500	k

Lane #	Distributed Lane Location - ft (based on left lane edge) Min	Max	Actual
Lane 1	0.00000	2.00000	2.00000
Lane 2	0.00000	2.00000	1.00000
Lane 3	0.00000	2.00000	0.00000
Lane 4	0.00000	2.00000	
Lane 5			
Lane 6			
Lane 7			
Lane 8			

Beam Reactions	MPF =		0.85
Beam #	Truck Rxn (k)	Lane Rxn (k)	Total Rxn (k)
1	0.167	0.138	0.304
2	0.505	0.572	1.078
3	0.603	0.565	1.168
4	0.603	0.565	1.168
5	0.505	0.572	1.078
6	0.167	0.138	0.304
7			
8			
9			
10			
Total	2.550	2.550	5.100



Looking Perpendicular to Roadway

Live Load Placement Screen showing the 13th live load case.

These are influence values.

Unit Load Results

For RC-Pier live load input:
1.) Impact should be excluded from the (truck) reactions.
2.) Multiple Presence Factors (MPF) are included in the reactions.
3.) Beam reactions for truck and lane loads should be entered separately.
4.) Truck load results should be entered first, followed by lane load results.
5.) Auto-generation of live load in RC-Pier assumes the overhang is continuous over the exterior beam.
6.) Auto-generation of live load in RC-Pier assumes 10' traffic lanes.

These are the pier 1 LL reactions from QConBridge (no impact).

Export Pier 1 Loads to Text Files

This button allows the user to export these loads to a text file that may be imported into RC-Pier.

Pier 1		
Enter Truck Axle Rxn at Pier	94.589	k
Enter Lane Axle Rxn at Pier	59.754	k
Truck Axle Rxn Used	94.589	k
Lane Axle Rxn Used	59.754	k

Unit Truck Axle Rxn Influence Value	1.000
Unit Lane Axle Rxn Influence Value	1.000

Beam Reactions (kips)

Case Number		Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	Beam 6	Beam 7	Beam 8	Beam 9	Beam 10
1	Truck	59.821	53.686	0.000	0.000	0.000	0.000				
	Lane	37.286	33.832	0.586	0.000	0.000	0.000				
2	Truck	0.000	0.000	0.000	0.000	53.686	59.821				
	Lane	0.000	0.000	0.000	0.586	33.832	37.286				
3	Truck	0.000	56.753	56.753	0.000	0.000	0.000				
	Lane	0.819	35.034	35.034	0.819	0.000	0.000				
4	Truck	0.000	0.000	0.000	56.753	56.753	0.000				
	Lane	0.000	0.000	0.819	35.034	35.034	0.819				
5	Truck	49.851	59.438	56.242	23.647	0.000	0.000				
	Lane	31.072	35.659	39.660	13.118	0.000	0.000				
6	Truck	31.317	70.942	63.272	23.647	0.000	0.000				
	Lane	19.222	44.117	43.051	13.118	0.000	0.000				
7	Truck	0.000	0.000	23.647	56.242	59.438	49.851				
	Lane	0.000	0.000	13.118	39.660	35.659	31.072				
8	Truck	0.000	0.000	23.647	63.272	70.942	31.317				
	Lane	0.000	0.000	13.118	43.051	44.117	19.222				
9	Truck	0.000	27.482	67.107	67.107	27.482	0.000				
	Lane	0.000	16.025	43.729	43.729	16.025	0.000				
10	Truck	49.851	44.738	0.000	0.000	44.738	49.851				
	Lane	31.072	28.193	0.489	0.489	28.193	31.072				
11	Truck	42.373	50.522	47.806	47.806	47.806	4.889				
	Lane	26.411	30.310	33.711	28.450	30.605	2.886				
12	Truck	4.889	47.806	47.806	47.806	50.522	42.373				
	Lane	2.886	30.605	28.450	33.711	30.310	26.411				
13	Truck	15.754	47.806	57.041	57.041	47.806	15.754				
	Lane	8.240	34.205	33.742	33.742	34.205	8.240				
14	Truck										
	Lane										
15	Truck										
	Lane										
16	Truck										
	Lane										
17	Truck										
	Lane										
18	Truck										
	Lane										
19	Truck										
	Lane										
20	Truck										
	Lane										

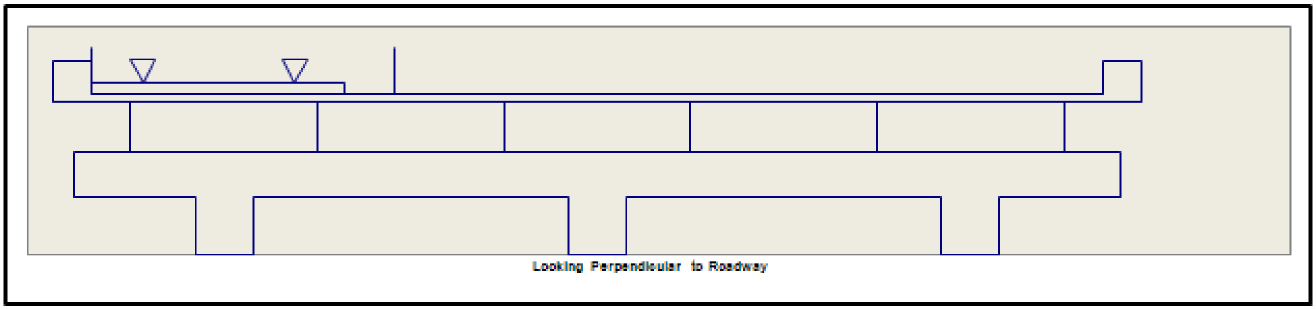
These loads may be entered or imported directly into RC-Pier. As required by RC-Pier: impact is not included, but multiple presence factors (MPFs) are included.

Location of Live Loads (ft)									
Case Number		Lane 1	Lane 2	Lane 3	Lane 4	Lane 5	Lane 6	Lane 7	Lane 8
1	Traffic Lane Location	0.000							
	Distributed Lane Location	0.000							
2	Traffic Lane Location	28.000							
	Distributed Lane Location	2.000							
3	Traffic Lane Location	6.600							
	Distributed Lane Location	1.000							
4	Traffic Lane Location	21.400							
	Distributed Lane Location	1.000							
5	Traffic Lane Location	0.000	12.000						
	Distributed Lane Location	0.000	0.000						
6	Traffic Lane Location	0.000	12.000						
	Distributed Lane Location	2.000	0.000						
7	Traffic Lane Location	16.000	28.000						
	Distributed Lane Location	2.000	2.000						
8	Traffic Lane Location	16.000	28.000						
	Distributed Lane Location	2.000	0.000						
9	Traffic Lane Location	8.000	20.000						
	Distributed Lane Location	2.000	0.000						
10	Traffic Lane Location	0.000	28.000						
	Distributed Lane Location	0.000	2.000						
11	Traffic Lane Location	0.000	12.000	24.000					
	Distributed Lane Location	0.000	0.000	0.000					
12	Traffic Lane Location	4.000	16.000	28.000					
	Distributed Lane Location	2.000	2.000	2.000					
13	Traffic Lane Location	2.000	14.000	26.000					
	Distributed Lane Location	2.000	1.000	0.000					
14	Traffic Lane Location								
	Distributed Lane Location								
15	Traffic Lane Location								
	Distributed Lane Location								
16	Traffic Lane Location								
	Distributed Lane Location								
17	Traffic Lane Location								
	Distributed Lane Location								
18	Traffic Lane Location								
	Distributed Lane Location								
19	Traffic Lane Location								
	Distributed Lane Location								
20	Traffic Lane Location								
	Distributed Lane Location								

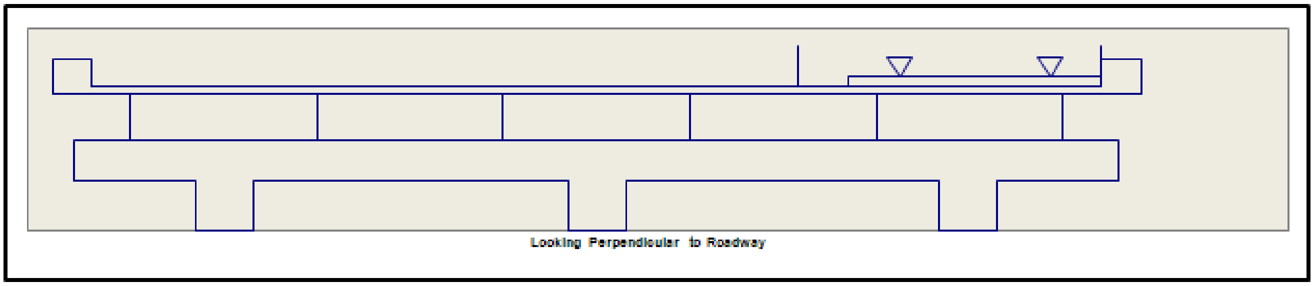
Describes the location of each live load for each load case with respect to the left gutterline.

Graphs depict live load locations for each load case.

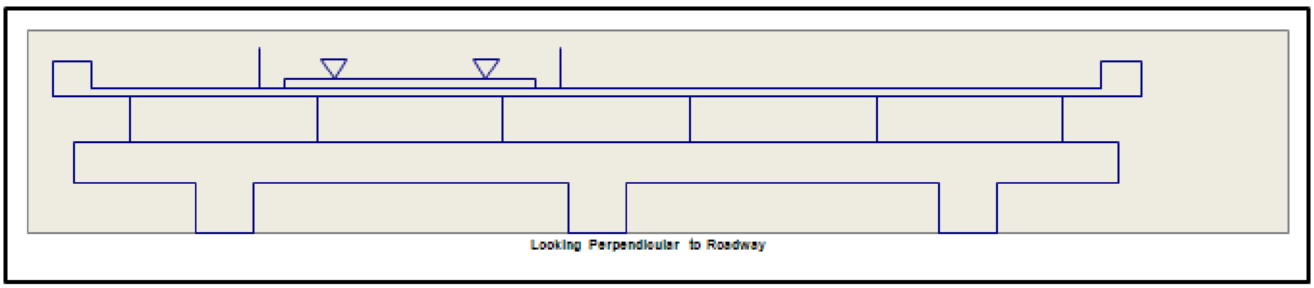
Load Case 1



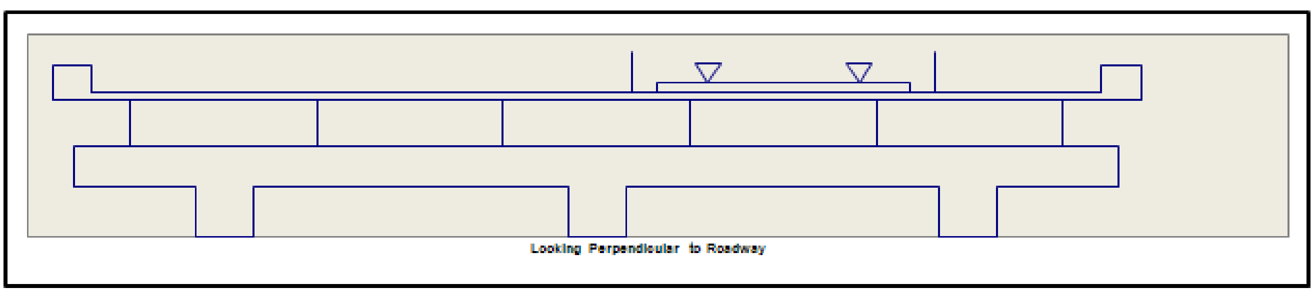
Load Case 2



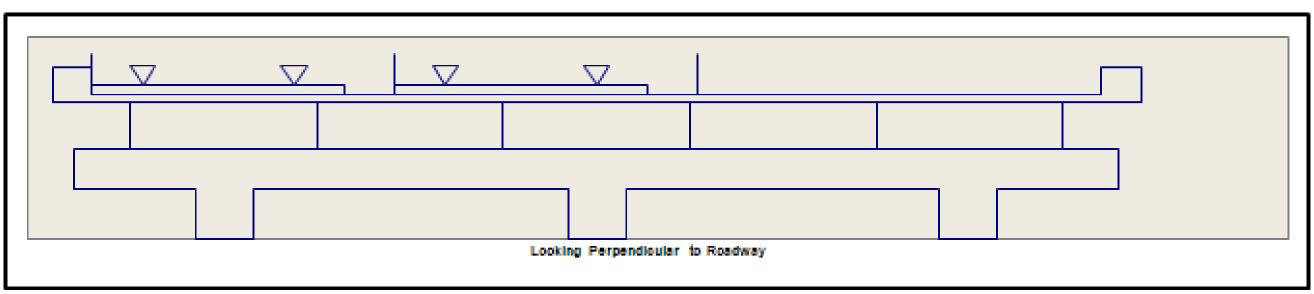
Load Case 3



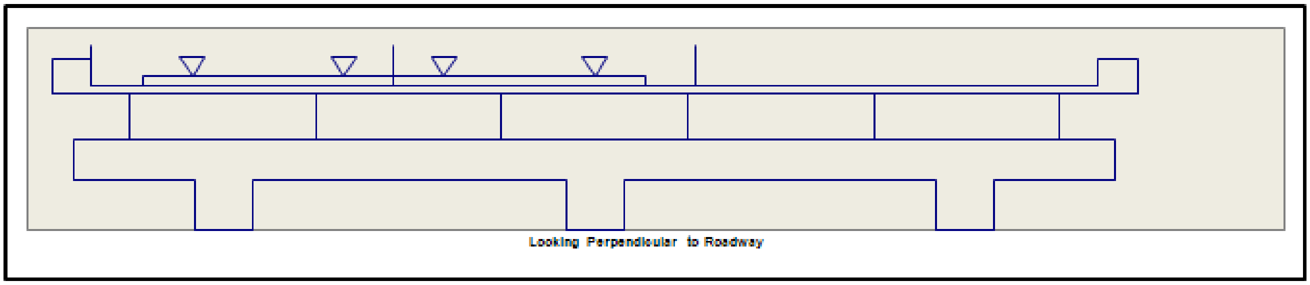
Load Case 4



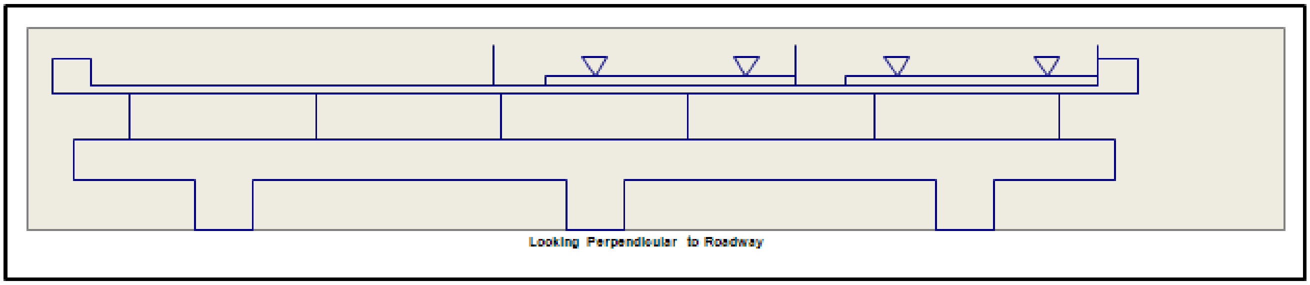
Load Case 5



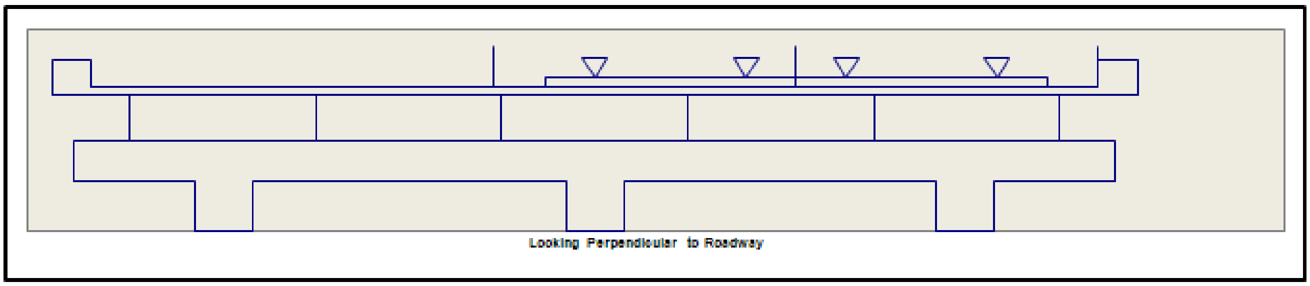
Load Case 6



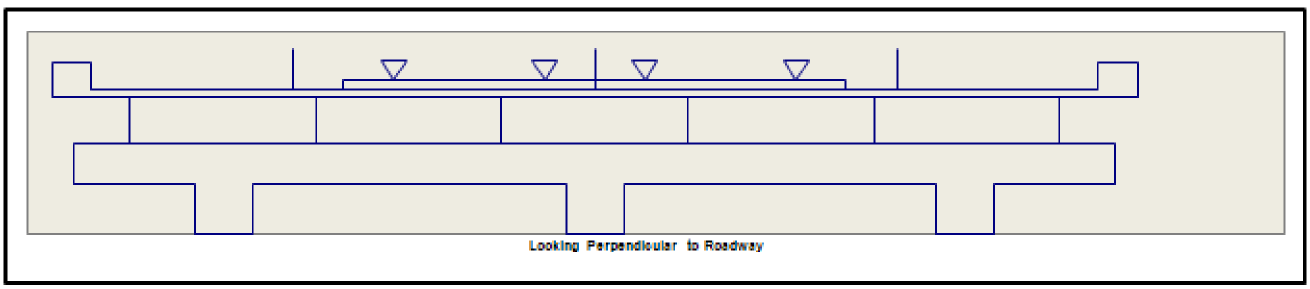
Load Case 7



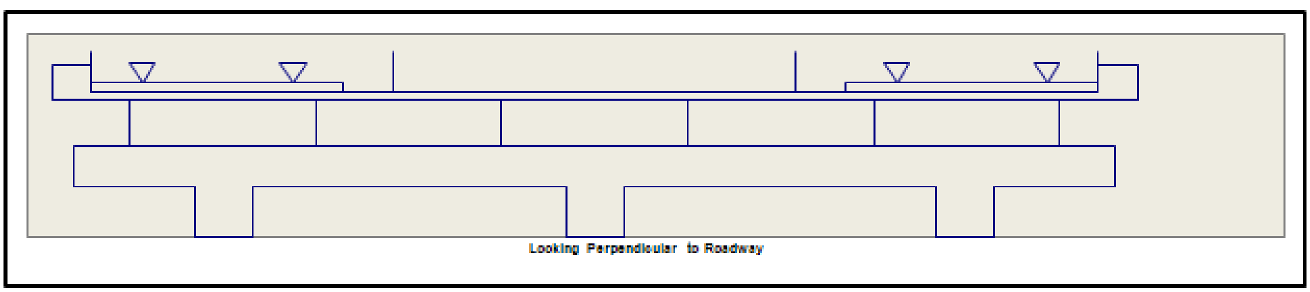
Load Case 8



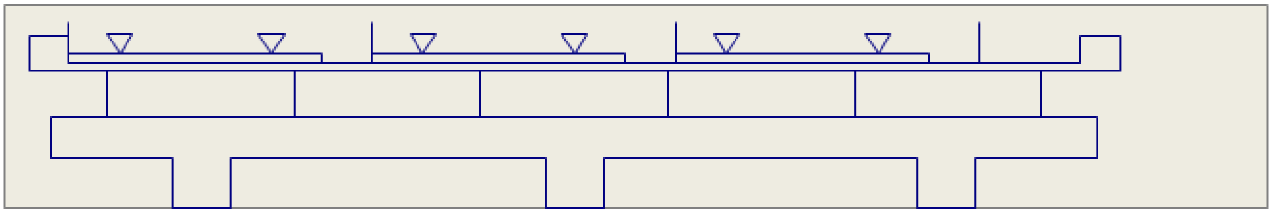
Load Case 9



Load Case 10

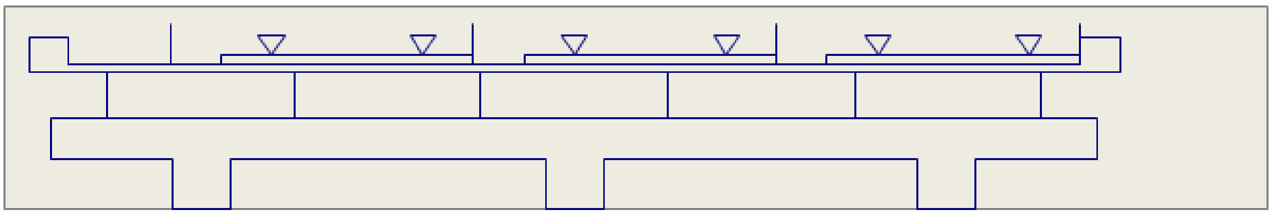


Load Case 11



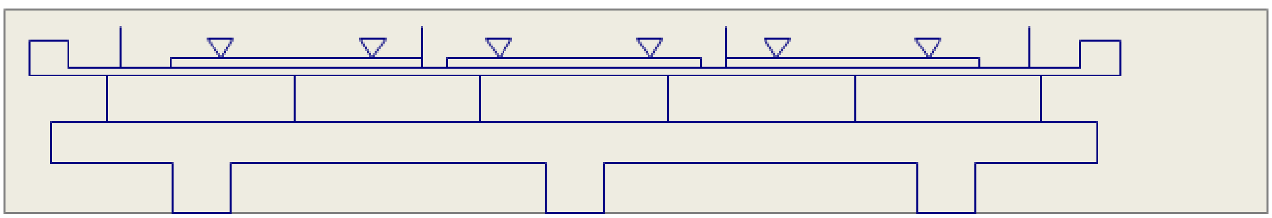
Looking Perpendicular to Roadway

Load Case 12



Looking Perpendicular to Roadway

Load Case 13



Looking Perpendicular to Roadway

Geometrical Considerations for Various Loadings

Average span length is used for superstructure wind loading and braking forces.

Avg Span Length = (0.5)*(50.75’ + 116.5’) = 83.625’

Typically average step height, bearing pad height, and average haunch are used for determining exposed wind areas and distances from lateral superstructure loads to the top of the pier cap. RC-Pier is not really able to accommodate this in its load auto-generation procedure. Since these additional depths are relatively small for this bridge they will be ignored.

Avg Step Hgt = [(811.01 + 811.15 + 811.30 + 811.36 + 811.22 + 811.06)/(6 Beams)] - 811.01
= 0.173’ -- Ignored

Bearing Pad Hgt = 1” = 0.083’ -- Ignored

Avg Haunch = 1” = 0.083’ -- Ignored

Superstructure Wind Area = (2.833’ SBC Hgt) + (8” Slab Thk)/(12 in/ft) + (4.5’ Beam Hgt)
= 8.00’

Center of Gravity of Superstructure Wind Area to Top of Pier Cap = (0.5)*(8.00’) = 4.00’

Dist from Slab Top to Top of Pier Cap = (8” Slab Thk)/(12 in/ft) + (4.5’ Beam Hgt) = 5.17’

Earth cover on top of footing is used not only for the fill weight on the footing, but also figures into the exposed wind area of the substructure.

Depth of Earth Cover over Top of Footing = 4’

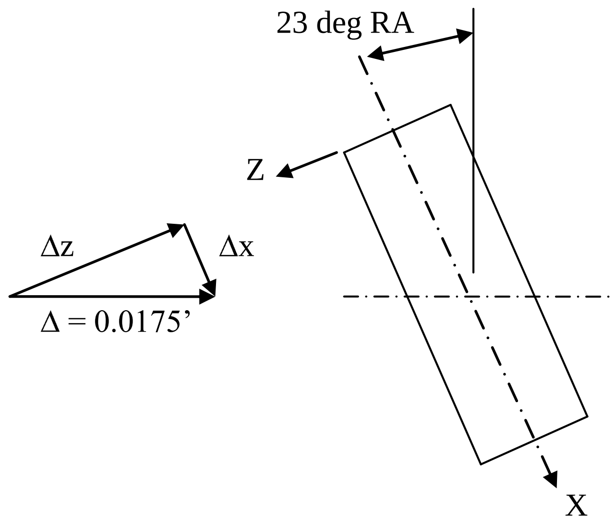
In-Plane Shrinkage and Temperature Considerations

1.) Superstructure Temperature Movement

Piers 1 and 2 have fixed bearings and the abutments are integral. The temperature change is 50 degrees each way from 50 degrees F. The coefficient of thermal expansion is $\mu = 6x10^{-6}$ per degree F.

$\Delta = (0.5)*(116.5')*(6x10^{-6}$ per degree F)*(50 degrees F) = 0.0175' = 0.210"

$\Delta z = \Delta*\cos\theta = (0.0175')*(\cos(23 \text{ deg})) = 0.01611'$
$\Delta x = \Delta*\sin\theta = (0.0175')*(\sin(23 \text{ deg})) = 0.00684'$



2.) Internal Shrinkage of Pier = 0.0002 in/in

3.) In-Plane Pier Temperature Change = ± 50 degrees F

This can be converted into a strain in order to facilitate input into RC-Pier
$(6x10^{-6}$ per degree F)*(± 50 degrees F) = ± 0.0003 in/in

Load Factors for Temperature and Shrinkage Loads

Aashto Lrfd 3.4.1, 5th Edition, Top of Page 3-12

For substructure design Iowa typically uses gross inertia for the pier components. This means we will use the smaller load factors in Aashto Lrfd Load Tables 3.4.1-1 and 3.4.1-3 for TU and SH when calculating force effects.

PIER TEMPERATURE FORCES

Developed on 6/27/2006
Last Modified on 8/26/2010

DOT refers to the Iowa Department of Transportation.
OBS refers to the Iowa DOT Office of Bridges and Structures

Please report any spreadsheet errors to the Iowa DOT OBS.

This spreadsheet was developed to aid the design of typical Iowa DOT piers. The Iowa DOT Bridge Design Manual (BDM) should be consulted for the most up-to-date policies.

Description:
This spreadsheet will determine the lateral forces that must be applied to the top of a pier in order to produce the corresponding lateral deflections that are typically induced by the superstructure temperature movements. The spreadsheet can also generate a STAAD input file for verification. There are a number of simplifying assumptions made in the spreadsheet. The user is responsible for understanding those assumptions and for determining their appropriateness for their design.

The "Description", "Input", and "Output" worksheets are the only worksheets the user needs to consider. Some intermediate calculations are performed on the "Input" worksheet. The worksheet area where these are done is clearly marked.

Assumptions & Limitations
1.) Bearing pad flexibility can be considered.
2.) The pier cap stiffness is not considered for single column piers. The cantilevers if present use the average cross-sectional dimension of the cap height at end and cap height at center. The cap can be considered "infinitely" stiff (the cap area and inertia are set to 10,000 ft^2 and 100,000 ft^2 respectively if this option is used).
3.) One to five columns can be input. Each column can be of a different height and cross-sectional dimension. Columns can be round or rectangular, and if rectangular they can also be tapered in one or both directions. Columns are split into five segments and the average cross-sectional dimensions of the individual segments are used for member properties. Column height input by the user is used in both directions. The column height is not adjusted by the spreadsheet (ie. column height is not adjusted by one-half of the cap height.)
4.) The rotational flexibilty of the pile footings due to axial pile shortening can be input. Up to 50 piles per footing can be entered. Each footing can have a different pile arrangement and pile type. Pile arrangements must be symmetrical about the centerline of the column.
5.) The Direct Stiffness Method is used to solve the problem.
6.) The pier is assumed to move as a unit in the weak-axis direction (all columns deflect the same amount). The cap does not contribute to the model in the weak-axis direction. In the strong-axis direction, the average deflection of the pier at the top of the columns due to a unit lateral force is used to establish the pier stiffness. This stiffness combined with the actual superstructure deflection due to temperature is then used to determine the actual temperature force.
7.) The action and response of the pier in the X and Z axes directions are treated independently of one another.

Pier Stiffness and Temperature Forces

This spreadsheet is designed to determine the forces induced in the pier by temperature movements from the superstructure. The temperature movement along the c.l. of the bridge roadway should be broken down into components transverse and parallel to the pier (always use positive displacements). Once these have been input and the geometry of the pier including any pads and piling have been defined then the program will determine the forces needed to produce those movements. These forces can be entered into RC-Pier as a temperature load. The user must ensure that the sign of the forces enterd into RC-Pier are correct.

See previous page for calculation of temperature movements.

Input

	X-Dir'n, ft	Z-Dir'n, ft
Temperature Movement	0.00684	0.01611

Fixed Pier

Number of Elastomeric Pads	0	0 if no pads
Pad Shear Modulus, G	130.000	psi
Thickness of Neoprene Only	1.000	in
Area of One Pad	200.000	in^2

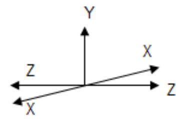
f'c	3500.000	psi

Cap Height at Center (Y)	4.000	ft
Cap Height at End (Y)	3.000	ft
Cap Depth (Z)	3.250	ft

Treat Cap as Infinitely Rigid?	N	(Y)es or (N)o

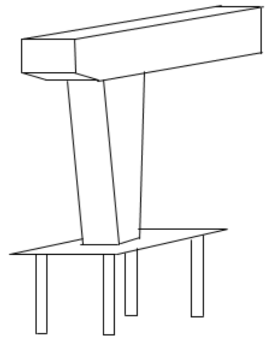
Left Overhang Length (X)	6.500	ft
Right Overhang Length (X)	6.500	ft

Number of Columns	3	Max of 5



Treat Pier Cap Axial Stiffness as Infinitely Rigid?	
Y	(Y)es or (N)o

Just axial stiffness not bending.



	Column 1-2	Column 2-3	Column 3-4	Column 4-5	
Distance Between Columns (X)	16.000	16.000			ft

All Columns the Same?	Y	(Y)es or (N)o	If yes then only enter information for column 1.			
Column Dimensions	Column 1, ft	Column 2, ft	Column 3, ft	Column 4, ft	Column 5, ft	
Column Width at Top (X)	2.500					Enter column diameter for round columns.
Column Width at Bottom (X)	0.000					Enter 0 if column is round.
Column Depth at Top (Z)	0.000					Enter 0 if column is round.
Column Depth at Bottom (Z)	0.000					Enter 0 if column is round.
Column Height (Y)	20.000					Column Height:

Frame Pier: Top of ftg to ctr of cap.
T-Pier: Top of ftg to top of cap.

Include Piling?	Y	(Y)es or (N)o	
All Piling the Same?	Y	(Y)es or (N)o	If yes then only enter information for footing 1.

Piling Information	Footing 1, ft		Footing 2, ft		Footing 3, ft		Footing 4, ft		Footing 5, ft	
Pile: 1=Steel,2=Wood,3=Conc	1									
If Concrete Pile, Enter f'c in psi										
Effective Pile Length, ft	35.000									
Area of One Pile, ft^2	0.117									
Pile Location	X, ft	Z, ft	X, ft	Z, ft	X, ft	Z, ft	X, ft	Z, ft	X, ft	Z, ft
1	-3.000	-3.000								
2	0.000	-3.000								
3	3.000	-3.000								
4	0.000	0.000								
5	-3.000	3.000								
6	0.000	3.000								
7	3.000	3.000								
8										

HP 10x57 Friction Pile = (0.5)*(70' pile length)

Pile arrangement based on existing plans.

Typically the designer does not take pile flexibility into account since the pile arrangement is unknown and it is conservative to neglect it. We will assume pile flexibility for illustrative purposes.

Results

Number of Columns	3

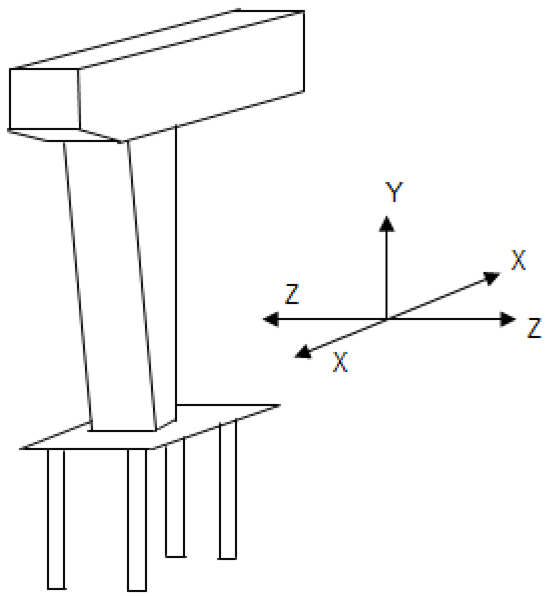
Are Pads Present?	N
Is Cap Infinitely Rigid?	N
Is Piling Considered?	Y

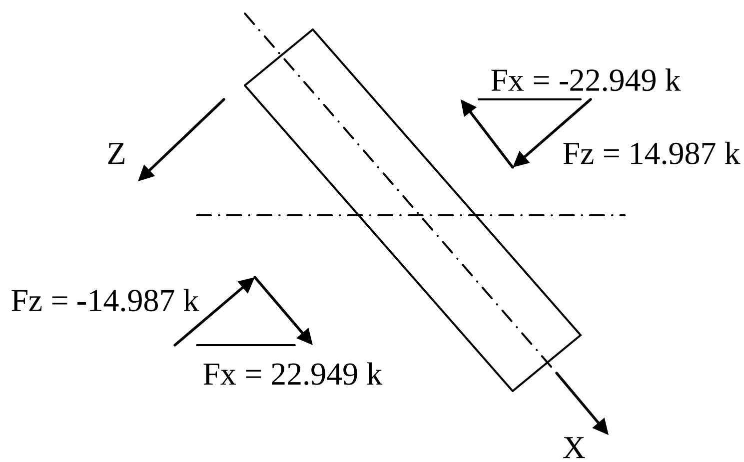
	X-Dir'n, ft	Z-Dir'n, ft
Temperature Movement	0.00684	0.01611

	X-Dir'n, k/ft	Z-Dir'n, k/ft
Pier Stiffness	3355.162	930.316

	X-Dir'n, k	Z-Dir'n, k
Temperature Force	22.949	14.987

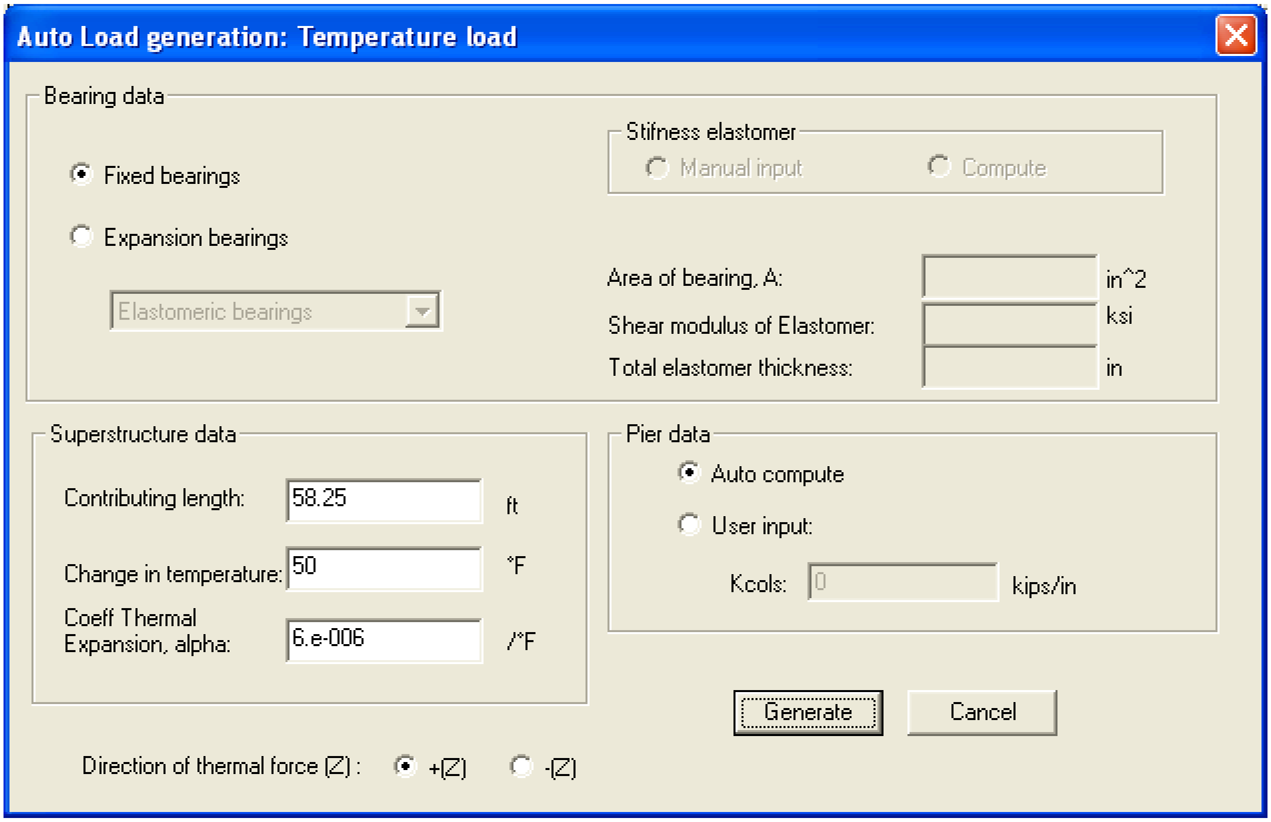
These forces are typically divided among the beams and applied to the top of the pier cap in RC-Pier.





Per Beam
Z: (14.987 k) / (6 beams) = 2.50 k
X: (22.949 k) / (6 beams) = 3.82 k

Auto-generated Temperature Loads in RC-Pier



RC-Pier does not auto-generate temperature loads for skewed piers according to Iowa DOT policy. First, the program calculates a thermal movement based on user input in the figure above. This thermal movement is assumed to act along the Z-axis of the pier. RC-Pier then calculates a thermal force based on the pier's stiffness about the weak axis (i.e. stiffness about the X-axis). The calculated thermal force is then inconsistently assumed to act along the C.L. of the roadway. Based on that inconsistent assumption, RC-Pier breaks the thermal force into component forces (and subsequently component beam forces) along the X- and Z-axes.

The Iowa DOT assumes the original thermal movement acts along the C.L. of the roadway. This movement is broken down into components along the X- and Z- axes. These component movements are used with the pier's stiffness about the strong and weak axes, respectively, in order to determine the thermal forces in each direction. RC-Pier will only determine the correct thermal forces for piers that are not skewed or for piers that have the same stiffness about both axes.

One additional note is that RC-Pier bases pier stiffness on column heights measured from the bottom of the column to the top of the pier cap. I would typically recommend that designers use the column member height from the structural model.

The temperature loads per beam that RC-Pier would auto-generate for this example based on the input in the figure above are:

$F_z = -2.2441$ k $\quad\quad$ $F_x = 0.9526$ k

These may be compared to the values calculated per beam on the previous page.

Application to Determine Pile Footing Stiffness Coefficients

Developed on 1/8/2006
Last Modified on 8/26/2010

DOT refers to the Iowa Department of Transportation.
OBS refers to the Iowa DOT Office of Bridges and Structures

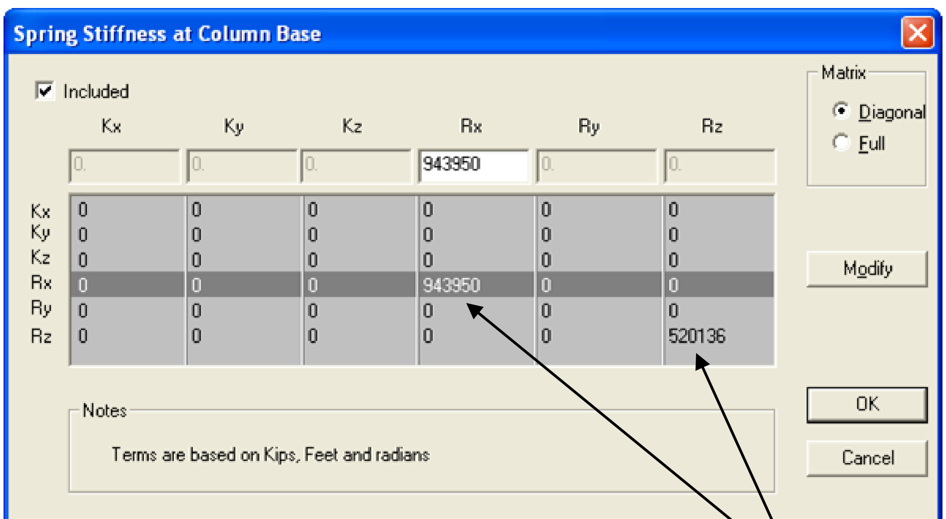
The OBS will only support those persons using this software in connection with Iowa DOT related business.

Please report any spreadsheet errors to the Iowa DOT OBS.

This spreadsheet was developed to aid the design of typical Iowa DOT piers. The Iowa DOT Bridge Design Manual (BDM) should be consulted for the most up-to-date policies.

<u>**Description:**</u>
The purpose of this spreadsheet is to calculate pile footing stiffness coefficients for input into RC-Pier.

In RC-Pier, on the "Geometry" tab under the "Column" button there is a "Spring!" Button. Clicking on this button brings up the window below which can be used to modify the fixity at the base of the columns. This spreadsheet can be used to calculate spring stiffnesses for RxRx and RzRz based upon the rotational stiffness of a pile footing. The Upstation or Downstation Pier View coordinate systems do not affect the values calculated in this spreadsheet. The units for RxRx and RzRz are k*ft/rad.



These are illustrative values only.

<u>**Notes:**</u>
1.) Piling is assumed to be symmetrical about the centerline of the column.
2.) The rotational stiffness of a pile footing that this spreadsheet develops is based on axial shortening/lengthening of the piles due to rotation of the rigid concrete pile cap.
3.) For typical piers the user may generally assume a pier is fixed at its base. This is typically a conservative assumption for pier design and does not require an iterative process.

Pile Footing Stiffness Coefficients

The rotational stiffness of the pile footing is based solely on the differential change in pile length.

Effective Pile Length, Lp	35	ft	§
Area of One Pile, Ap	16.8	in^2	
Pile Modulus Of Elasticity	29000	ksi	

Total Number of Footing Piles	7	
Ix, Pile Inertia	6.30000	ft^4
Iz, Pile Inertia	4.20000	ft^4

Only the pile locations in the positive quadrant should be entered since the pile footing is assumed to be symmetrical. The user should include any piles located on the +X and + Z axes and the pile at the center of the footing if present.

Pile Number	Positive x-coord (feet)	Positive z-coord (feet)
1	3	3
2	0	3
3	0	0
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
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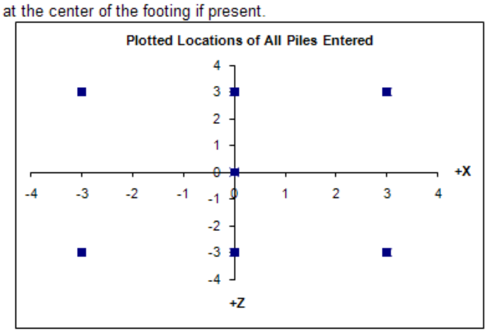
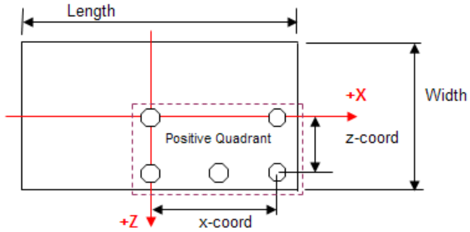
28		
29		
30		





The values below can be used in RC-Pier to model foundation stiffness. All other terms should be set to 0 (0 = fixed).

RxRx, Pile Rotational Stiffness	751680.000	k*ft/rad
RzRz, Pile Rotational Stiffness	501120.000	k*ft/rad

H Pile Size	Area in^2
HP 10x42	12.4
HP 10x57	16.8
HP 12x53	15.5
HP 12x74	21.8
HP 12x84	24.6
HP 14x73	21.4
HP 14x89	26.1
HP 14x117	34.4

§ For friction piles, such as wood or steel piles not driven to bedrock, use 50% of the actual length. For end bearing piles, such as steel piles driven to bedrock, use 75% of the actual length.

These values will be input into RC-Pier.

Typically the designer will assume column bases are fully fixed since the pile arrangement is unknown and it is conservative to make that assumption. We will assume partial fixity for illustrative purposes.

$M = J*\phi$ where J is the flexural stiffness coefficient

$J = (E_p)*(I_{pile\ group})/(L_p)$

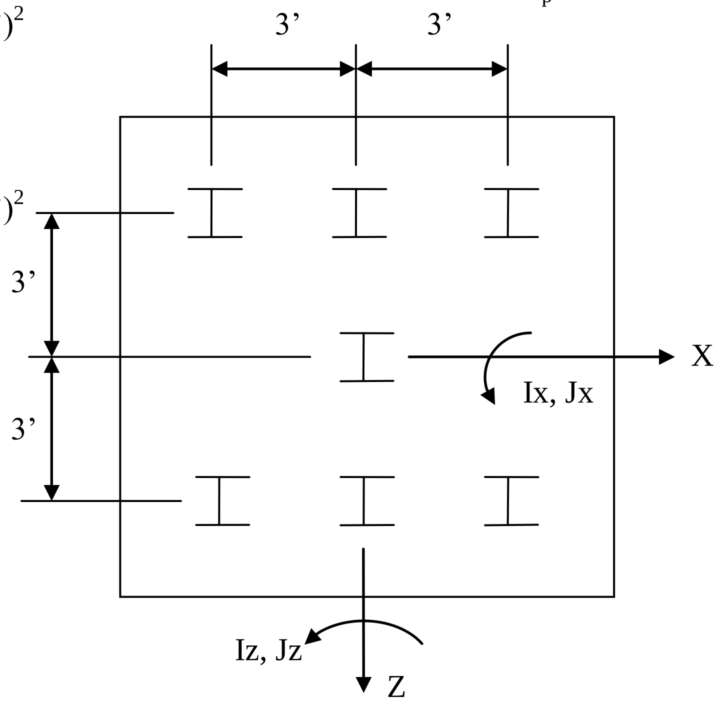
HP10x57
$A_p = 16.8\ in^2$
$E_p = 29{,}000\ ksi$
$L_p = 35'$

$$I_x = \sum_{k=1}^{N} A_p * Z_k^2$$

$= (6\ piles)*[(16.8\ in^2)/(144\ in^2/ft^2)]*(3')^2$
$= 6.3\ ft^4$

$$I_z = \sum_{k=1}^{N} A_p * X_k^2$$

$= (4\ piles)*[(16.8\ in^2)/(144\ in^2/ft^2)]*(3')^2$
$= 4.2\ ft^4$



$J_x = R_xR_x = E_p*I_x/L_p = (29{,}000\ ksi)*(144\ in^2/ft^2)*(6.3\ ft^4)/(35\ ft) = 751{,}680\ k*ft/rad$

$J_z = R_zR_z = E_p*I_z/L_p = (29{,}000\ ksi)*(144\ in^2/ft^2)*(4.2\ ft^4)/(35\ ft) = 501{,}120\ k*ft/rad$

BR & CE Pier Forces

Developed on 12/15/2005
Last Modified on 8/26/2010

DOT refers to the Iowa Department of Transportation.
OBS refers to the Iowa DOT Office of Bridges and Structures

This spreadsheet was developed to aid the design of typical Iowa DOT piers. The Iowa DOT Bridge Design Manual (BDM) should be consulted for the most up-to-date policies.

General Input Tab:

1.) The axes and direction of skew angle match the sign convention used in RC-Pier.
2.) The spreadsheet can handle up to 10 beam lines. The beam spacings can be constant or variable.
3.) Slab and beam dimensions should be entered perpendicular to the centerline of the roadway. Do not enter them along the skew of the pier.
4.) RC-Pier does not have an option to enter haunch thickness, bearing device thickness, or average step height on its Superstructure Parameters screen. This means the auto-generated loads for BR and CE will not be based on these additional dimensions.

RC-Pier Import Feature:

The loads generated by this spreadsheet can be exported to a text file that can be imported directly into RC-Pier.

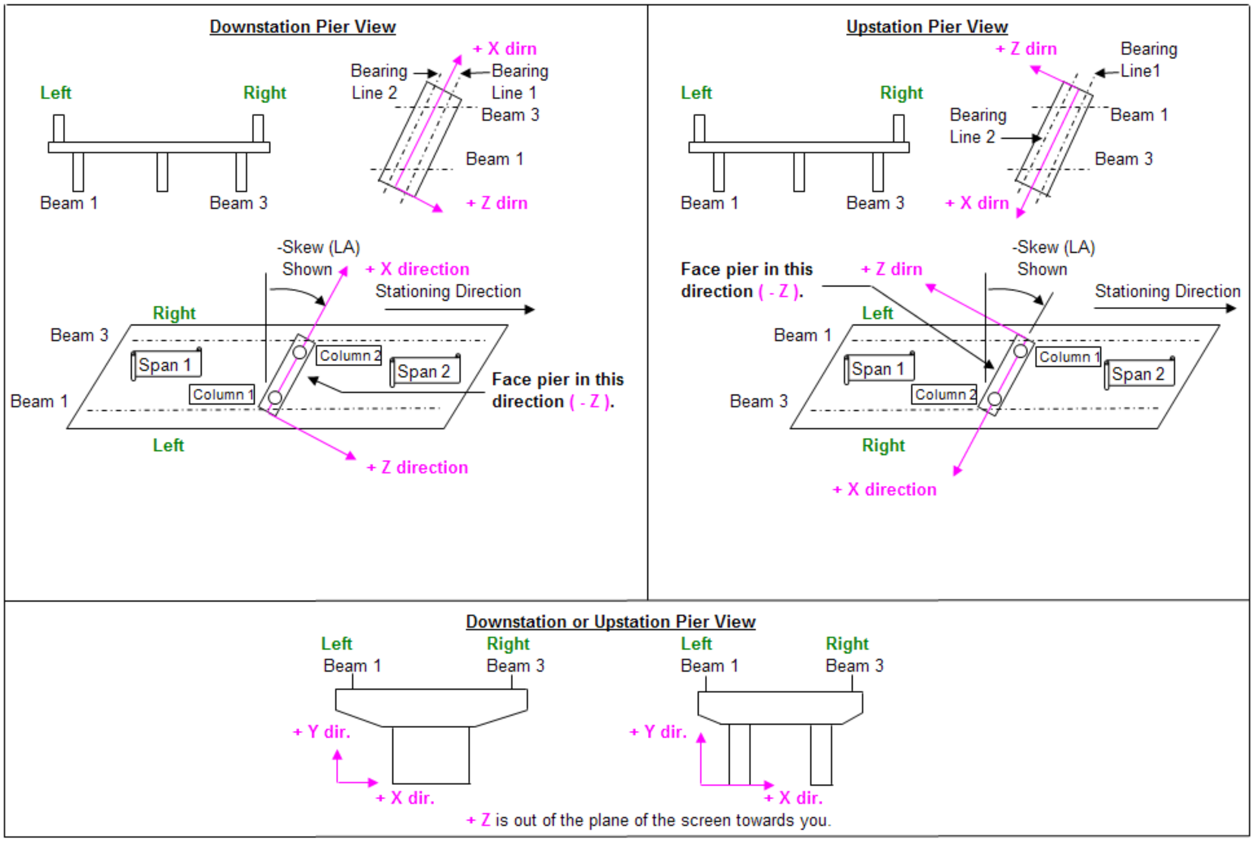
BR Force Tab:

1.) If you auto-generate the BR loads in RC-Pier, then the entire truck portion of the BR load is applied to the pier. In RC-Pier, the truck portion of the BR load is not distributed proportionally to the pier of interest based on total number of bents, average span length, or bearing types. This contradicts office practice.
2.) If you auto-generate the BR loads in RC-Pier, then a torsional moment (Mx) will be generated about the pier cap. Office practice is to exclude such a moment about the pier's weak axis because we assume there is not a sufficient mechansim to transmit a moment from the superstructure to the substructure about this axis.
3.) Do not use the "Two Design Trucks + Lane Load" or "Two Design Tandem + Lane Load" options for auto-generating BR loads. These options use 90% of the axle weight for two trucks/tandems. This would contradict office practice.
4.) Office practice is to base the BR forces on the maximum number of lanes for all load combinations regardless of the number of lanes used for vertical live load. That is, we treat the BR load case (singular) independantly from the vertical live load cases.

CE Force Tab:

1.) The total calculated CE force should, in its entirety, be applied to the pier of interest. The CE force is not distributed among the bents.
2.) If you auto-generate the CE loads in RC-Pier for a skewed bridge, then a torsional moment (Mx) will be generated about the pier cap. Office practice is to exclude such a moment about the pier's weak axis because we assume there is not a sufficient mechansim to transmit a moment from the superstructure to the substructure about this axis.
3.) Do not use the "Two Design Trucks + Lane Load" or "Two Design Tandem + Lane Load" options for auto-generating CE loads. These options use 90% of the axle weight for two trucks/tandems. This would contradict office practice.
4.) Office practice is to base the number of lanes for the CE force on the number of loaded lanes for vertical live load. That is, the CE load cases (plural) and the vertical live load cases are dependant on each other.
5.) The sign of the CE load depends upon the Pier View Direction. A +CE load in the downstation view is a -CE load in the Upstation View.

Definitions



Distribution of BR and CE through Beams

Aashto Lrfd 3.6.4 & 3.6.3

BR = Braking Force   CE = Centrifugal Force

Important Note:	**Roadway dimensions and beam spacing should be taken as perpendicular to roadway.**

Note:	**Blue text is for user input**
	Red text is typ. calculated

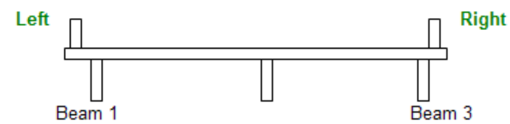
Pier View Direction (vVIEW), D or U	U	D is downstation, U is upstation	
Skew (vSKW), RA is "+" LA is "-"	23.000	deg	

Slab Thickness (vST)	8.000	in	
Haunch Thickness (vHT)	0.000	in	*
Beam Height (vBH)	54.000	in	
Bearing Device Thickness (vBDT)	0.000	in	*
Average Step Height (vASH)	0.000	in	*

Number of Lanes	MPF
1 Lane (vMPF1)	1.20
2 Lanes (vMPF2)	1.00
3 Lanes (vMPF3)	0.85
> 3 Lanes (vMPF4)	0.65

* RC-Pier does not provide the option to enter these dimensions for its auto-generate features on the Superstructure Parameters screen, so you may want to set them to 0.

Out to Out Slab Width (vOOS)	43.160	ft
Roadway Width (vRW)	40.000	ft
Left Curb Width (vLCW)	1.580	ft



Number of Beams (vNB)	6	
Left Slab Edge to Beam 1 (vBM01)	3.080	ft
Beam 1 to Beam 2 (vBM12)	7.400	ft
Beam 2 to Beam 3 (vBM23)	7.400	ft
Beam 3 to Beam 4 (vBM34)	7.400	ft
Beam 4 to Beam 5 (vBM45)	7.400	ft
Beam 5 to Beam 6 (vBM56)	7.400	ft
Beam 6 to Beam 7 (vBM67)		ft
Beam 7 to Beam 8 (vBM78)		ft
Beam 8 to Beam 9 (vBM89)		ft
Beam 9 to Beam 10 (vBM910)		ft
Last Beam to Right Slab Edge	3.080	ft
Tot. Distance bet. Ext. Beams (vTBD)	37.000	ft

Distribution of BR through Beams

Aashto Lrfd 3.6.4

Total Truck Axle Weight (72 kips) or Total Tandem Axle Weight (50 kips) (Do not include impact.)	72.000	k	*
Uniformly Distr. Lane Load (0.640 klf)	0.640	klf	
Average Span Length	83.625	ft	#
Total Bridge Length	218.000	ft	%
Number of Traffic Lanes Loaded	2		

++

BR: 25% of Truck/Tandem	13.810	k	+
BR: 5% of Design Truck/Tandem + Lane	8.114	k	
Enter BR Force to be Used (vBRF)	13.810	k	!

Height of BR above Top of Slab (vHTS)	6.000	ft	**

How is Mz Transferred to the Pier? Enter 1 if by Exterior Beams Only Enter 2 if All Beams Participate	1	##	Export Loads to Text Files

* Enter the tandem weight if the tandem controls the live load pier reaction. Enter the truck weight if the truck or double truck controls the live load pier reaction.

\# For some piers it may be necessary to use a different length than the average length (ex. bridges with friction-acting bearings require special consideration).

% This could also be the distance between superstructure expansion joints.

\+ Loads include the number of lanes loaded and the appropriate MPF.

! Allows the user to override the calculated values. Enter positive value.

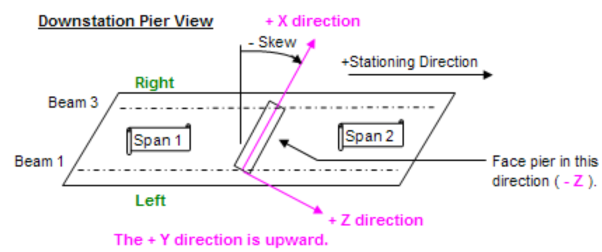
** BR is typically assumed to act 6' above top of slab.

\## The overturning moment, Mz, is transferred to the pier by equal and opposite Fy forces acting through the beams. RC-Pier assumes that only the exterior beams are involved.

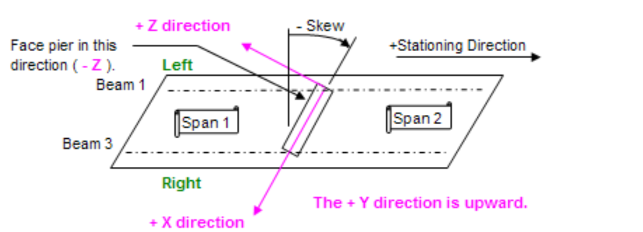
Beam #	BR Loads to the Pier (kips)						
	BR acts in +Stationing Dir'n			BR acts in -Stationing Dir'n			
	Fx	Fy	Fz	Fx	Fy	Fz	
1	0.899	1.499	-2.119	-0.899	-1.499	2.119	
2	0.899	0.000	-2.119	-0.899	0.000	2.119	
3	0.899	0.000	-2.119	-0.899	0.000	2.119	
4	0.899	0.000	-2.119	-0.899	0.000	2.119	
5	0.899	0.000	-2.119	-0.899	0.000	2.119	
6	0.899	-1.499	-2.119	-0.899	1.499	2.119	
7							
8							
9							
10							
		Mx (k*ft)	-141.949	%%	Mx (k*ft)	141.949	%%

%% Bridge office practice is to set the overturning moment Mx to 0.00 even though RC-Pier does not. Mx is provided here for information only.

Downstation Pier View



Upstation Pier View



++ Typically 25% of the truck will control for short to medium bridge lengths.

Braking Example   Sample Calculations

Use 2 lanes for all vertical LL cases, MPF = 1.0.
Total Truck Axle Weight = 8 k + 32 k + 32 k = 72 k
Average Span Length = (0.5)*(50.75' + 116.5') = 83.625'
Total Bridge Length = 50.75' + 116.5' + 50.75' = 218'

BR Load = (0.25)*(72 k)*(2 lanes)*[(83.625')/(218')]*(1.0) = 13.810 k   ← Controls
OR
= (0.05)*[72 k + (0.640 k/ft)*(218')]*(2 lanes)*[(83.625')/(218')]*(1.0) = 8.114 k

F_{z_total} = (13.810 k)*(cos(23 deg)) = -12.712 k
F_{x_total} = (13.810 k)*(sin(23 deg)) = 5.396 k

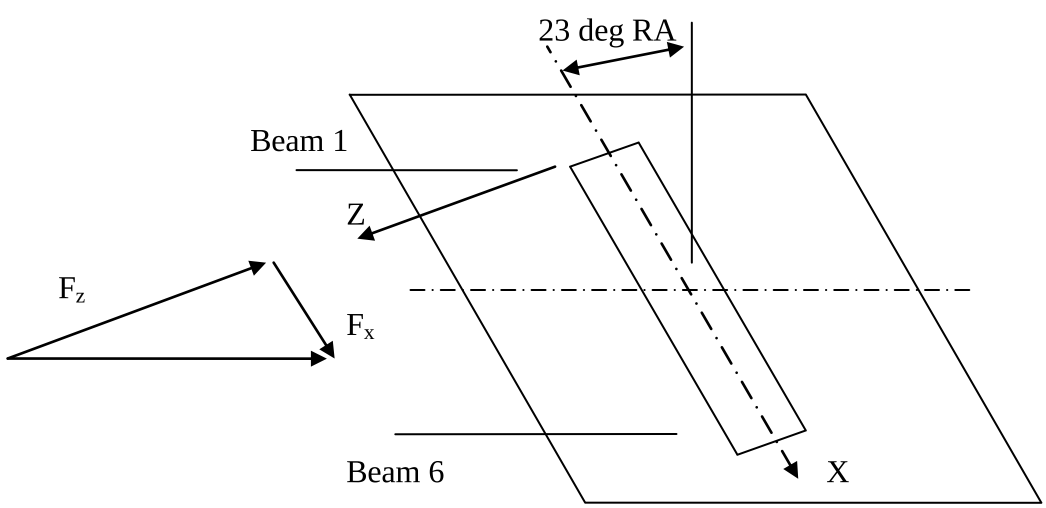
F_z per beam = (-12.712 k)/(6 Beams) = -2.119 k
F_x per beam = (5.396 k)/(6 Beams) = 0.899 k

M_z = (5.396 k)*[6' + (8")/(12 in/ft) + (54")/(12 in/ft)] = 60.254 k*ft

F_y = (60.254 k*ft)/[(5 Beam Spa)*(7.4')/(cos(23 deg))] = 1.499 k

Beam 1, F_y = 1.499 k
Beam 6, F_y = -1.499 k

M_x = (12.712 k)*[6' + (8" + 54")/(12 in/ft)] = 141.949 k*ft   ← Office Policy is to ignore



LRFD Simplified Wind Loading for Usual Girder Bridges

Developed on 12/20/2005
Last Modified on 8/26/2010

DOT refers to the Iowa Department of Transportation.
OBS refers to the Iowa DOT Office of Bridges and Structures

The OBS will only support those persons using this software in connection with Iowa DOT related business.

Please report any spreadsheet errors to the Iowa DOT OBS.

This spreadsheet was developed to aid the design of typical Iowa DOT piers. The Iowa DOT Bridge Design Manual (BDM) should be consulted for the most up-to-date policies.

Description:
This spreadsheet is designed to generate the wind loads used as input into RC-Pier. Specifically, it generates the wind loads for the simplified wind loading case. The definition of "usual girder bridge" in AASHTO Lrfd 3.8.1.2.2 and 3.8.1.3 has been modified by the Iowa DOT in order to extend the range of applicabilty. The individual span length has been extended from 125' to 160' in order to allow this loading to be used for the BTE155s. The maximum height of 30' above low ground or water level has been increased to 100'.

RC-Pier Import Feature:
The loads generated by this spreadsheet can be exported to a text file that can be imported directly into RC-Pier.

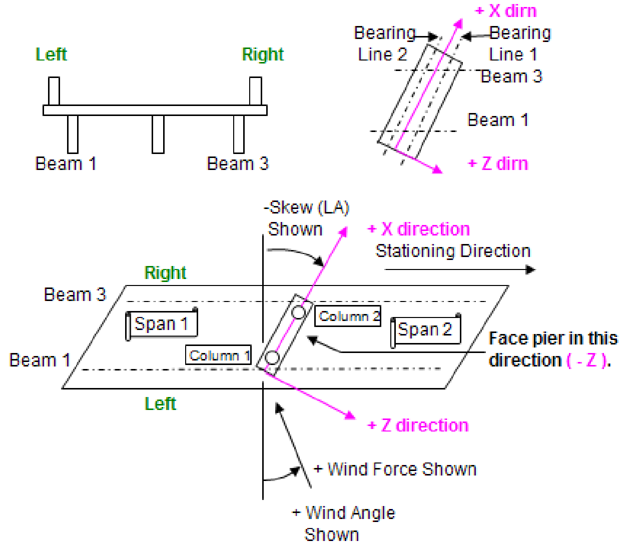
General Input Tab:
1.) The axes and direction of skew angle match the sign convention used in RC-Pier.
2.) Axes may be based on Downstation or Upstation Pier View.
3.) The spreadsheet can handle up to 10 beam lines. The beam spacings can be constant or variable.
4.) Slab and beam dimensions should be entered perpendicular to the centerline of the roadway. Do not enter them along the skew of the pier.
5.) RC-Pier does not have an option to enter haunch thickness, bearing device thickness, or average step height on its Superstructure Parameters screen. This means the auto-generated loads for W and WL will not be based on these additional dimensions.
6.) The spreadsheet can handle up to 5 columns. The column spacing input is not required.

W Force Tab:
1.) If the top of barrier rail is less than or equal to 100' above the ground or water surface then office practice is to use the simplified wind pressure values from Aashto Lrfd 3.8.1.2.2. If the top of barrier rail is above the 100' mark then the user should adjust the wind pressures upward.
2.) If the top of the pier cap is less than or equal to 100' above the ground or water surface then office practice is to use a 0.040 ksf wind pressure (Aashto Lrfd 3.8.1.2.3) in both orthogonal directions at the same time. If the top of the pier cap is above the 100' mark then the user should adjust the wind pressure upward.
3.) Currently RC-Pier's auto-generate feature for WS loads does not handle uplift forces correctly when they are reversible, nor does it restrict uplift to Strength 3 loading. Aashto Lrfd 3.8.2 also restricts vertical wind pressure to a 0 degree wind skew angle. Uplift will be used in conjuction with Strength 3 loading and the simplified wind loading. Typical piers should use a set of wind load cases both with and without uplift.

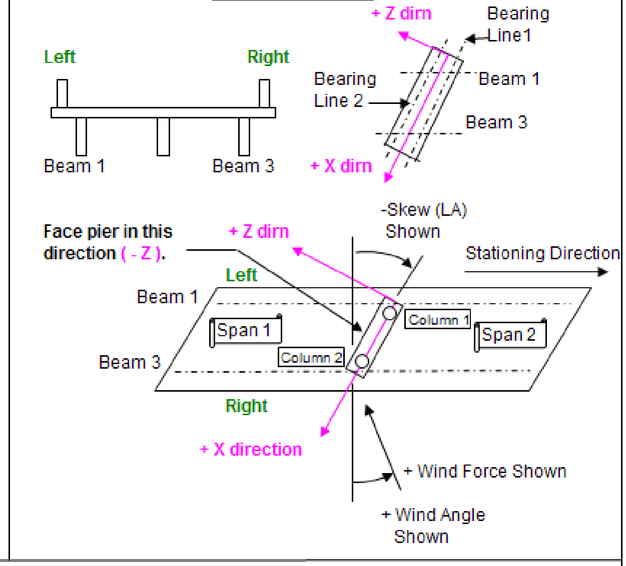
WL Force Tab:
1.) This spreadsheet uses the wind pressure values from Aashto Lrfd 3.8.1.3. The user may adjust these pressures.

Definitions

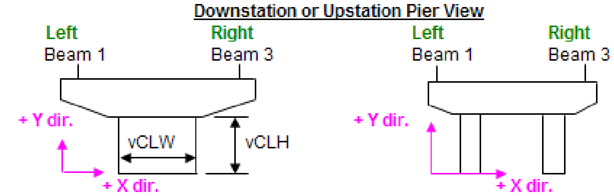
Downstation Pier View



Upstation Pier View

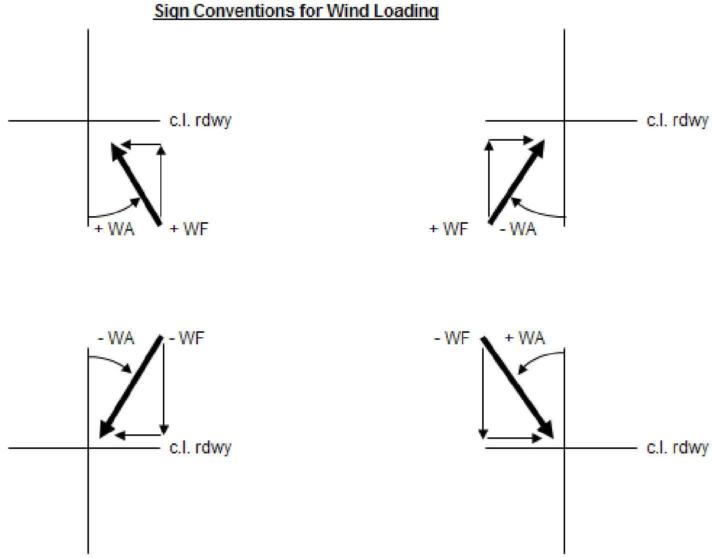


Downstation or Upstation Pier View



+ Z is out of the plane of the screen towards you.

Sign Conventions for Wind Loading



WF = Wind Force
WA = Wind Angle

General Input

Note:	Blue text is for user input
	Red text is typ. calculated

Pier View Direction (vVIEW), D or U	U		D is downstation, U is upstation
Skew (vSKW), RA is "+" LA is "-"	23.000	deg	

Important Note:	**Roadway dimensions and beam spacing should be taken as perpendicular to roadway.**

Out to Out Slab Width (vOOS)	43.160	ft
Roadway Width (vRW)	40.000	ft
Left Curb Width (vLCW)	1.580	ft

Barrier Rail Height (vBRH)	34.000	in	
Slab Thickness (vST)	8.000	in	
Haunch Thickness (vHT)	0.000	in	#
Beam Height (vBH)	54.000	in	
Bearing Device Thickness (vBDT)	0.000	in	#
Average Step Height (vASH)	0.000	in	#

RC-Pier does not provide the option to enter these dimensions, so you may want to set them to 0.

Number of Beams (vNB)	6	
Left Slab Edge to Beam 1 (vBM01)	3.080	ft
Beam 1 to Beam 2 (vBM12)	7.400	ft
Beam 2 to Beam 3 (vBM23)	7.400	ft
Beam 3 to Beam 4 (vBM34)	7.400	ft
Beam 4 to Beam 5 (vBM45)	7.400	ft
Beam 5 to Beam 6 (vBM56)	7.400	ft
Beam 6 to Beam 7 (vBM67)		ft
Beam 7 to Beam 8 (vBM78)		ft
Beam 8 to Beam 9 (vBM89)		ft
Beam 9 to Beam 10 (vBM910)		ft
Last Beam to Right Slab Edge	3.080	ft
Tot. Distance bet. Ext. Beams (vTBD)	37.000	ft

Important Note:	**Cap length and column spacing should be taken along the skew (X axis).**

Cap Length (vCPL)	45.000	ft	
Length of Non-Tapered Segment (vCPT)	35.000	ft	It is assumed to be centered.
Cap Min. Height (vCPMN)	36.000	in	
Cap Max. Height (vCPMX)	48.000	in	
Cap Depth (vCPD)	39.000	in	

Column Width or Diameter (vCLW)	30.000	in	
Col. Depth - enter 0 for round col. (vCLD)	0.000	in	
Col. Height (vCLH)	18.000	ft	Bottom of column to bottom of cap.

Number of Columns (vNC)	3	
Left Cap Edge to Col. 1 (vCL01)	6.500	ft
Col. 1 to Col. 2 (vCL12)	16.000	ft
Col. 2 to Col. 3 (vCL23)	16.000	ft
Col. 3 to Col. 4 (vCL34)		ft
Col. 4 to Col. 5 (vCL45)		ft
Last Column to Right Cap Edge	6.500	ft

Distribution of W to Pier

Aashto Lrfd 3.8

V_{30}, wind velocity at 30' height	80.000	mph	Office policy is to use V_{30} = 80 mph
V_B, base wind velocity of 100 mph at 30' hgt	100.000	mph	

Aashto Lrfd Table 3.8.1.1-1

Upstream Surface Condition	Open Country 1	Suburban 2	City 3	User Option 4
Vo (mph)	8.20	10.90	12.00	0.00
Zo (ft)	0.23	3.28	8.20	0.00

Enter Upstream Surface Condition (1 to 4)	1		Office policy is to use Open Country
Vo, friction velocity	8.200	mph	
Zo, friction length of upstream fetch	0.23	ft	

Hgt above col. base to ground / water surf.	4.000	ft	This height is used to establish Z and is also used in determining the exposed column area for wind loading. RC-Pier always bases Z on the bottom of the column.

Aashto Lrfd Eqn 3.8.1.1-1 and 3.8.1.2.1-1

$V_{DZ} = 2.5 * Vo * (V_{30} / V_B) * \ln(Z / Zo)$ --> where V_{DZ} is the wind design velocity at elevation Z

$P_D = P_B * (V_{DZ} / V_B)^2$ --> where P_B is the base wind pressure

Superstructure Wind

Calc. Z (top of curb to ground/water surf.)	26.000	ft
User's Z	100.000	ft

V_{DZ}, calculated design velocity at User's Z	99.627	mph
User's V_{DZ}	100.000	mph

Aashto Lrfd 3.8.1.2.2

Skew Angle of Wind	Simplified Superstr. Wind Press. for Girders	
	Lateral	Longit.
Degrees	ksf	ksf
N/A	0.050	0.012

Based on User's Factor

Adjusted Simplified Superstr. Wind Press. for Girders	
Lateral	Longit.
ksf	ksf
0.050	0.012

Calc. Wind Press. Factor due to User's V_{DZ}	1.000	See Aashto Lrfd Eqn 3.8.1.2.1-1
User's Factor	1.000	

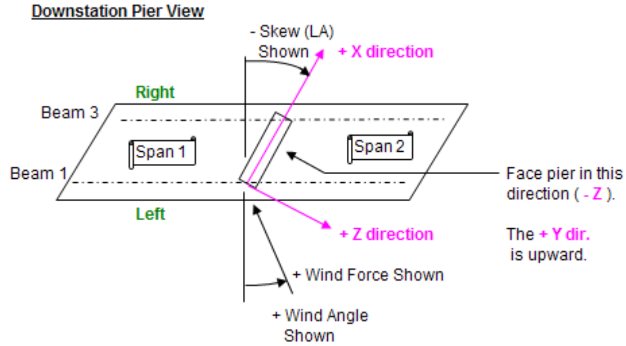
Office practice is to use the simplified wind pressures given in Aashto Lrfd 3.8.1.2.2 when individual span lengths are 160' or less and when Z is 100' or less. In other words, the Factor will typically be 1.000. **Do not use a Factor less than 1.000.** Minimum wind pressures should be 0.050 ksf lateral and 0.012 ksf longitudinal.

Above 100' pressures are to be adjusted based on V30 = 80 mph for individual span lengths less than 160'. In other words, the Factor will be greater than 1.000.

RC-Pier's auto-generate feature uses adjusted wind pressures if Z is greater than 30' and if the adjusted pressures are higher than those in Aashto Lrfd Table 3.8.1.2.2-1.

The user needs to ensure that the transverse wind loading requirement of 0.300 klf in Aashto Lrfd 3.8.1.2.1 is met. If the superstructure height (includes rail) is greater than 6.00' then this requirement is met because (0.050ksf)*(6') = 0.300 klf.

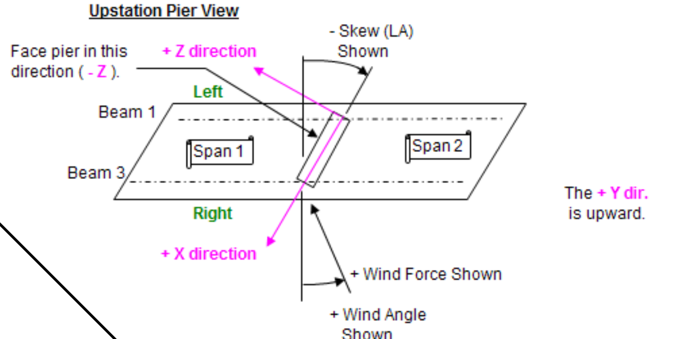
Downstation Pier View



Enter positive wind pressures to be used on the superstructure below.

User's Lateral Spstr. Wind Pressure	0.050	ksf
User's Longit. Spstr. Wind Pressure	0.012	ksf

Upstation Pier View



According to Aashto the vertical wind pressure or uplift force should only be applied when the wind skew angle is 0 degrees. Also, it should only be included in W combinations which do not involve WL (ie. Strength 3). See Aashto Lrfd 3.8.2. Office practice is to apply uplift in the Strength 3 combination for simplified wind loading. Vertical wind pressure is not to be adjusted for height or velocity.

Vertical Wind Pressure	0.020	ksf

Average Span Length	83.625	ft

Uplift Moment Arm	Past in-house pier programs based moment
Enter 1 for In-House	arm on length along pier cap which will
Enter 2 for RC-Pier	be greater for skewed bridges. RC-Pier
2	uses perpendicular distance.

The overturning moment, Mz, is transferred to the pier by equal and opposite Fy forces acting through the beams. RC-Pier assumes that only the exterior beams are involved. User can involve all beams if desired.

How is Mz Transferred to the Pier? Enter 1 if by Exterior Beams Only Enter 2 if All Beams Participate	1

Wind forces with and without uplift will conservatively be applied to all loading combinations. This is in lieu of doing separate RC-Pier runs in order to keep the proper wind loading with the proper load combinations.

The results shown in the tables below are split into 4 groups based on combinations of the ± wind force and ± wind angle. In RC-Pier the user can vary the wind angle magnitude and sign. The wind force direction is typically varied automatically because the load, by default, is treated as a reversible load. In RC-Pier the vertical wind load, if used, is reversed as well which is undesirable. This spreadsheet always assumes vertical wind acts upward.

Wind Uplift Not Included

Beam #	W Loads to the Pier (kips)											
	+ Wind Force, + Wind Angle			+ Wind Force, - Wind Angle			- Wind Force, + Wind Angle			- Wind Force, - Wind Angle		
	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz
1	-5.655	-3.376	-0.947	-4.609	-2.752	-3.410	5.655	3.376	0.947	4.609	2.752	3.410
2	-5.655	0.000	-0.947	-4.609	0.000	-3.410	5.655	0.000	0.947	4.609	0.000	3.410
3	-5.655	0.000	-0.947	-4.609	0.000	-3.410	5.655	0.000	0.947	4.609	0.000	3.410
4	-5.655	0.000	-0.947	-4.609	0.000	-3.410	5.655	0.000	0.947	4.609	0.000	3.410
5	-5.655	0.000	-0.947	-4.609	0.000	-3.410	5.655	0.000	0.947	4.609	0.000	3.410
6	-5.655	3.376	-0.947	-4.609	2.752	-3.410	5.655	-3.376	0.947	4.609	-2.752	3.410
7												
8												
9												
10												
		Mx (k*ft)	-22.721		Mx (k*ft)	-81.839		Mx (k*ft)	22.721		Mx (k*ft)	81.839

Bridge office practice is to set the overturning moment Mx to 0.00. Currently, RC-Pier does not calculate an Mx for W, however, Leap's plans are to include it in a future version (probably as an option).

Wind Uplift Included

Beam #	W Loads to the Pier (kips)											
	+ Wind Force, + Wind Angle			+ Wind Force, - Wind Angle			- Wind Force, + Wind Angle			- Wind Force, - Wind Angle		
	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz
1	-5.655	-10.723	-0.947	-4.609	-10.098	-3.410	5.655	34.784	0.947	4.609	34.160	3.410
2	-5.655	12.031	-0.947	-4.609	12.031	-3.410	5.655	12.031	0.947	4.609	12.031	3.410
3	-5.655	12.031	-0.947	-4.609	12.031	-3.410	5.655	12.031	0.947	4.609	12.031	3.410
4	-5.655	12.031	-0.947	-4.609	12.031	-3.410	5.655	12.031	0.947	4.609	12.031	3.410
5	-5.655	12.031	-0.947	-4.609	12.031	-3.410	5.655	12.031	0.947	4.609	12.031	3.410
6	-5.655	34.784	-0.947	-4.609	34.160	-3.410	5.655	-10.723	0.947	4.609	-10.098	3.410
7												
8												
9												
10												
		Mx (k*ft)	-22.721		Mx (k*ft)	-81.839		Mx (k*ft)	22.721		Mx (k*ft)	81.839

Bridge office practice is to set the overturning moment Mx to 0.00. Currently, RC-Pier does not calculate an Mx for W, however, Leap's plans are to include it in a future version (probably as an option).

Substructure Wind

Calc. Z (pier cap top to ground/water surf.)	18.000	ft
User's Z	100.000	ft

V_{DZ}, calculated design velocity at User's Z	99.627	mph
User's V_{DZ}	100.000	mph

Calc. Wind Press. Factor due to User's V_{DZ}	1.000	See Aashto Lrfd Eqn 3.8.1.2.1-1
User's Factor	1.000	

Aashto Lrfd 3.8.1.2.3

Skew Angle of Wind	Simplified Substr. Wind Press. for Piers	
	Parallel	Perpend.
Degrees	ksf	ksf
N/A	0.040	0.040

Based on User's Factor

Adjusted Simplified Substr. Wind Press. for Piers	
Parallel	Perpend.
ksf	ksf
0.040	0.040

Office practice is to use simplified wind pressures for substructure when individual span lengths are 160' or less and when Z is 100' or less. In other words, the Factor will typically be 1.000. Do not use a Factor less than 1.000. Minimum wind pressures should be 0.040 ksf parallel (X dir.) and 0.040 ksf perpendicular (Z dir.) to the pier. This loading is unique to Iowa and cannot be found in the Aashto Specifications.

Above 100' pressures are to be adjusted based on V30 = 80 mph for individual span lengths less than 160'. In other words, the Factor will be greater than 1.000.

RC_Pier will only use the adjusted wind pressures if Z is greater than 30' and if the adjusted pressures are higher 0.040 ksf (see Aashto Lrfd 3.8.1.2.3).

Enter positive wind pressures to be used on the substructure below.

User's Parallel Wind Press. (X dir.)	0.040	ksf
User's Perpendicular Wind Press. (Z dir.)	0.040	ksf

Export Super- and Sub-structure Wind Loads to Text Files

Cap Loads

Exposed Pier Cap Area (ft^2)	Type of Load	Direction	+ WF + WA Magnitude	+ WF - WA Magnitude	- WF + WA Magnitude	- WF - WA Magnitude	Start Fraction x1 / L	End Fraction x2 / L
13.000	Force (k)	X	-0.520	-0.520	0.520	0.520	0.500	---------
175.000	UDL (k/ft)	Z	-0.156	-0.156	0.156	0.156	0.000	1.000

The sign of the substr. wind loads depends on the sign of the superstr. wind loads.

Note: UDL stands for unformly distributed load.
The total load in the Z direction is averaged over the length of the pier cap.

Column Loads (apply loads to all columns in pier)

Exposed Pier Col. Area (ft^2)	Type of Load	Direction	+ WF + WA Magnitude	+ WF - WA Magnitude	- WF + WA Magnitude	- WF - WA Magnitude	Start Fraction y1 / L	End Fraction y2 / L
35.000	UDL (k/ft)	X	-0.100	-0.100	0.100	0.100	0.195	0.878
35.000	UDL (k/ft)	Z	-0.100	-0.100	0.100	0.100	0.195	0.878

The sign of the substr. wind loads depends on the sign of the superstr. wind loads.

Note: UDL stands for unformly distributed load.
The start and end fractions are based on a column height that extends to the middle of of the minimum cap height.

Wind on Live Load Aashto Lrfd 3.8.1.3

Height of WL above Top of Slab	6.000	ft

Typically, WL is assumed to act 6' above the top of the slab.

Enter positive wind loads to be used on the supertstructure below.

User's Normal WL	0.100	klf
User's Parallel WL	0.040	klf

WL is not varied based on elevation or wind speed.

Aashto Lrfd 3.8.1.3

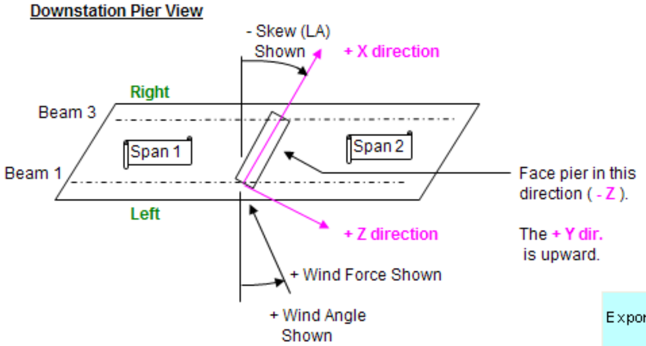
Skew Angle of Wind	Simplified Wind Components on Live Load	
	Normal	Parallel
Degrees	klf	klf
N/A	0.100	0.040

Average Span Length	83.625	ft

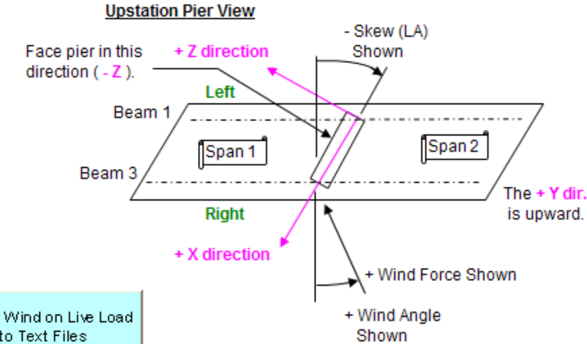
The overturning moment, Mz, is transferred to the pier by equal and opposite Fy forces acting through the beams. RC-Pier assumes that only the ext. beams are involved. User can involve all beams if desired.

How is Mz Transferred to the Pier? Enter 1 if by Exterior Beams Only Enter 2 if All Beams Participate	1

Downstation Pier View



Upstation Pier View



Export Wind on Live Load to Text Files

The results shown in the tables below are split into 4 groups based on combinations of the ± wind force and ± wind angle. In RC-Pier the user can vary the wind angle magnitude and sign. The wind force direction is typically varied automatically because the load, by default, is treated as a reversible load.

For Strength 5 and Service 1 Loading

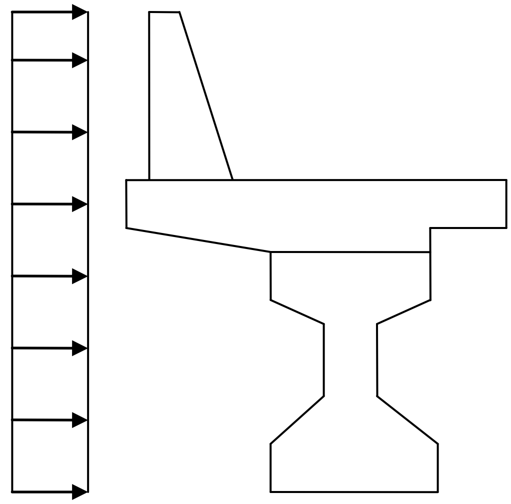
Beam #	WL Loads to the Pier (kips)											
	+ Wind Force, + Wind Angle			+ Wind Force, - Wind Angle			- Wind Force, + Wind Angle			- Wind Force, - Wind Angle		
	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz
1	-1.501	-2.502	-0.031	-1.065	-1.775	-1.058	1.501	2.502	0.031	1.065	1.775	1.058
2	-1.501	0.000	-0.031	-1.065	0.000	-1.058	1.501	0.000	0.031	1.065	0.000	1.058
3	-1.501	0.000	-0.031	-1.065	0.000	-1.058	1.501	0.000	0.031	1.065	0.000	1.058
4	-1.501	0.000	-0.031	-1.065	0.000	-1.058	1.501	0.000	0.031	1.065	0.000	1.058
5	-1.501	0.000	-0.031	-1.065	0.000	-1.058	1.501	0.000	0.031	1.065	0.000	1.058
6	-1.501	2.502	-0.031	-1.065	1.775	-1.058	1.501	-2.502	0.031	1.065	-1.775	1.058
7												
8												
9												
10												
		Mx (k*ft)	-2.104		Mx (k*ft)	-70.870		Mx (k*ft)	2.104		Mx (k*ft)	70.870

Bridge office practice is to set the overturning moment Mx to 0.00. Currently, RC-Pier does not calculate an Mx for WL, however, Leap's plans are to include it in a future version (probably as an option).

These loads will have to be used twice in RC-Pier. WL is dependent on WS and WS is applied with and without uplift.

Wind on Superstructure   Sample Calculations

Uplift = 20 psf
Lateral Pressure = 50 psf
Longitudinal Pressure = 12 psf



Superstructure Wind Area

SBC Height	34”
Slab Thickness	8”
Beam Height	54”
Total	96” = 8.00’

Average Span Length = (0.5)*(50.75’ + 116.5’) = 83.625’

$F_{x'}$ = (-0.050 ksf)*(8’)*(83.625’) = -33.450 k
$F_{z'}$ = (0.012 ksf)*(8’)*(83.625’) = -8.028 k

F_x = (-33.450 k)*(cos(23 deg)) - (8.028 k)*(sin(23 deg)) = -33.928 k
F_z = (-33.450 k)*(sin(23 deg)) + (8.028 k)*(cos(23 deg)) = -5.680 k

F_x per beam = (-33.928 k)/(6 Beams) = -5.655 k
F_z per beam = (-5.680 k)/(6 Beams) = -0.947 k

Assigning the sign to the results is done by observation.

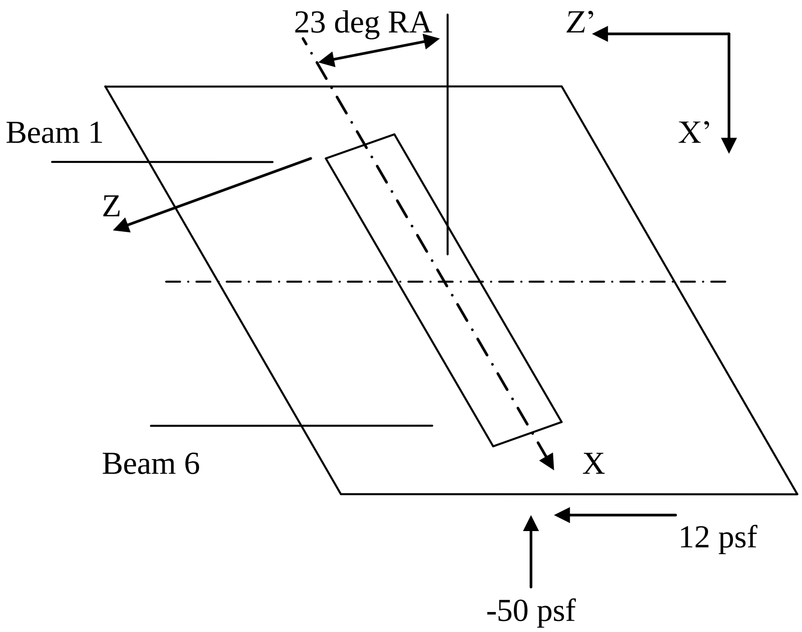
Uplift = (0.02 ksf)*(43.16)*(83.625’) = 72.185 k

F_y for Beams 2-5 = (72.185 k)/(6 Beams) = 12.031 k

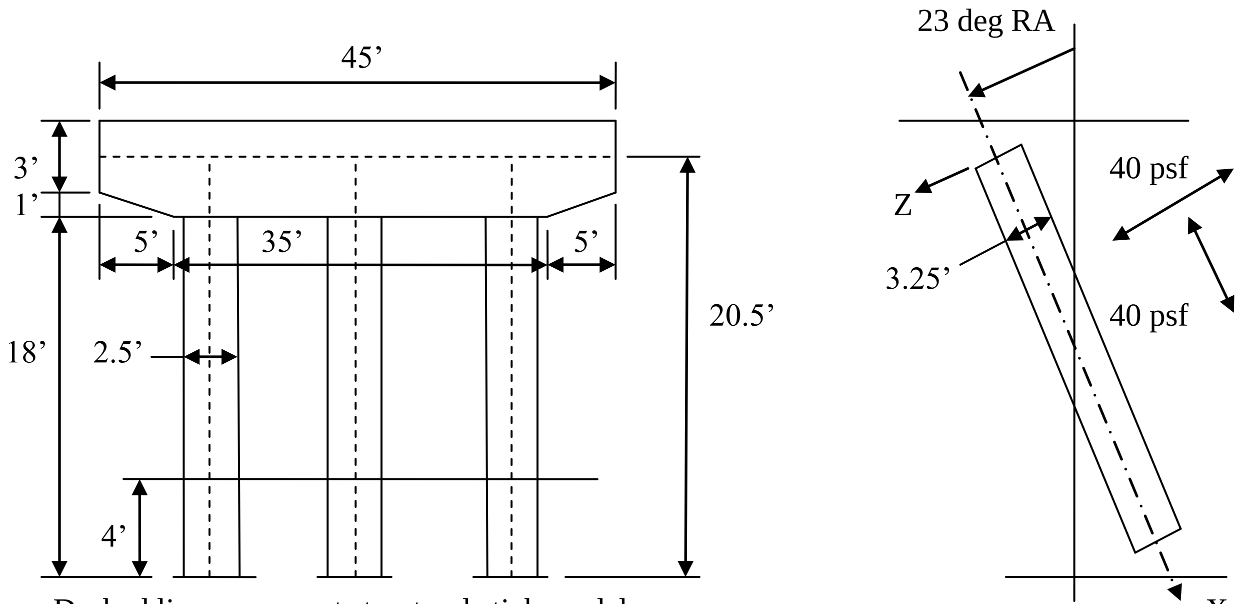
M_z = (33.928 k)*(0.5)*(8’) + (72.185 k)*(0.25)*(43.16’) = 914.587 k*ft

F_y for Beams 1 and 6 = ±(914.587 k*ft)/[(5 Beam Spa)*(7.4’)/(cos(23 deg))] + 12.03 k
Beam 1, F_y = -10.723 k
Beam 6, F_y = 34.784 k

M_x = (-5.680 k)*(0.5)*(8’) = -22.721 k*ft   ← Office Policy is to ignore



Wind on Substructure   Sample Calculations



Dashed lines represent structural stick model

The signs for simplified substructure wind loads are made to correspond with the sign for the superstructure wind loads.

$Acap_x$ = (4’)*(3.25’) = 13 ft^2
$Acap_z$ = (45’)*(4’) – (5’)*(1’) = 175 ft^2
Acol = (2.5’)*(18’ – 4’ fill) = 35 ft^2 per column (exposed)

$Fcap_x$ = (0.04 ksf)*(13 ft^2) = -0.52 k   Signs based on 1^{st} case sign convention
$Fcap_z$ = (0.04 ksf)*(175 ft^2) = -7.00 k   for superstructure wind loads.

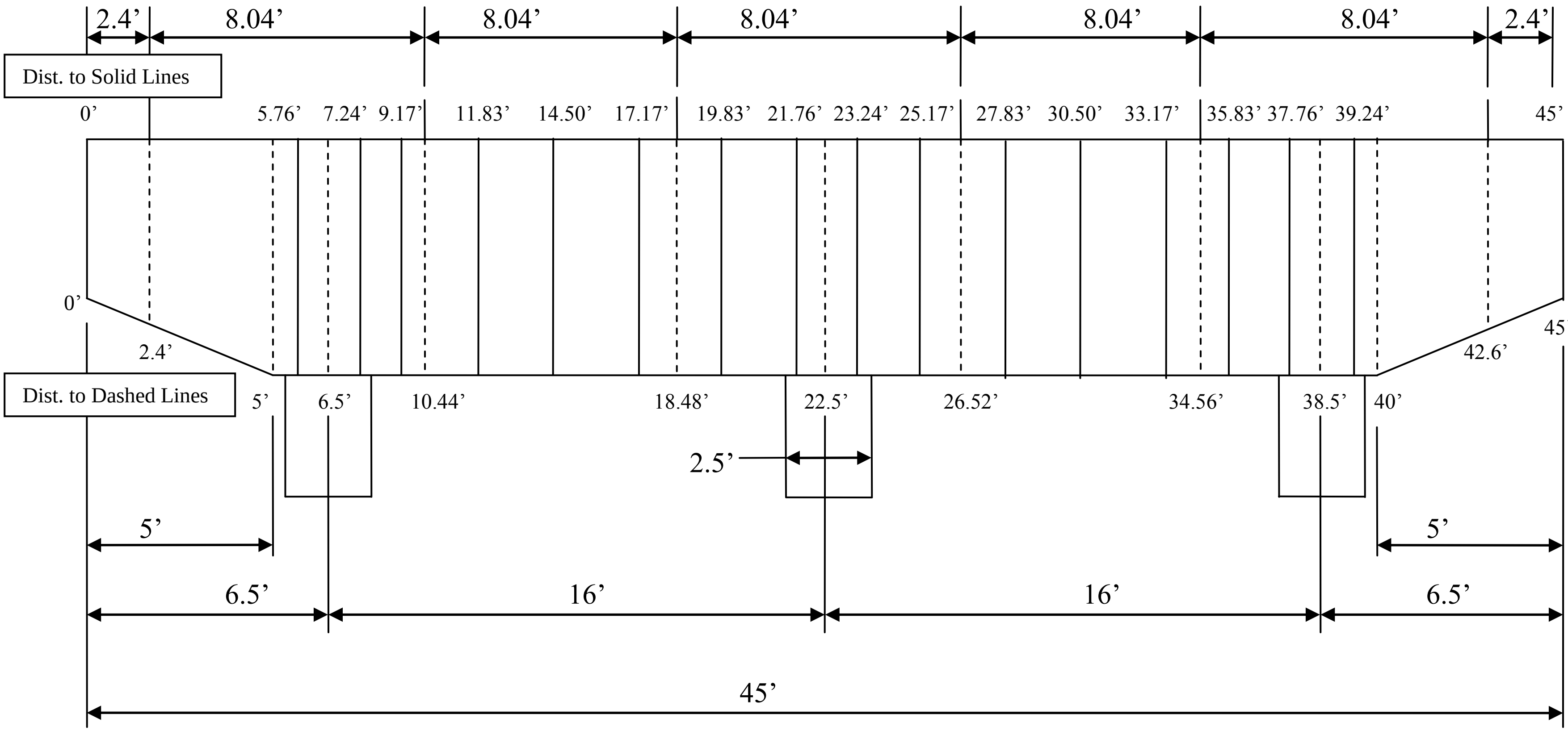
$Fcol_x$ = (0.04 ksf)*(35 ft^2) = -1.40 k
$Fcol_z$ = (0.04 ksf)*(35 ft^2) = -1.40 k

Cap Loads F_x = -0.52 k applied at midpoint of cap
UDL_z = (-7.00 k)/(45’) = -0.156 klf applied along cap length

Column Load UDL_x = (-1.40 k)/(14’) = -0.100 klf from (4’)/(20.5’) = 0.195
UDL_z = (-1.40 k)/(14’) = -0.100 klf to (18’)/(20.5’) = 0.878
Loads are applied to each column.

Cap and Column Design

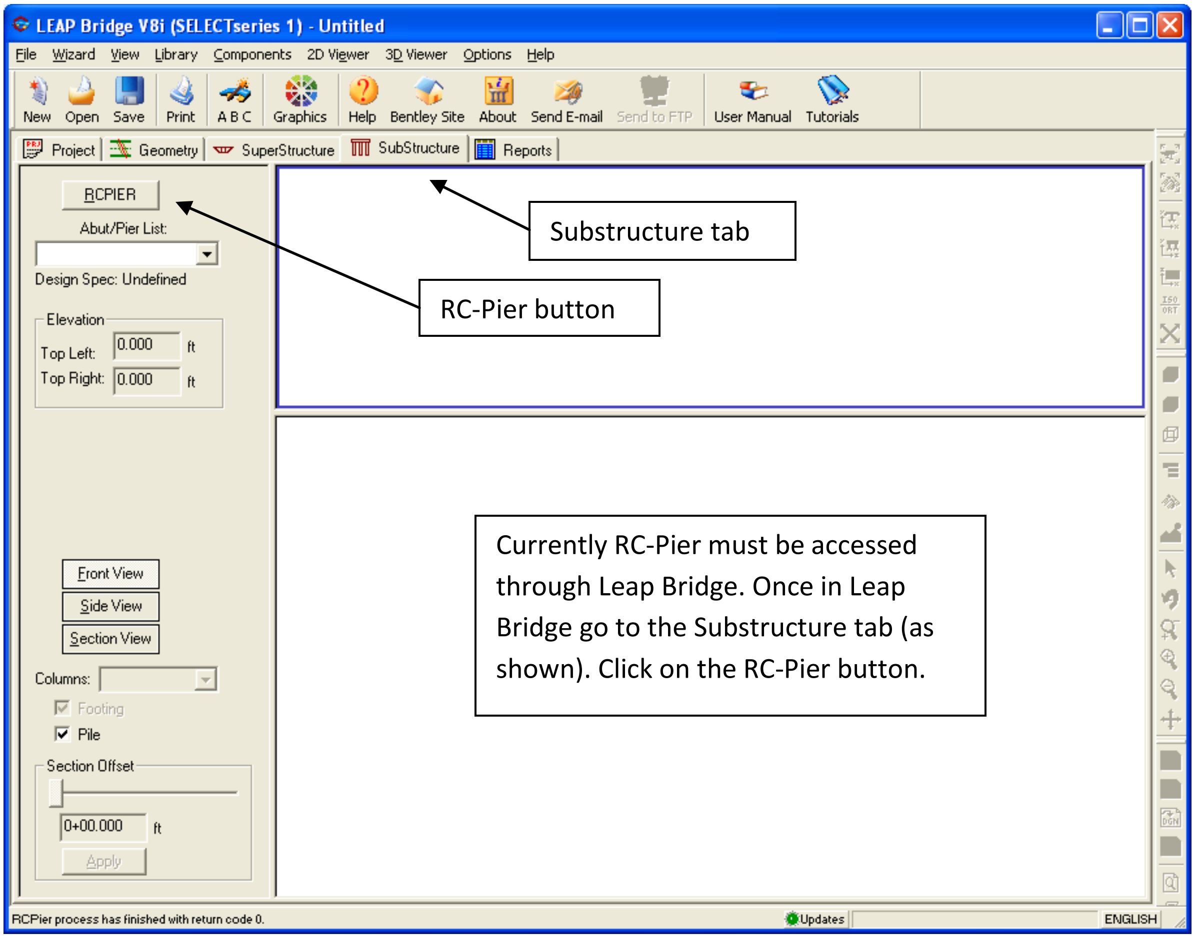
Beam Cap Check Points

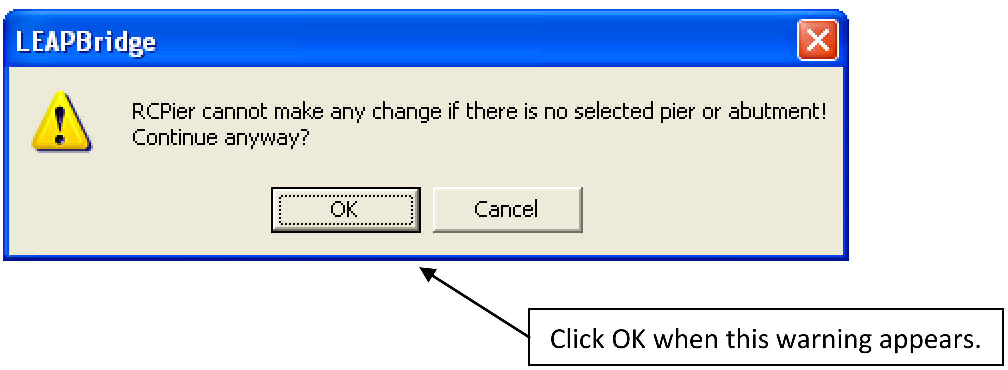


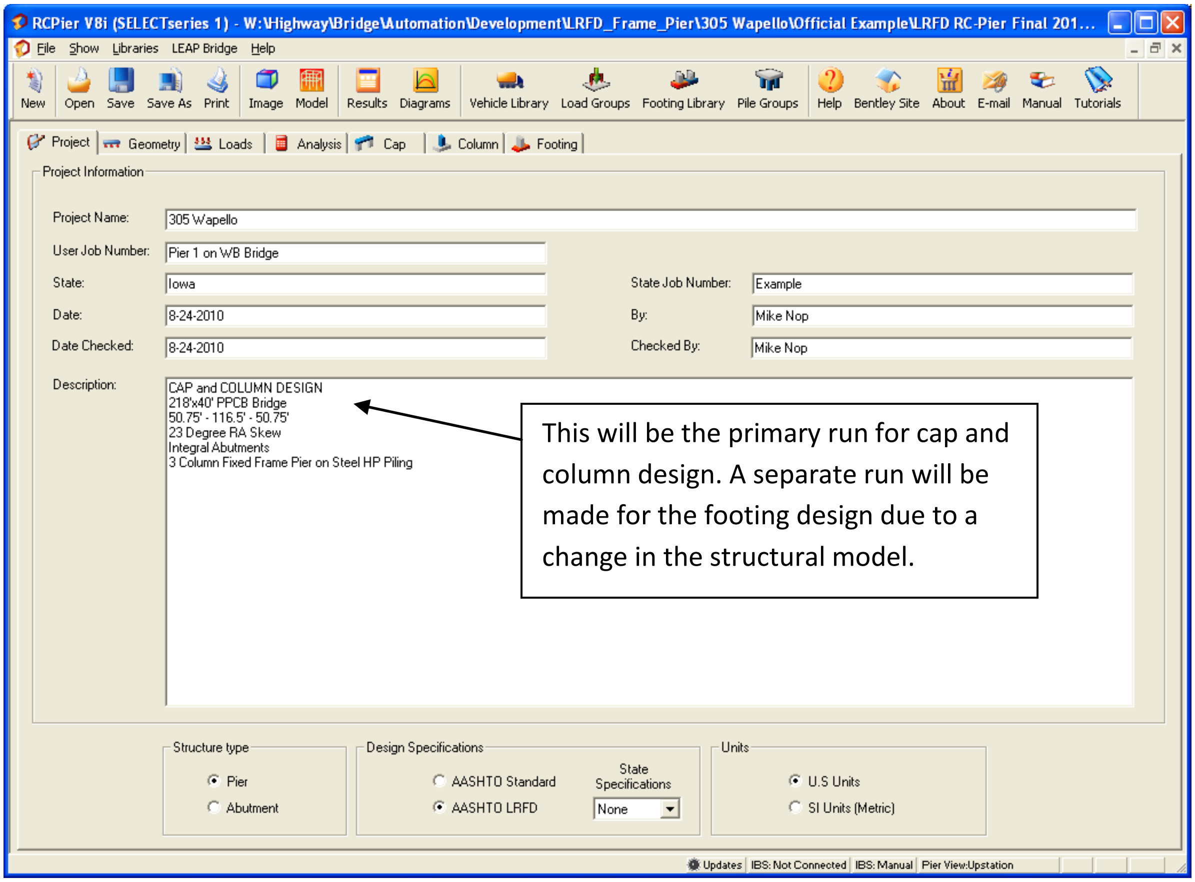
Dashed lines are automatically set up as cap nodal points by RC-Pier.

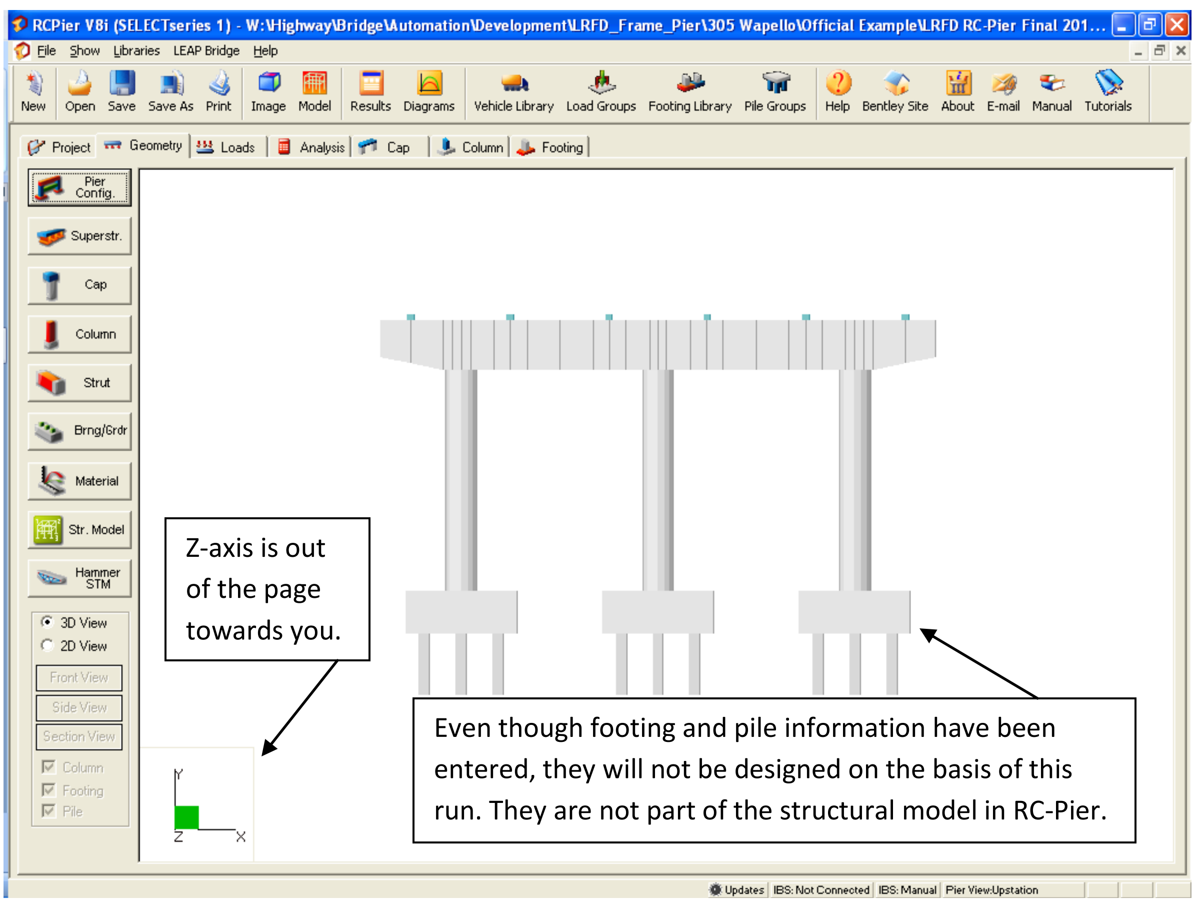
The solid lines are additional check points that have been added. These include 6th points between c.l. columns. The critical section points at W/3 from the c.l. of the columns have also been included by using “Offset from C.L. of the column” in RC-Pier.

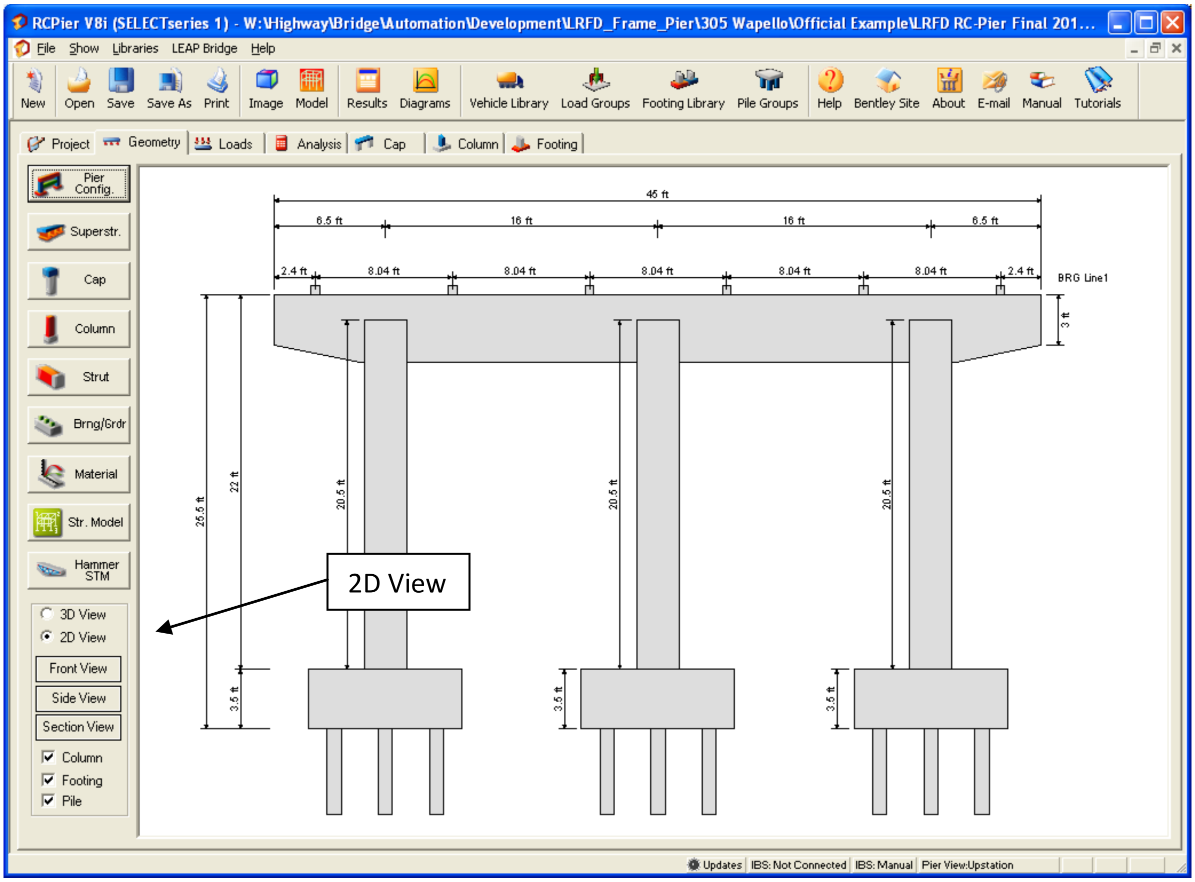
$W = [\pi*(0.5*2.5')^2]^{0.5} = 2.216'$ $\qquad$ $W/3 = 0.74'$

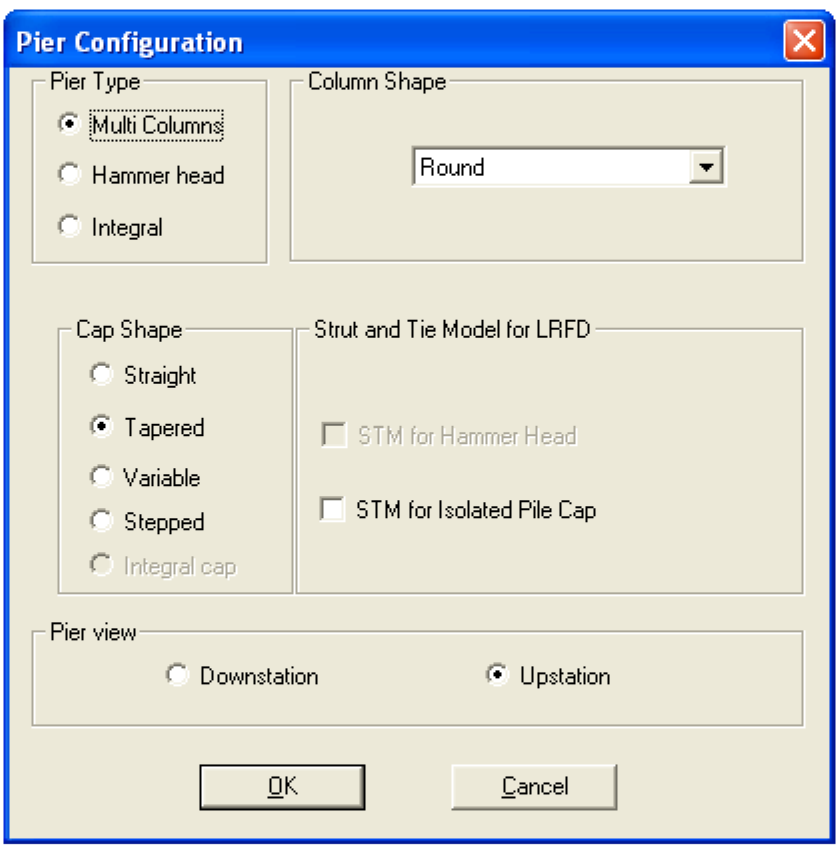

LEAP Bridge V8i (SELECTseries 1) - Untitled
File Wizard View Library Components 2D Viewer 3D Viewer Options Help
New Open Save Print A B C Graphics Help Bentley Site About Send E-mail Send to FTP User Manual Tutorials
Project Geometry SuperStructure SubStructure Reports
RCPIER
Abut/Pier List:
Design Spec: Undefined
Elevation
Top Left: 0.000 ft
Top Right: 0.000 ft
Front View
Side View
Section View
Columns:
Footing
Pile
Section Offset
0+00.000 ft
Apply
RCPier process has finished with return code 0.
Updates
ENGLISH
Substructure tab
RC-Pier button
Currently RC-Pier must be accessed through Leap Bridge. Once in Leap Bridge go to the Substructure tab (as shown). Click on the RC-Pier button.


LEAPBridge
RCPier cannot make any change if there is no selected pier or abutment! Continue anyway?
OK
Cancel
Click OK when this warning appears.


RCPier V8i (SELECTseries 1) - W:\Highway\Bridge\Automation\Development\LRFD_Frame_Pier\305 Wapello\Official Example\LRFD RC-Pier Final 201...
File Show Libraries LEAP Bridge Help
New Open Save Save As Print Image Model Results Diagrams Vehicle Library Load Groups Footing Library Pile Groups Help Bentley Site About E-mail Manual Tutorials
Project Geometry Loads Analysis Cap Column Footing
Project Information
Project Name: 305 Wapello
User Job Number: Pier 1 on WB Bridge
State: Iowa
State Job Number: Example
Date: 8-24-2010
By: Mike Nop
Date Checked: 8-24-2010
Checked By: Mike Nop
Description:
CAP and COLUMN DESIGN
218'x40' PPCB Bridge
50.75' - 116.5' - 50.75'
23 Degree RA Skew
Integral Abutments
3 Column Fixed Frame Pier on Steel HP Piling
This will be the primary run for cap and column design. A separate run will be made for the footing design due to a change in the structural model.
Structure type
Pier
Abutment
Design Specifications
AASHTO Standard
AASHTO LRFD
State Specifications
None
Units
U.S Units
SI Units (Metric)
Updates IBS: Not Connected IBS: Manual Pier View:Upstation

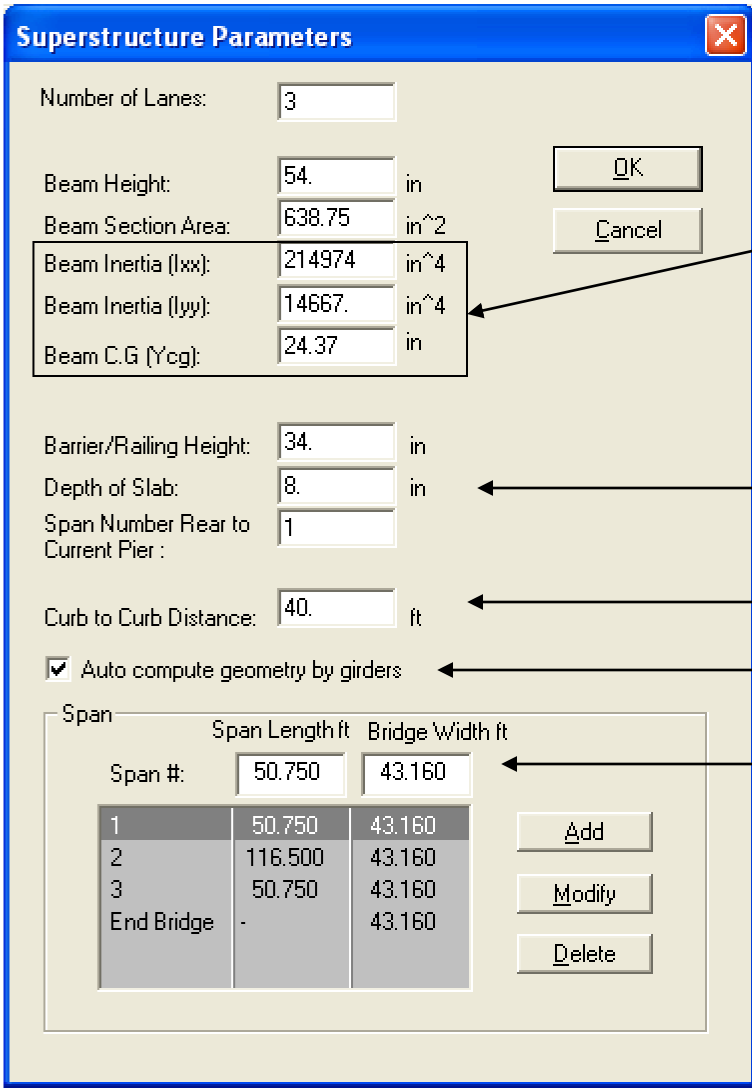

RCPier V8i (SELECTseries 1) - W:\Highway\Bridge\Automation\Development\LRFD_Frame_Pier\305 Wapello\Official Example\LRFD RC-Pier Final 201...
File Show Libraries LEAP Bridge Help
New Open Save Save As Print Image Model Results Diagrams Vehicle Library Load Groups Footing Library Pile Groups Help Bentley Site About E-mail Manual Tutorials
Project Geometry Loads Analysis Cap Column Footing
Pier Config.
Superstr.
Cap
Column
Strut
Brng/Grdr
Material
Str. Model
Hammer STM
3D View
2D View
Front View
Side View
Section View
Column
Footing
Pile
Z-axis is out of the page towards you.
Y
Z
X
Even though footing and pile information have been entered, they will not be designed on the basis of this run. They are not part of the structural model in RC-Pier.
Updates IBS: Not Connected IBS: Manual Pier View:Upstation





Typical frame piers in Iowa have round columns and tapered pier caps.

I recommend using the "Upstation" pier view option.



This screen only needs to be filled out if you intend to auto-generate loads.

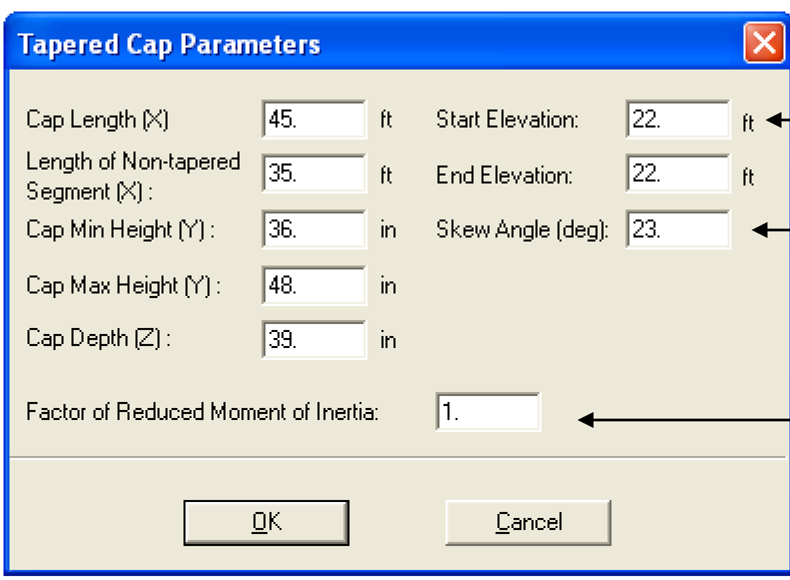
These entries are only important for auto-generation of EQ loads.

There are no entries for haunch thickness, pad thickness, or step height. This means auto-generated wind area excludes haunch. Also, lateral superstructure loads can’t be elevated further above top of pier cap.

Gutter to Gutter

Not sure what this check box does.

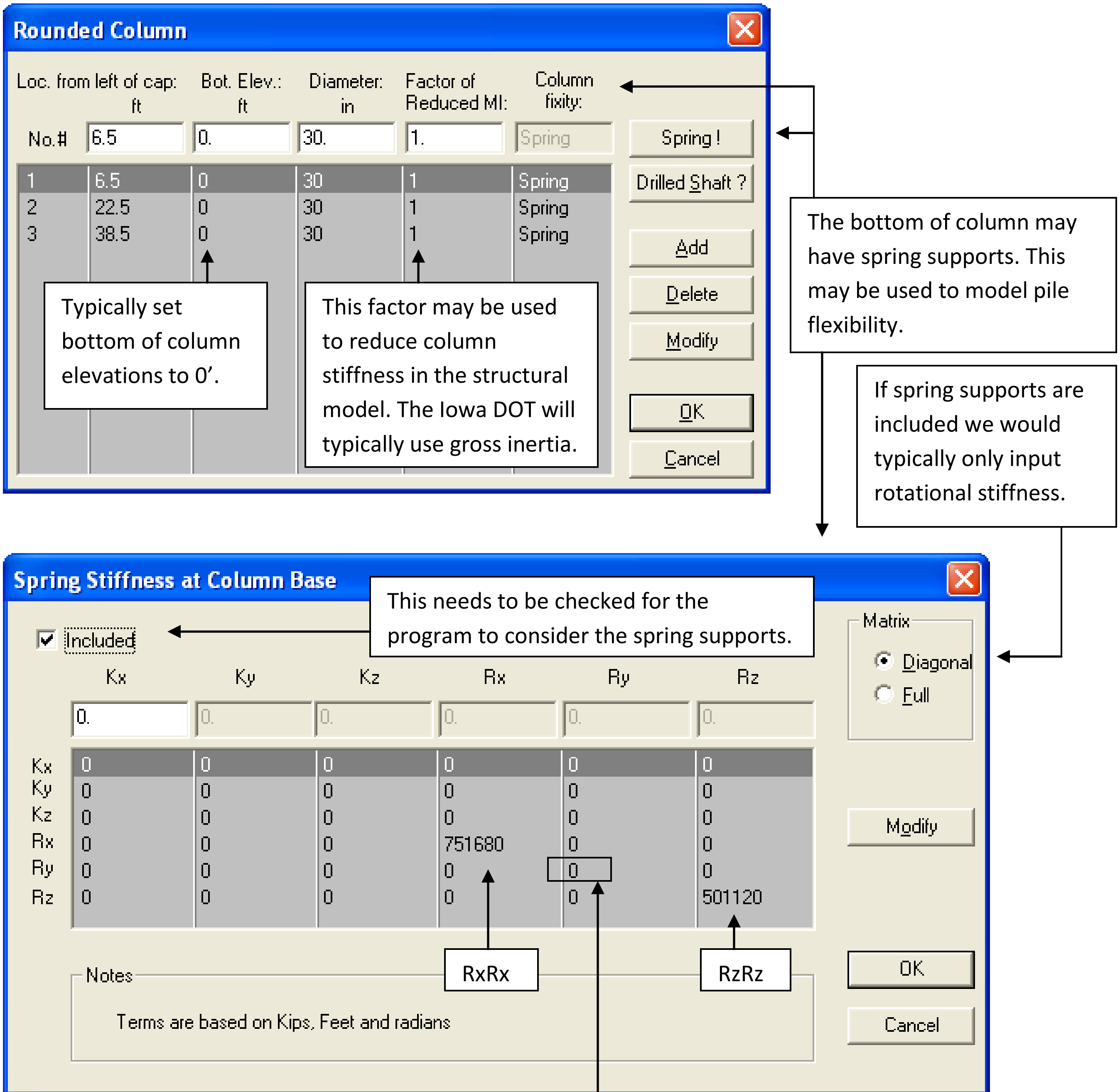
Bridge width was added to allow for flared girders.



Recommend setting bottom of column elevation at 0’. This way the (top of) cap elevation will simply be the height from the bottom of the column to the top of the cap.

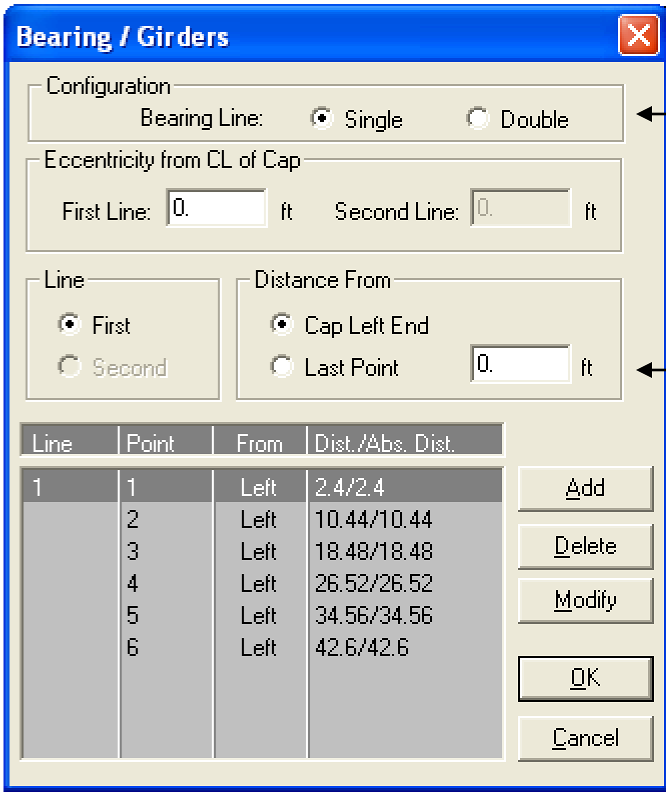
Right ahead skews are positive. The skew angle only needs to be input if you intend to auto-generate loads.

This factor may be used to reduce member stiffness in the structural model (i.e. simulate a cracked section). The Iowa DOT will typically use gross inertia.



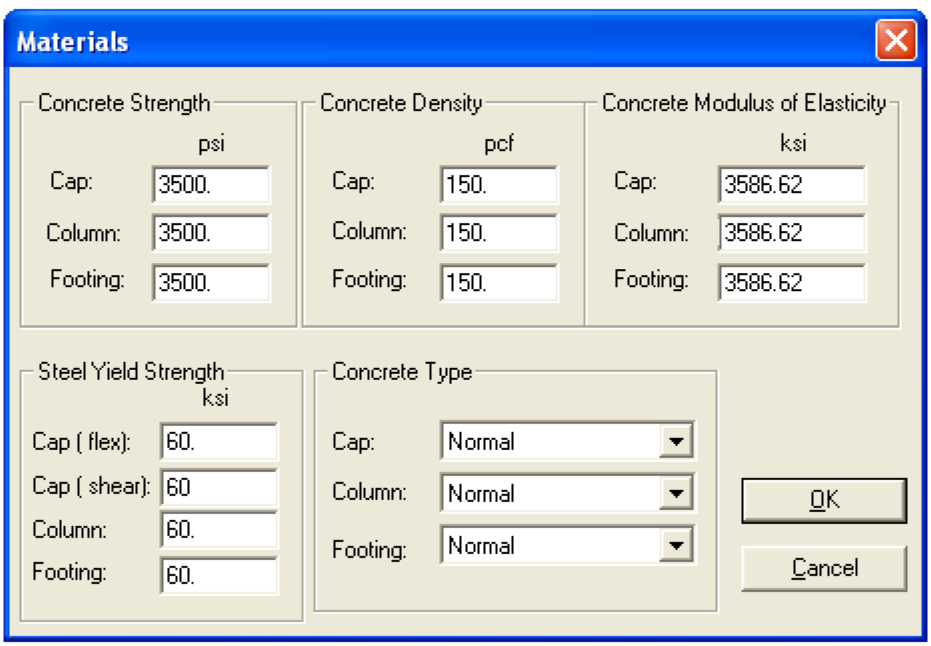
A value of 0 on the diagonal means that degree-of-freedom is fixed. This a bit counter-intuitive because entering extremely large values produces a condition that is nearly fixed while entering extremely small values essentially releases the restraint.

As stated earlier the designer should generally assume a fully-fixed condition, but we will illustrate a partially fixed condition in this example.

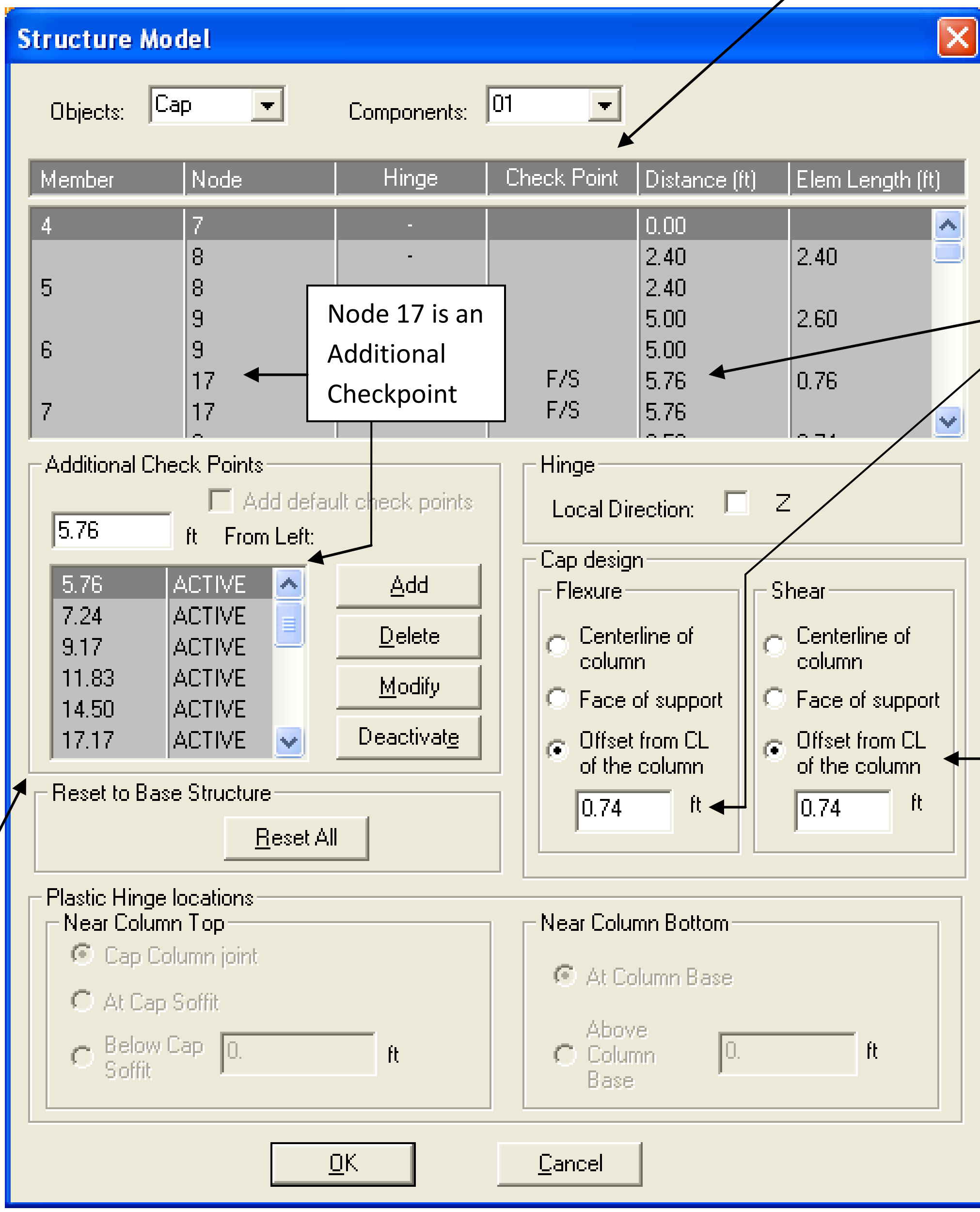


Bearing lines will typically be modeled as "Single" for both steel and prestressed bridges. Exceptions may include situations that yield a significant unsymmetrical loading about the c.l. of the cap such as would be caused by unbalanced spans or actual eccentric bearing lines. Variable width bridges that drop a beam line at a pier is another example.

Enter beam spacing along the skew of the pier cap.



Blank entries indicate a default check point generated automatically by RC-Pier. “F/S” of “f” indicates face of support points which may be generated using the option buttons under “Cap design”. Entries with an “*” indicate “Additional Check Points” that have been entered directly by the user.

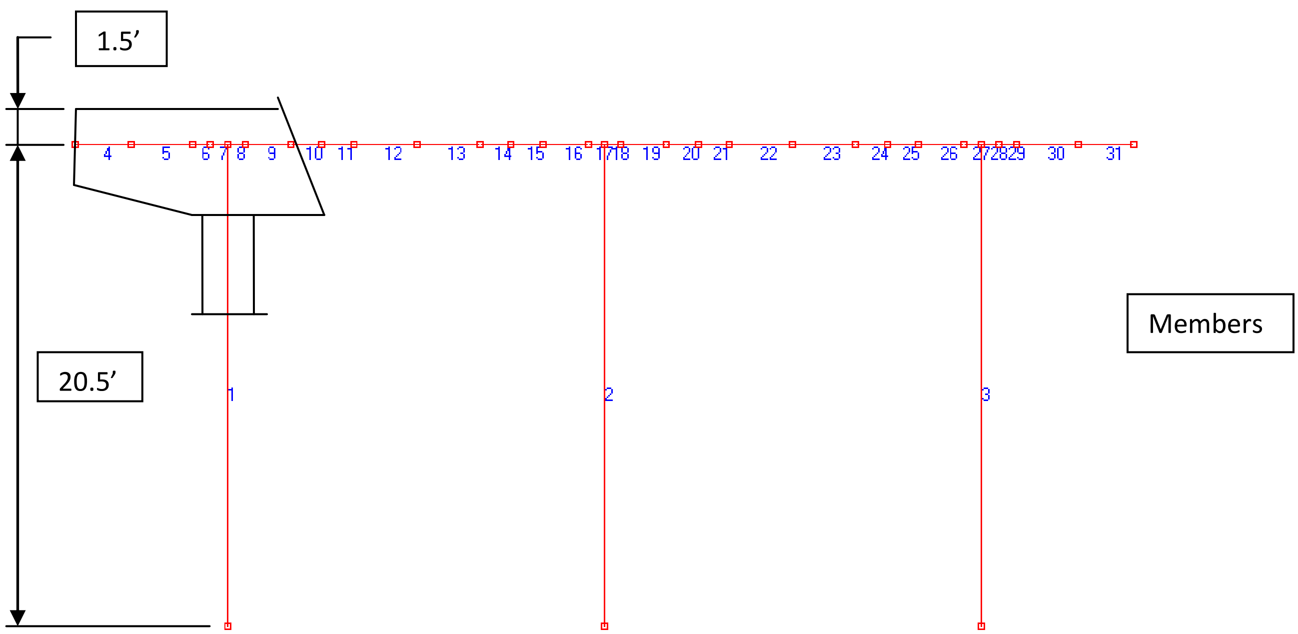


6.50’ overhang
- 0.74’ offset
5.76’

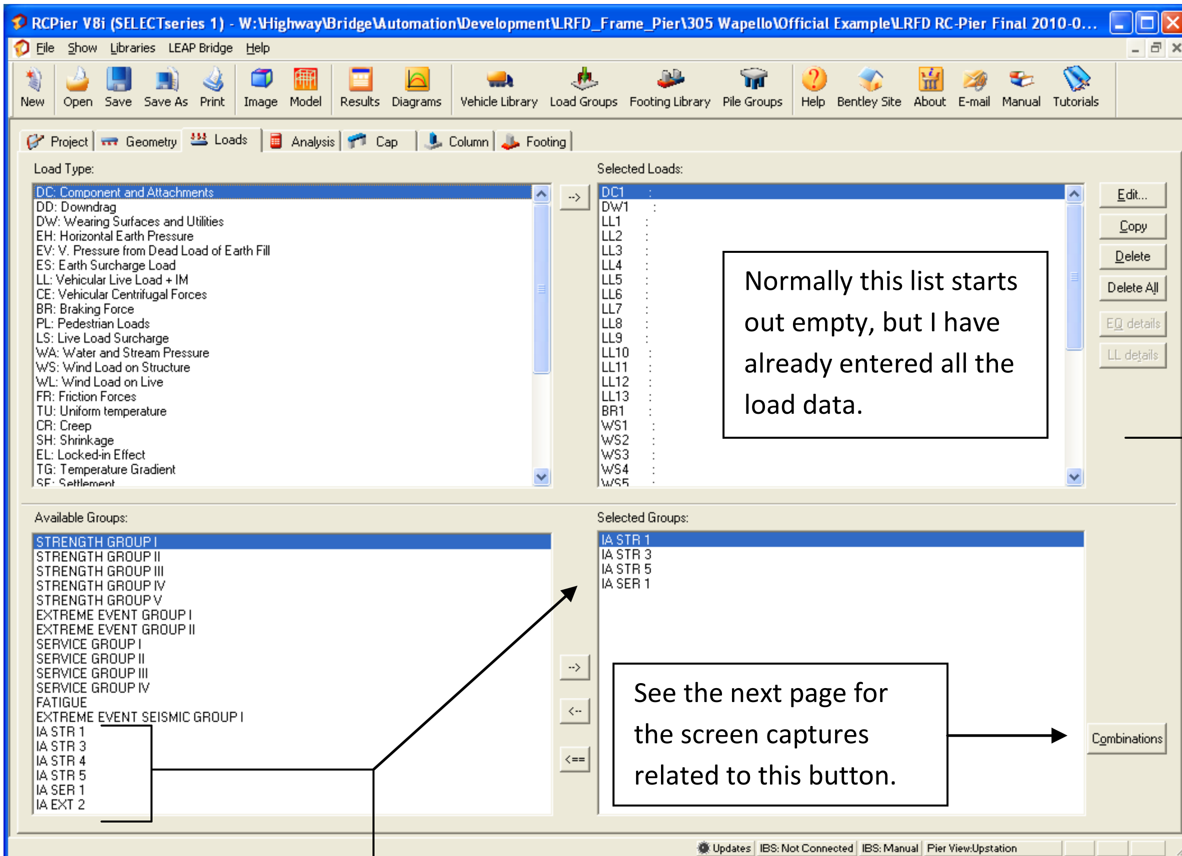
This 3rd option allows us to create additional check points on each side of each column to act as the critical section or new “Face of support”. So here we may enter the W/3 distance.

One short-coming of this 3rd option is that the c.l. of the column is still used for a cap design point when designing cap R/I on the “Cap” tab. The 2nd option, “Face of Support”, suppresses the c.l. of the column point when doing cap design. It would be nice if the 3rd option followed the pattern of the 2nd option.

I have placed additional check points at the sixth points between c.l. columns in order to allow for a more complete design of the pier cap. There is a plot of nodal points on the next page. I didn’t add additional points on the cantilever since that design is typically done by spreadsheet. However, the user could do a separate RC-Pier run for the cantilever with the special loading requirements and with additional points on the cantilever.



The c.l. of the cap for the structural model is placed at the midpoint of the minimum cap height. The minimum cap height is the end cap taper height.

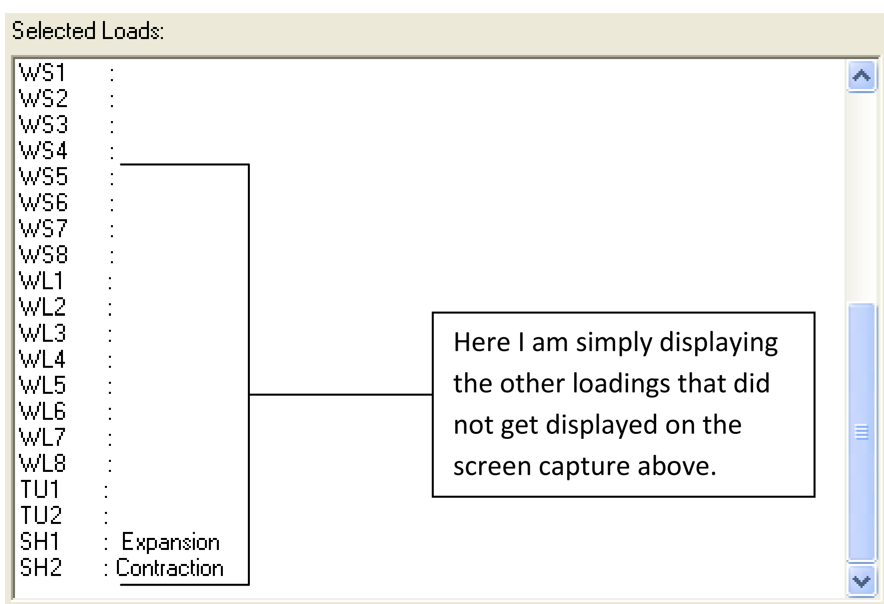


Normally this list starts out empty, but I have already entered all the load data.

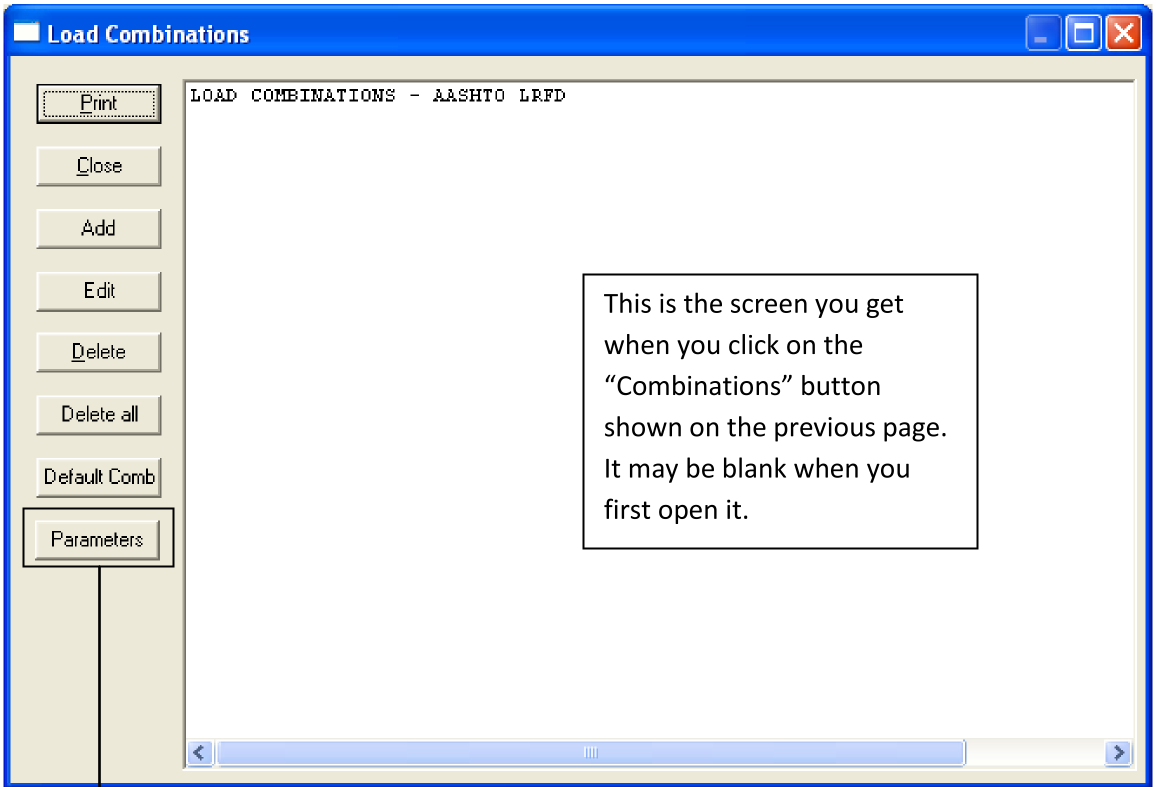
See the next page for the screen captures related to this button.

The Iowa load groups correspond with the default load groups except that the reversible feature for the wind loads has been set to uni-directional. This was done because wind uplift is uni-directional and should not be reversed as will be done in RC-Pier if the setting is not changed.

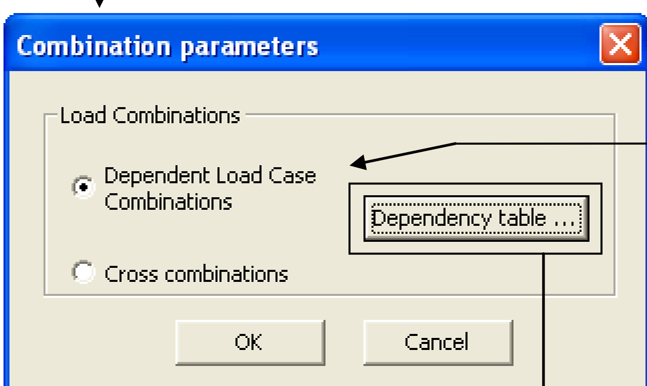
This example uses simplified wind loads. These loads will be entered twice – once without wind uplift (WS1 to WS4) and once with wind uplift (WS5 to WS8). Technically wind uplift only needs to be applied for the Strength 3 combination when the wind angle is 0 degrees. However, it is simpler and conservative to check each group with and without uplift. Since the WS loads are repeated the WL loads are also repeated due to the dependency between them.



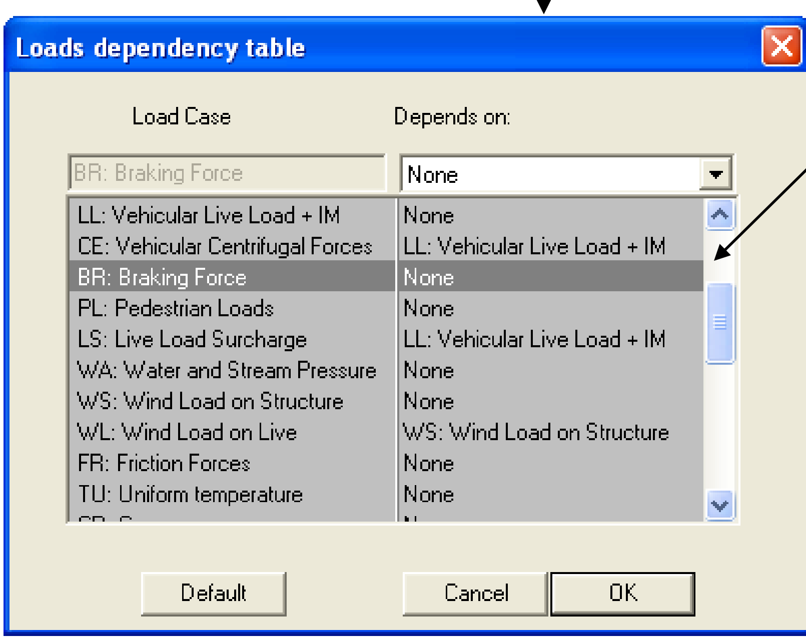
Here I am simply displaying the other loadings that did not get displayed on the screen capture above.



This is the screen you get when you click on the "Combinations" button shown on the previous page. It may be blank when you first open it.



Use "Dependent Load Case Combinations" since it will reduce the number of load cases by maintaining load dependencies.



RC-Pier defaults with a dependency of BR on LL. Often we set this to "None" because we typically use only the worst BR load irrespective of how many lanes of LL are on the bridge.

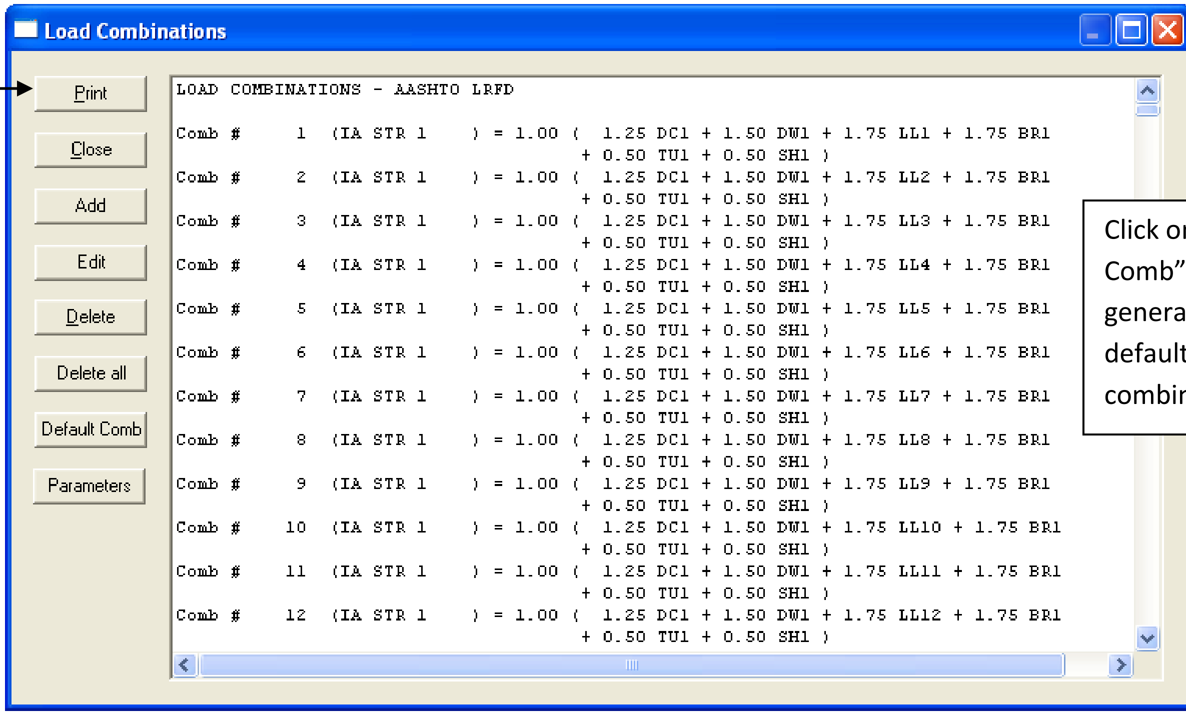
The only dependent loads we typically use for frame piers are:

CE to LL

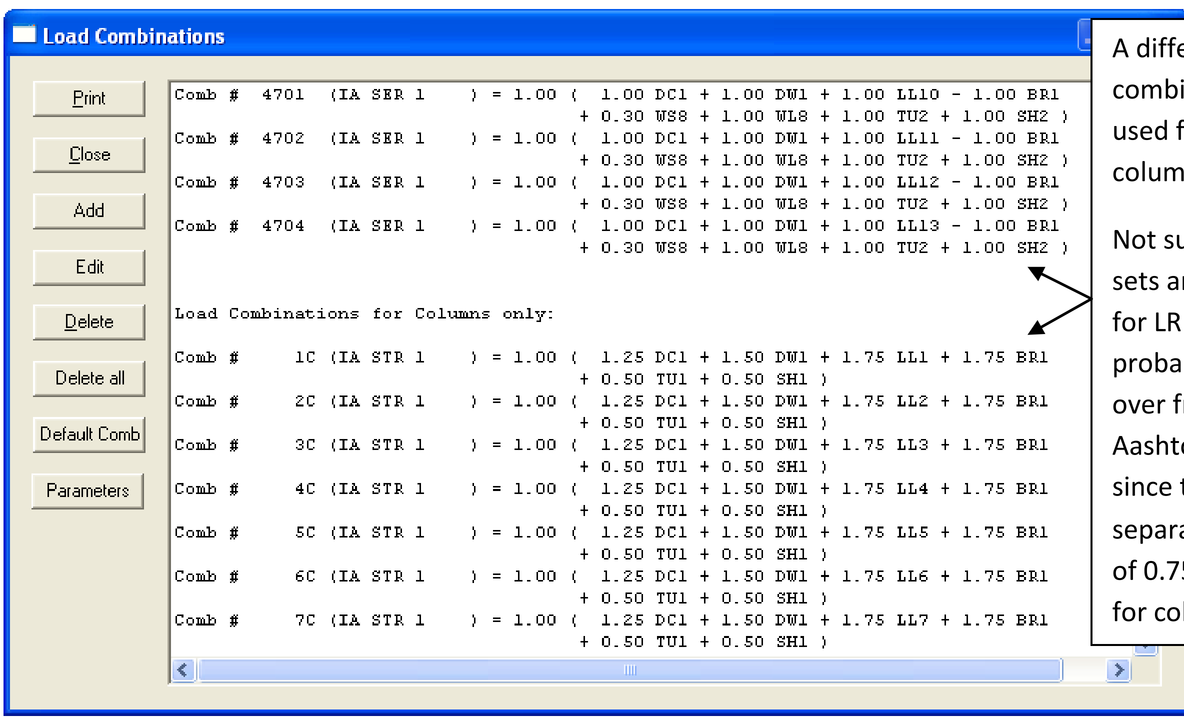
WL to WS

This means that if we have 8 WS cases then we must have 8 WL cases. Thus, WL1 is always and only associated with WS1, and WL2 is always and only associated with WS2, and so on.

CAUTION: Clicking “Print” sends ALL the load combination equations immediately to a printer.



Click on “Default Comb” to generate all the default load combinations.



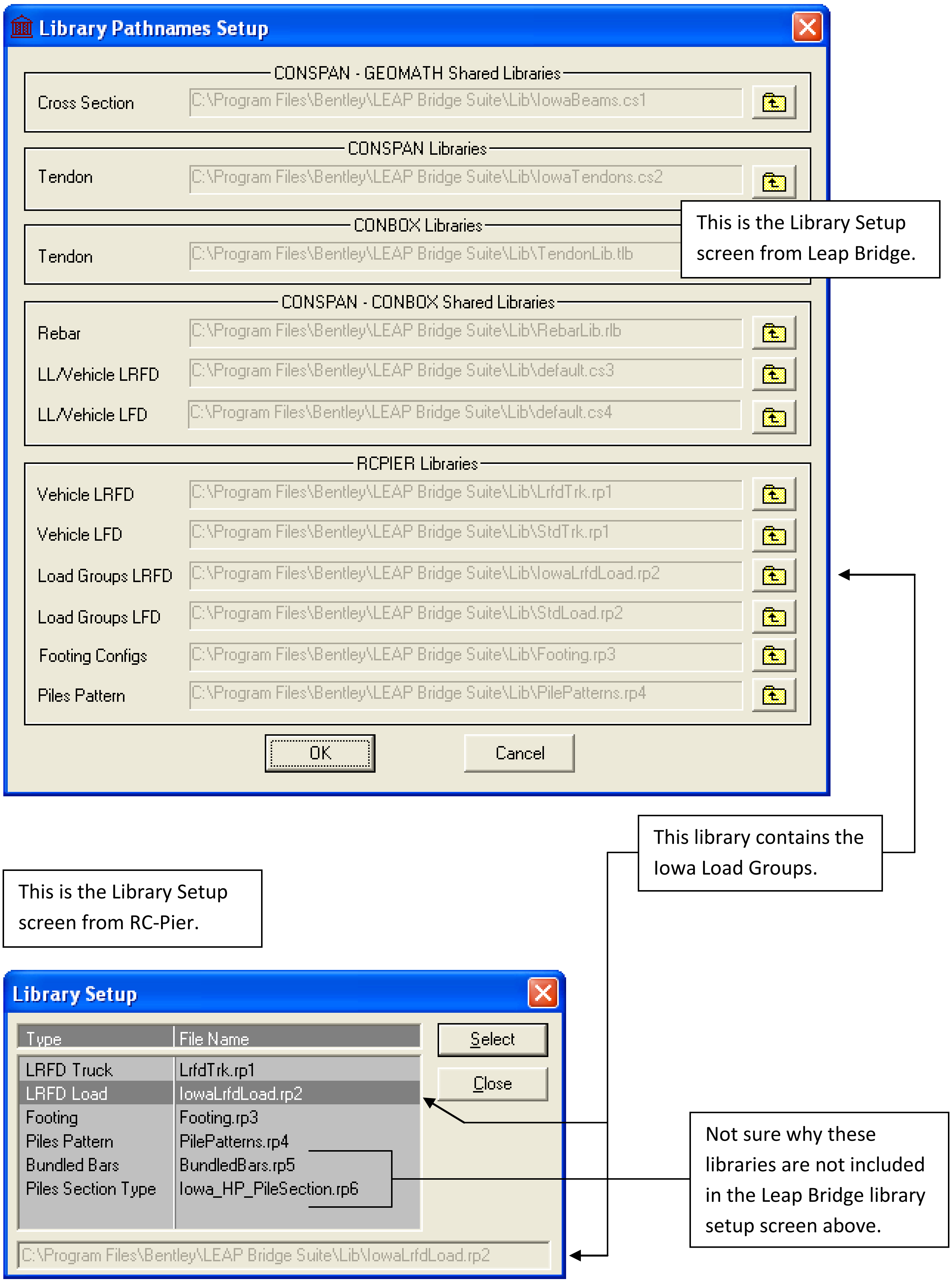
A different set of combinations is used for the column and cap.

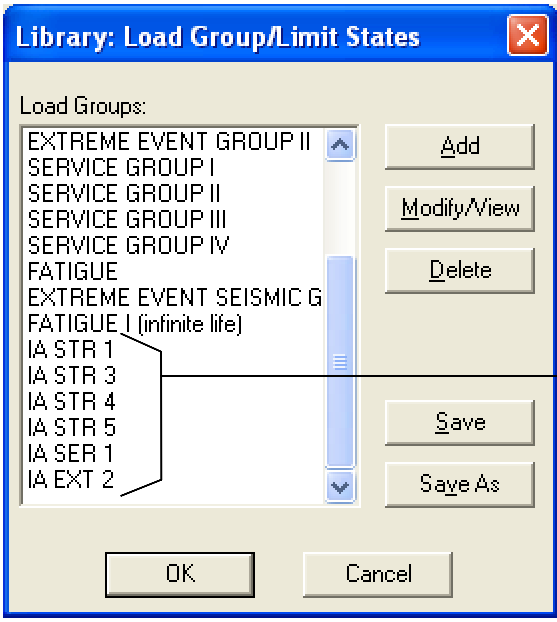
Not sure the two sets are necessary for LRFD. It is probably a hold-over from the Aashto Std. Spec. since there was a separate βd factor of 0.75 and 1.00 for columns.

Note: There are a number of features on this screen that will not be demonstrated in this example. For instance, you can

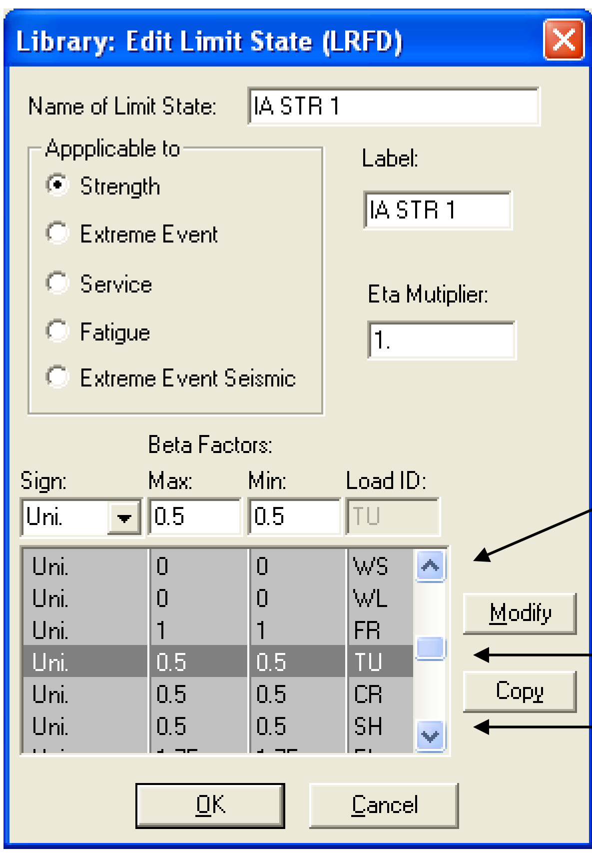
- delete combinations
- edit combinations
- add your own combinations

These features are particularly useful for testing loading scenarios and trouble-shooting problems.


Library Pathnames Setup
CONSPAN - GEOMATH Shared Libraries
Cross Section
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\IowaBeams.cs1
CONSPAN Libraries
Tendon
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\IowaTendons.cs2
CONBOX Libraries
Tendon
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\TendonLib.tlb
This is the Library Setup screen from Leap Bridge.
CONSPAN - CONBOX Shared Libraries
Rebar
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\RebarLib.rlb
LL/Vehicle LRFD
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\default.cs3
LL/Vehicle LFD
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\default.cs4
RCPIER Libraries
Vehicle LRFD
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\LrfdTrk.rp1
Vehicle LFD
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\StdTrk.rp1
Load Groups LRFD
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\IowaLrfdLoad.rp2
Load Groups LFD
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\StdLoad.rp2
Footing Configs
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\Footing.rp3
Piles Pattern
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\PilePatterns.rp4
OK
Cancel
This library contains the Iowa Load Groups.
This is the Library Setup screen from RC-Pier.
Library Setup
Type
File Name
Select
Close
LRFD Truck
LrfdTrk.rp1
LRFD Load
IowaLrfdLoad.rp2
Footing
Footing.rp3
Piles Pattern
PilePatterns.rp4
Bundled Bars
BundledBars.rp5
Piles Section Type
Iowa_HP_PileSection.rp6
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\IowaLrfdLoad.rp2
Not sure why these libraries are not included in the Leap Bridge library setup screen above.



The Iowa Load Groups

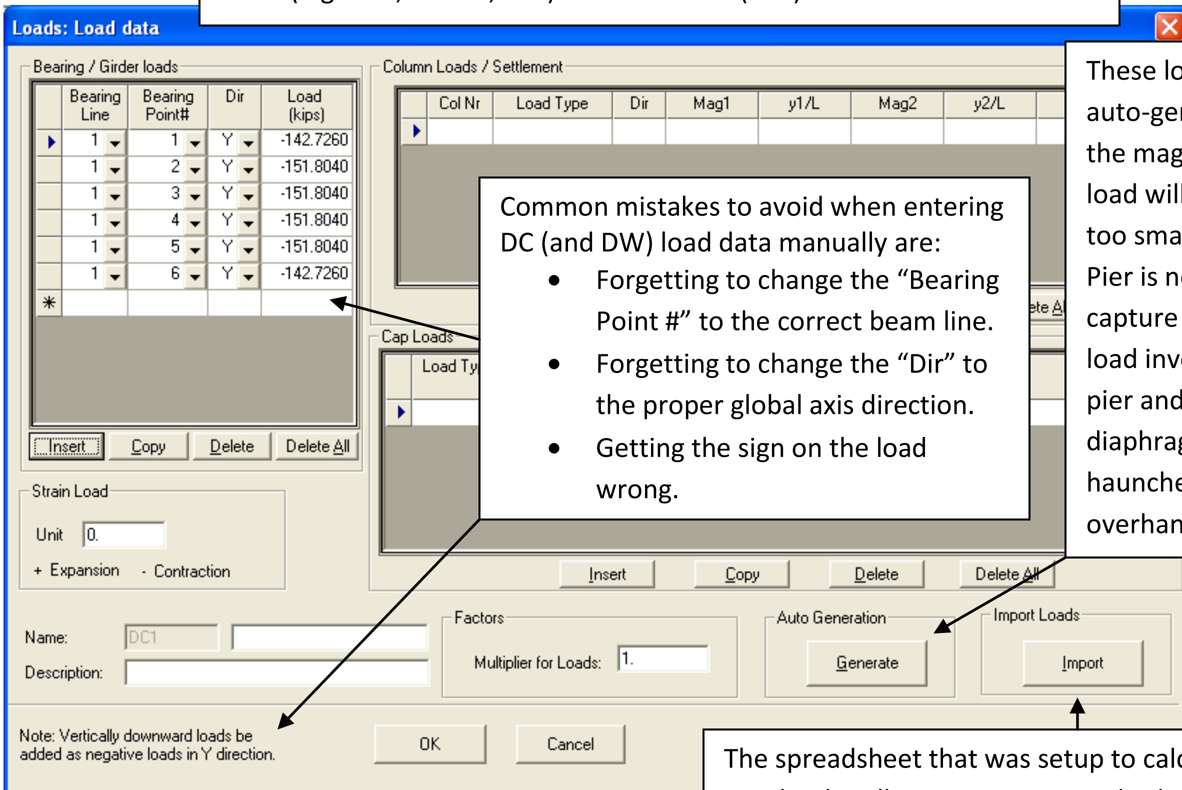


Notice that the wind loads have been switched from “Reversible” to “Uni-directional”

In general Iowa will use the smaller values for the TU and SH load factors along with gross inertia of the pier members. See Aashto Lrfd 3.4.1, the first paragraph on page 3-12 of the 5th edition (2010).

DC1

The “1” in “DC1” refers to the 1st load case number in the generic DC load type numbering sequence. So, the “DC1” load data includes DC1 loads (e.g. slab, beams, etc.) and DC2 loads (SBC).



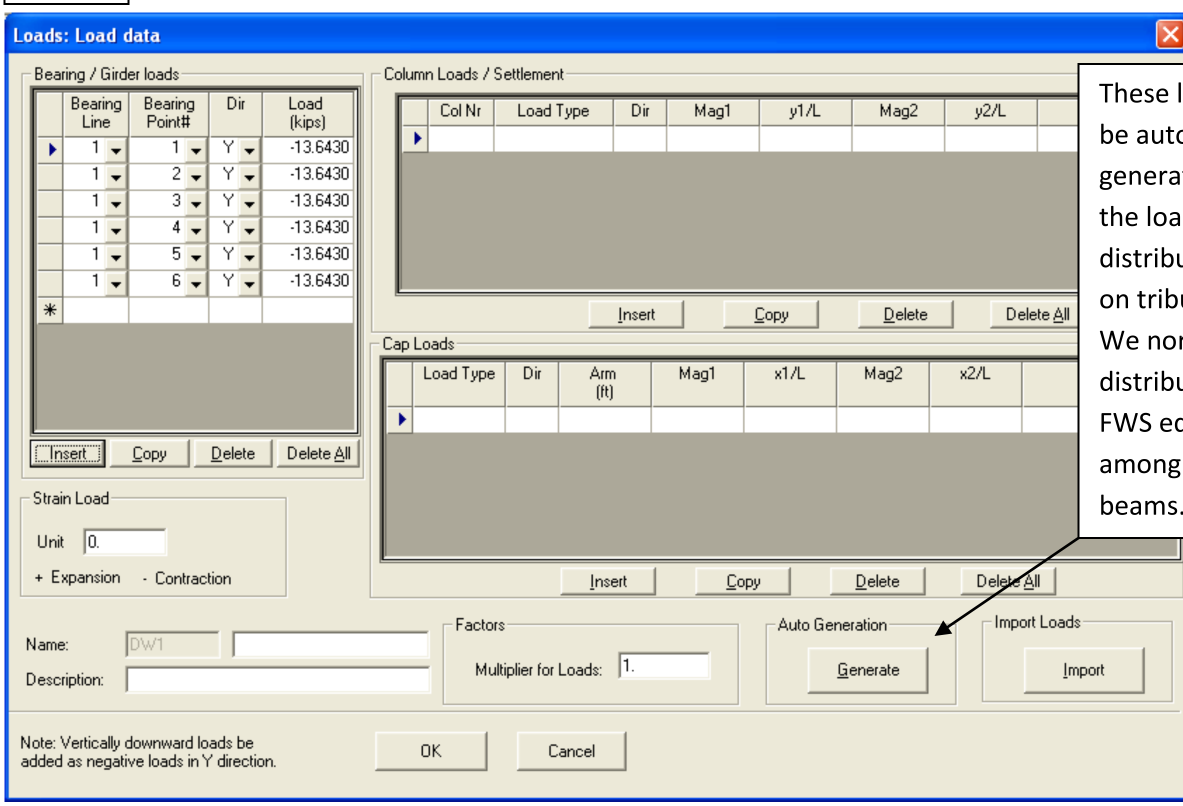
Common mistakes to avoid when entering DC (and DW) load data manually are:

- Forgetting to change the “Bearing Point #” to the correct beam line.
- Forgetting to change the “Dir” to the proper global axis direction.
- Getting the sign on the load wrong.

These loads can be auto-generated, but the magnitude of the load will typically be too small since RC-Pier is not able to capture all the dead load involved (e.g. pier and intermediate diaphragms, haunches, slab overhang thickening).

The spreadsheet that was setup to calculate DC1 loads will not generate standardized text files in order to import the loads. The DC (and DW) input is relatively minor for this load case.

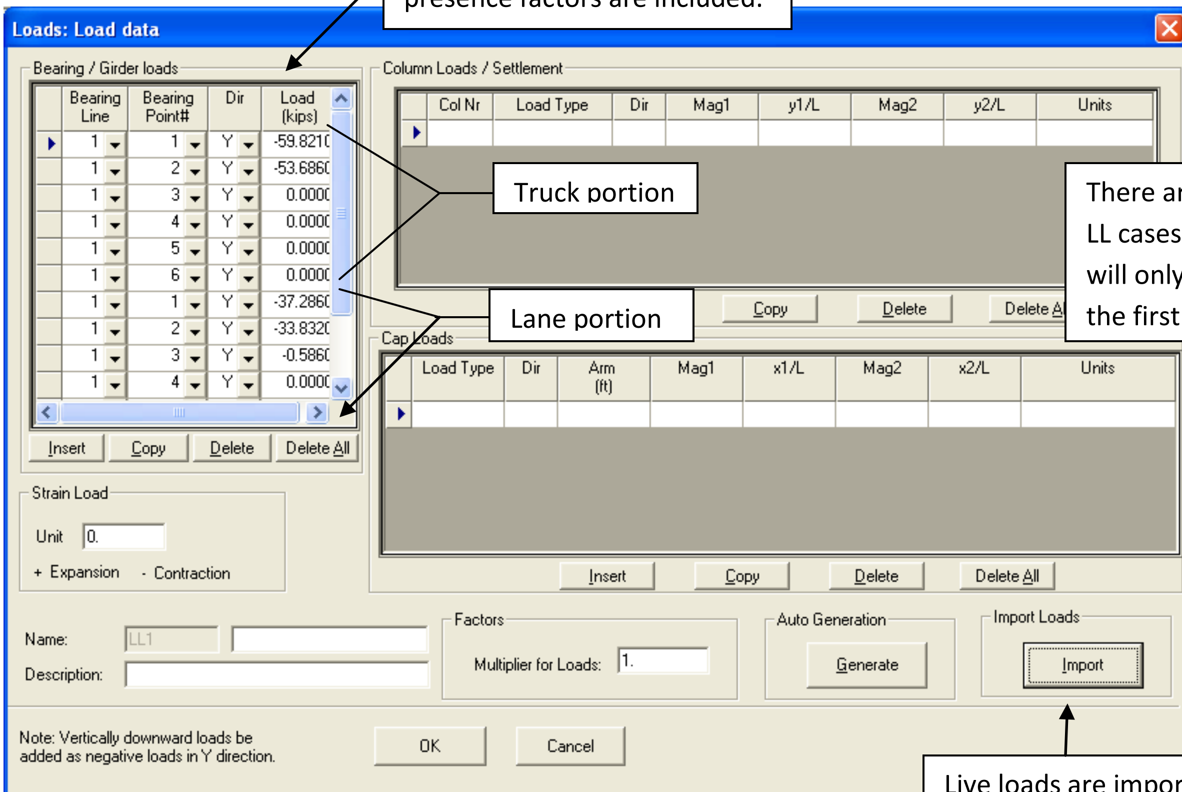
DW1



These loads can be auto-generated, but the load is distributed based on tributary area. We normally distribute the FWS equally among all the beams.

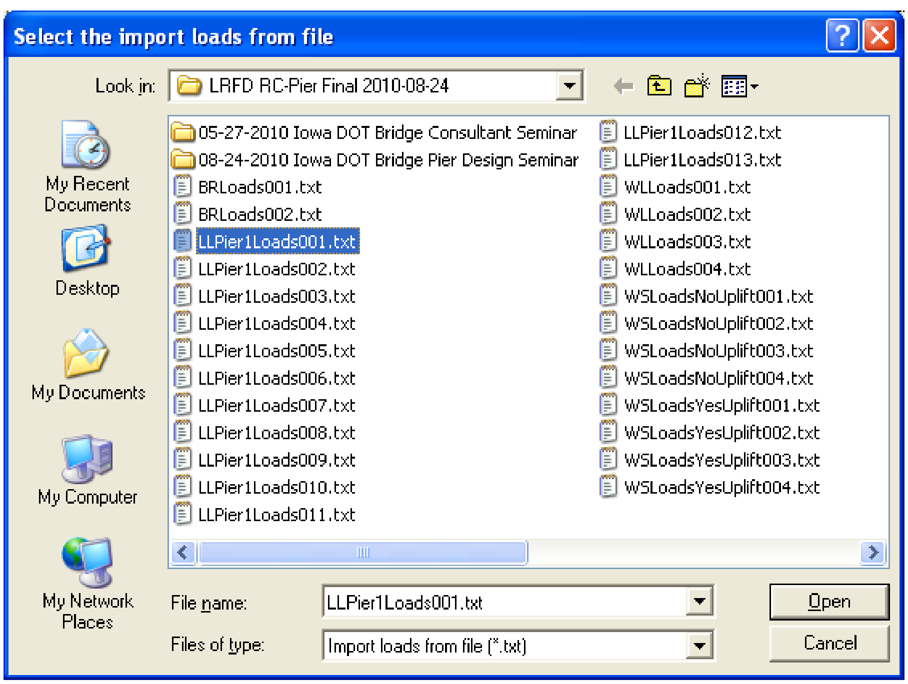
LL1

Impact is excluded. Multiple presence factors are included.



There are 13 LL cases, but I will only show the first one.

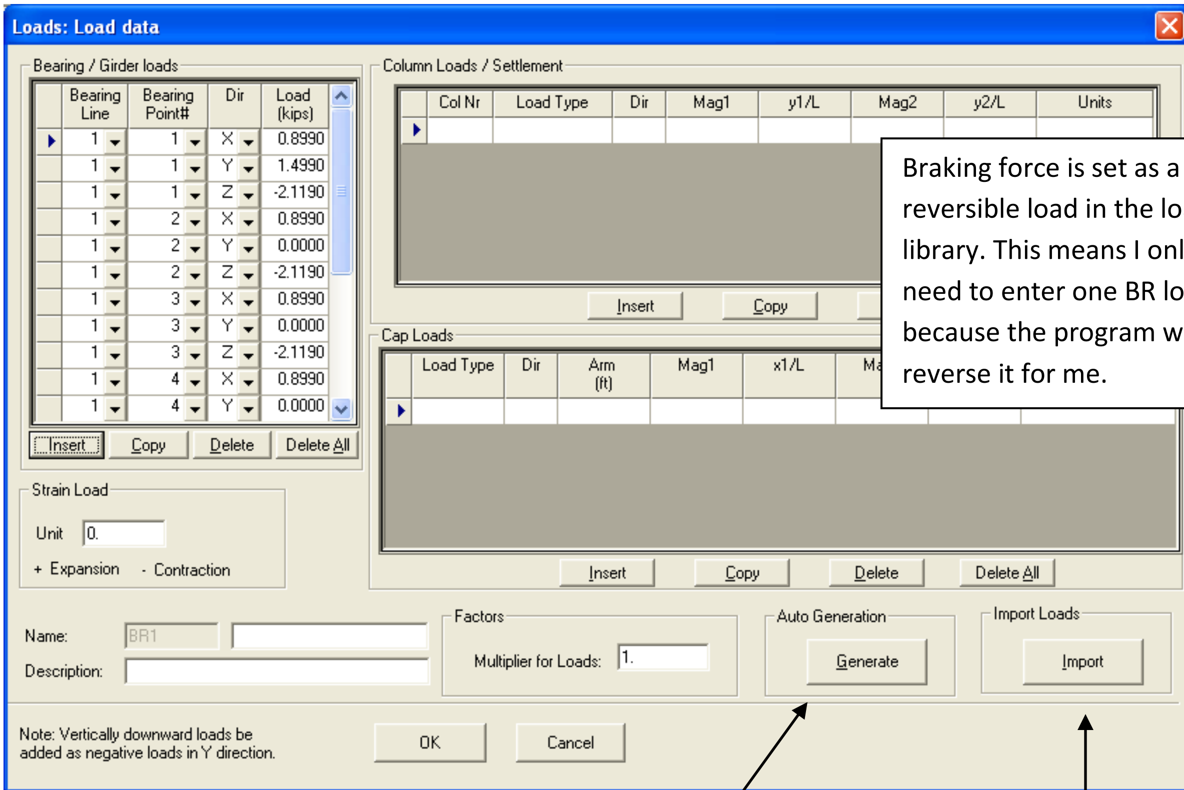
Live loads are imported from the text files generated by the in-house spreadsheet for bridge pier live loads.




```
Bearing Loads
1, 1, Y, -59.821
1, 2, Y, -53.686
1, 3, Y, 0
1, 4, Y, 0
1, 5, Y, 0
1, 6, Y, 0
1, 1, Y, -37.286
1, 2, Y, -33.832
1, 3, Y, -0.586
1, 4, Y, 0
1, 5, Y, 0
1, 6, Y, 0
```

This is the content of "LLPier1Loads001.txt" which is imported into RC-Pier for LL1.

BR1



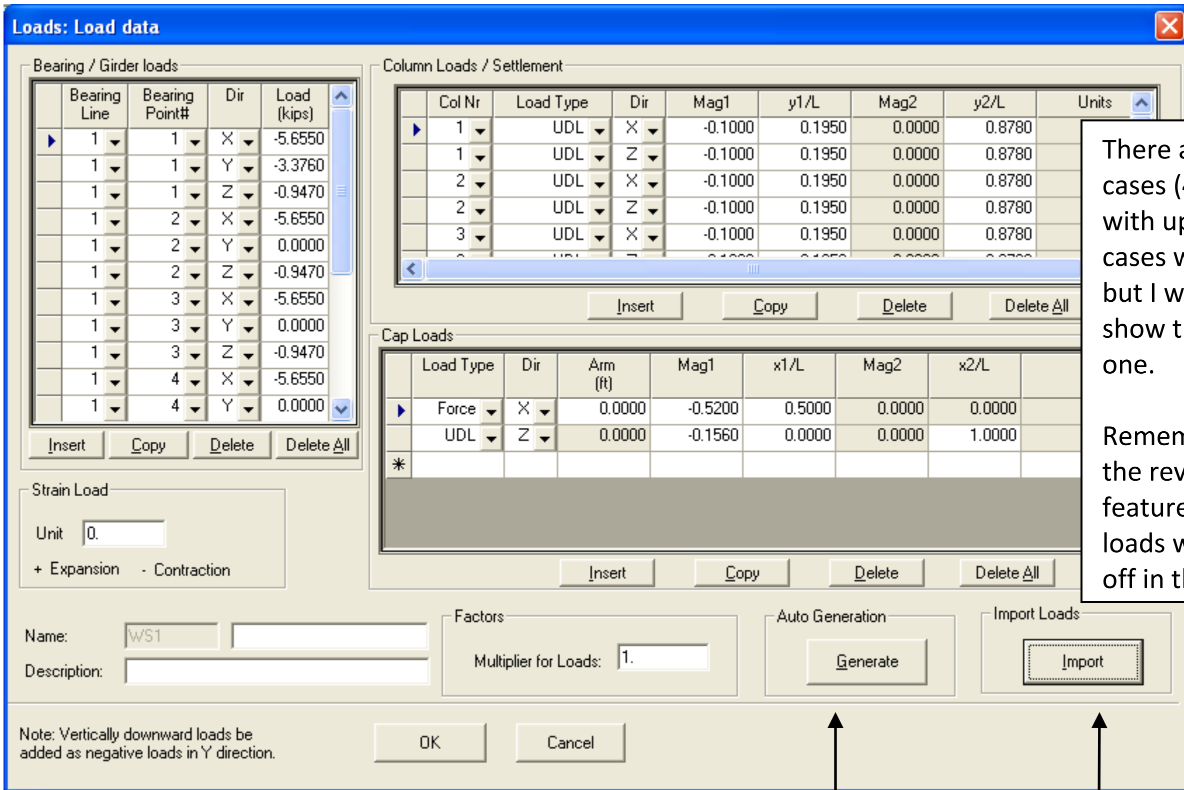
Braking force is set as a reversible load in the load library. This means I only need to enter one BR load because the program will reverse it for me.

Braking can be auto-generated. We typically distribute the concentrated truck portion of this force among the contributing bents; however, RC-Pier puts the entire load on the pier being analyzed.

These loads were imported from text file "BRLoads001.txt" generated by the spreadsheet for BR loads.

```
Bearing Loads
1, 1, X, 0.899
1, 1, Y, 1.499
1, 1, Z, -2.119
1, 2, X, 0.899
1, 2, Y, 0
1, 2, Z, -2.119
1, 3, X, 0.899
1, 3, Y, 0
1, 3, Z, -2.119
1, 4, X, 0.899
1, 4, Y, 0
1, 4, Z, -2.119
1, 5, X, 0.899
1, 5, Y, 0
1, 5, Z, -2.119
1, 6, X, 0.899
1, 6, Y, -1.499
1, 6, Z, -2.119
```

WS1



There are 8 WS cases (4 cases with uplift and 4 cases without), but I will only show the first one.

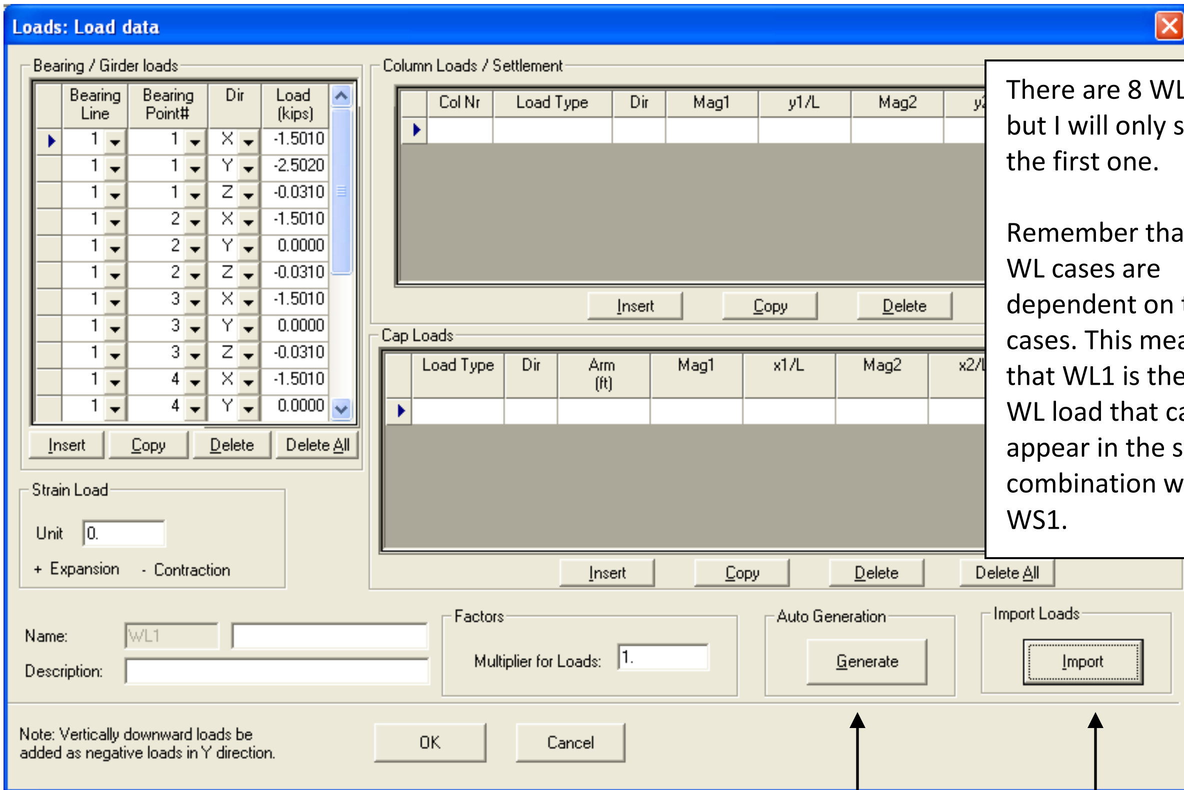
Remember that the reversible feature for wind loads was turned off in the library.

Wind loads can be auto-generated, but generating simplified wind loads requires a few tricks and some editing of the load results.

The wind loads above were imported from text file “WSLoadsNoUplift001.txt” which was generated from the in-house spreadsheet for wind loads.

```
Bearing Loads
1, 1, X, -5.655
1, 1, Y, -3.376
1, 1, Z, -0.947
1, 2, X, -5.655
1, 2, Y, 0
1, 2, Z, -0.947
1, 3, X, -5.655
1, 3, Y, 0
1, 3, Z, -0.947
1, 4, X, -5.655
1, 4, Y, 0
1, 4, Z, -0.947
1, 5, X, -5.655
1, 5, Y, 0
1, 5, Z, -0.947
1, 6, X, -5.655
1, 6, Y, 3.376
1, 6, Z, -0.947
Cap Loads
Force, X, 0, -0.52, 0.5
UDL, Z, -0.156, 0, 1
Column Loads
1, UDL, X, -0.1, 0.195, 0.878
1, UDL, Z, -0.1, 0.195, 0.878
2, UDL, X, -0.1, 0.195, 0.878
2, UDL, Z, -0.1, 0.195, 0.878
3, UDL, X, -0.1, 0.195, 0.878
3, UDL, Z, -0.1, 0.195, 0.878
```

WL1



There are 8 WL cases, but I will only show the first one.

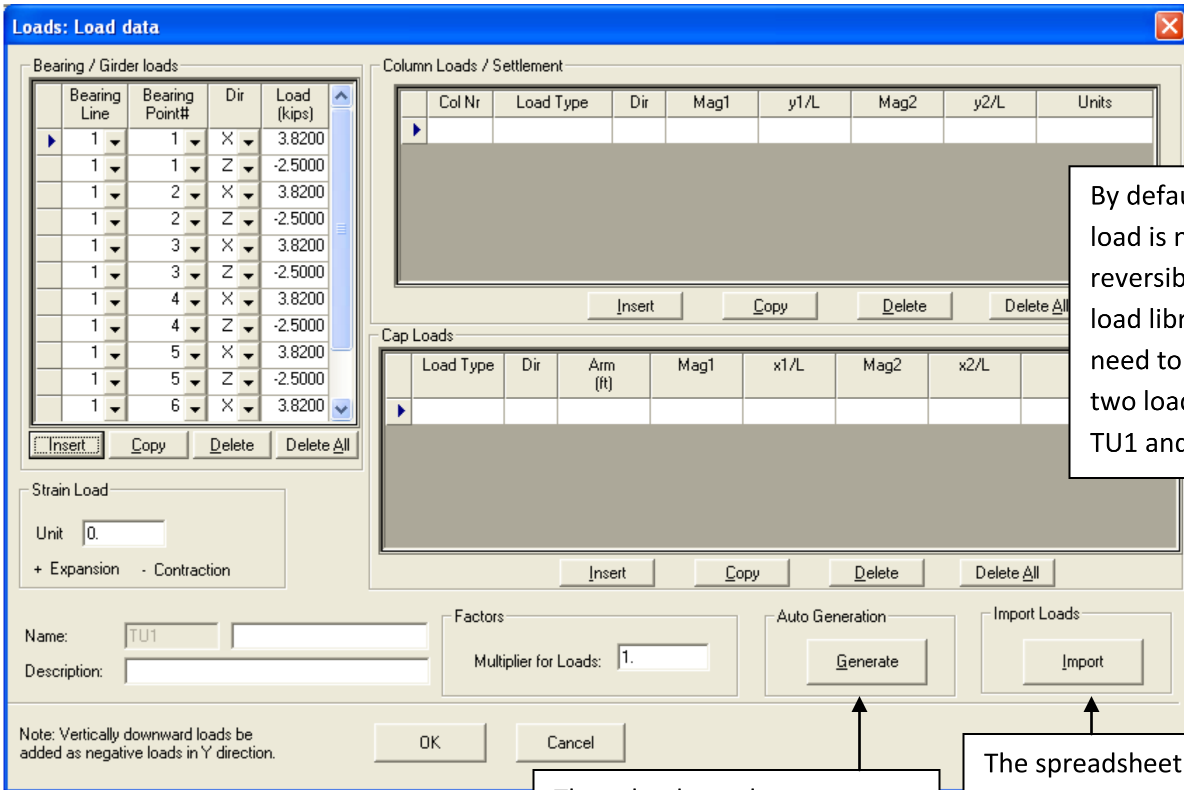
Remember that the WL cases are dependent on the WS cases. This means that WL1 is the only WL load that can appear in the same combination with WS1.

Wind loads can be auto-generated, but generating simplified wind loads requires some editing of the load results.

```
Bearing Loads
1, 1, X, -1.501
1, 1, Y, -2.502
1, 1, Z, -0.031
1, 2, X, -1.501
1, 2, Y, 0
1, 2, Z, -0.031
1, 3, X, -1.501
1, 3, Y, 0
1, 3, Z, -0.031
1, 4, X, -1.501
1, 4, Y, 0
1, 4, Z, -0.031
1, 5, X, -1.501
1, 5, Y, 0
1, 5, Z, -0.031
1, 6, X, -1.501
1, 6, Y, 2.502
1, 6, Z, -0.031
```

The wind loads above were imported from text file “WLLoads001.txt” which was generated from the in-house spreadsheet for wind loads.

TU1

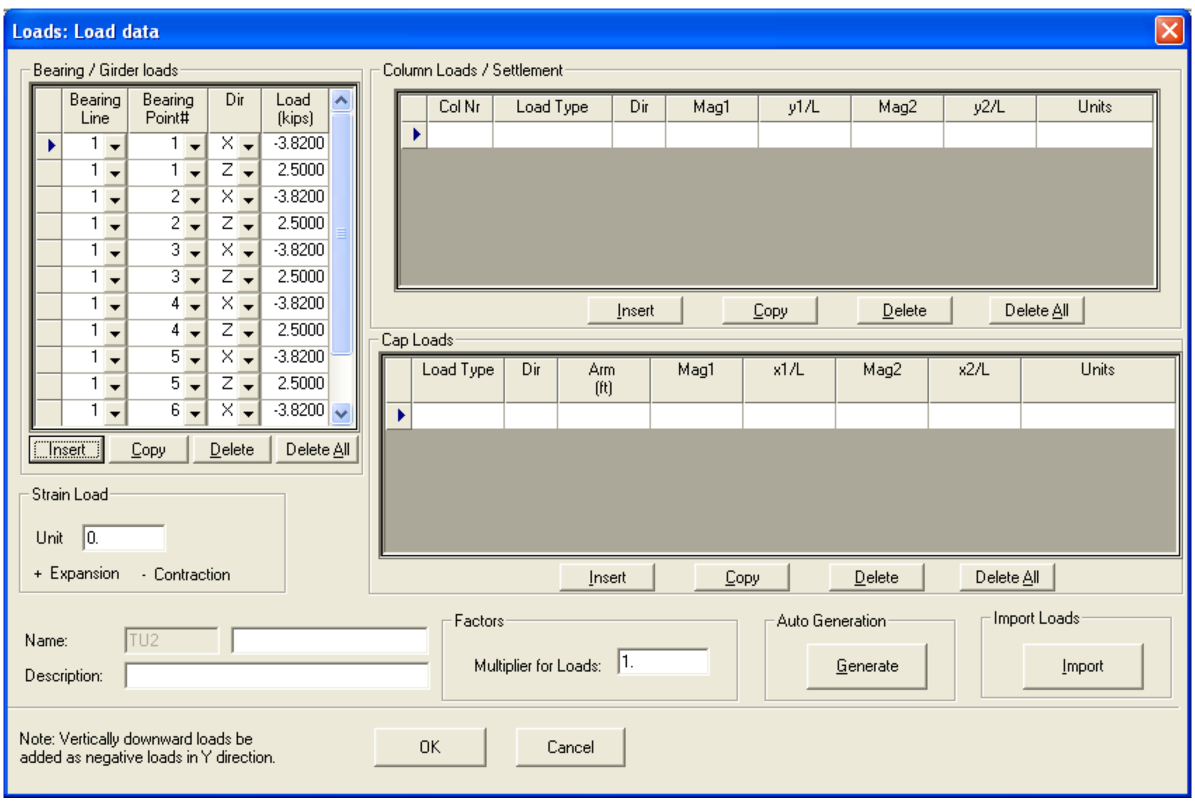


By default this load is not reversible in the load library so I need to enter two load cases: TU1 and TU2.

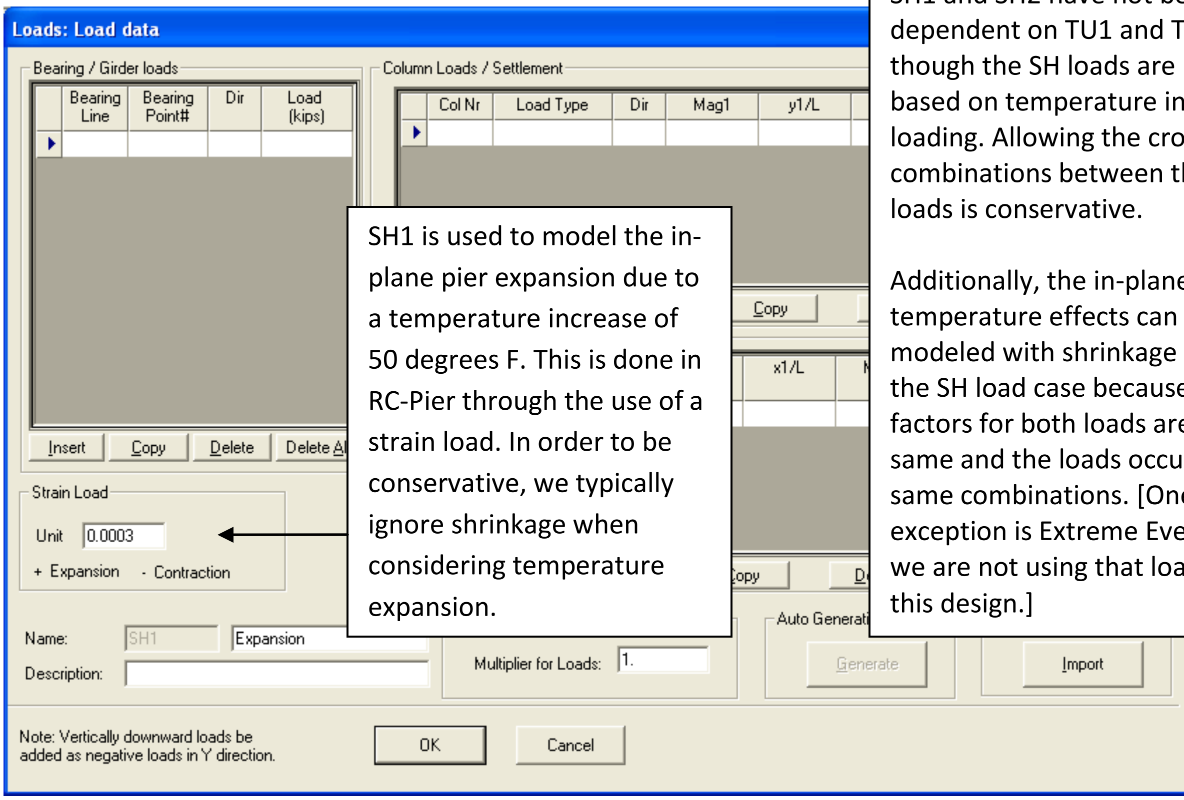
These loads can be auto-generated, but the procedure is not according to DOT policy.

The spreadsheet that was setup to calculate TU loads will not generate standardized text files in order to import the loads.

TU2



SH1

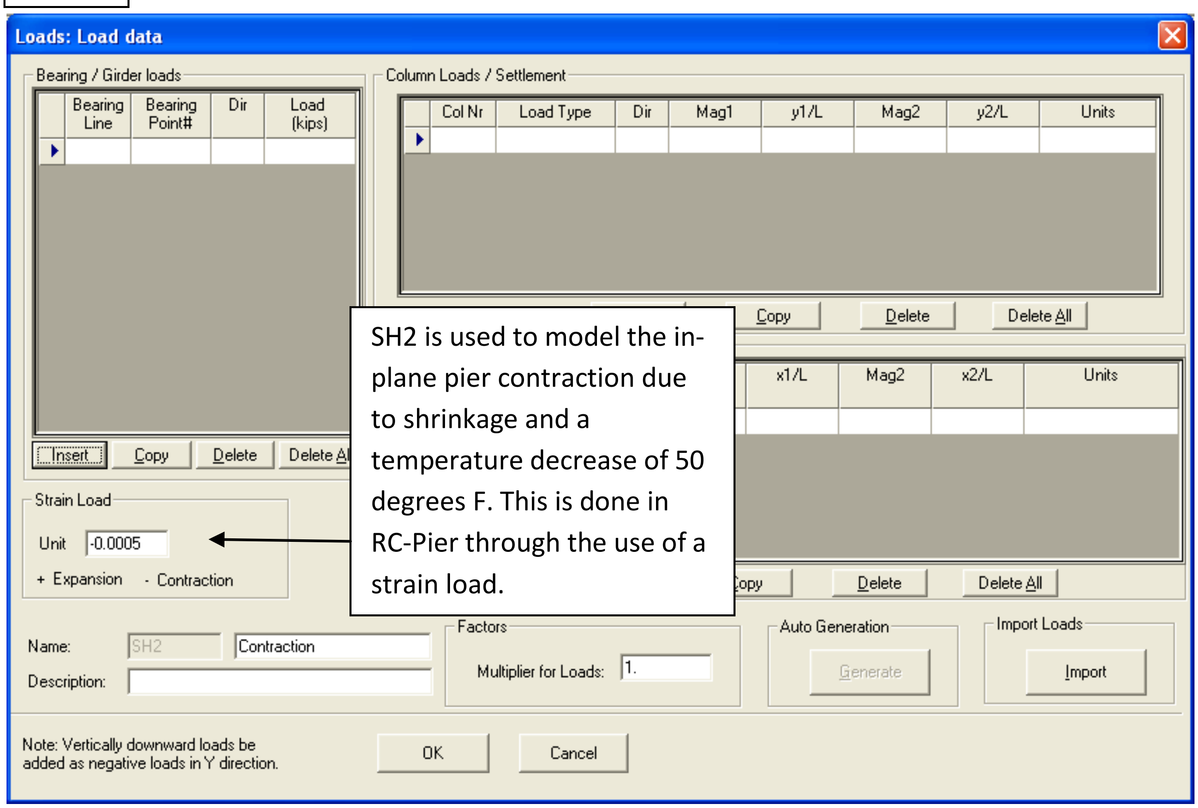


SH1 is used to model the in-plane pier expansion due to a temperature increase of 50 degrees F. This is done in RC-Pier through the use of a strain load. In order to be conservative, we typically ignore shrinkage when considering temperature expansion.

SH1 and SH2 have not been made dependent on TU1 and TU2 even though the SH loads are partially based on temperature induced loading. Allowing the cross-combinations between these loads is conservative.

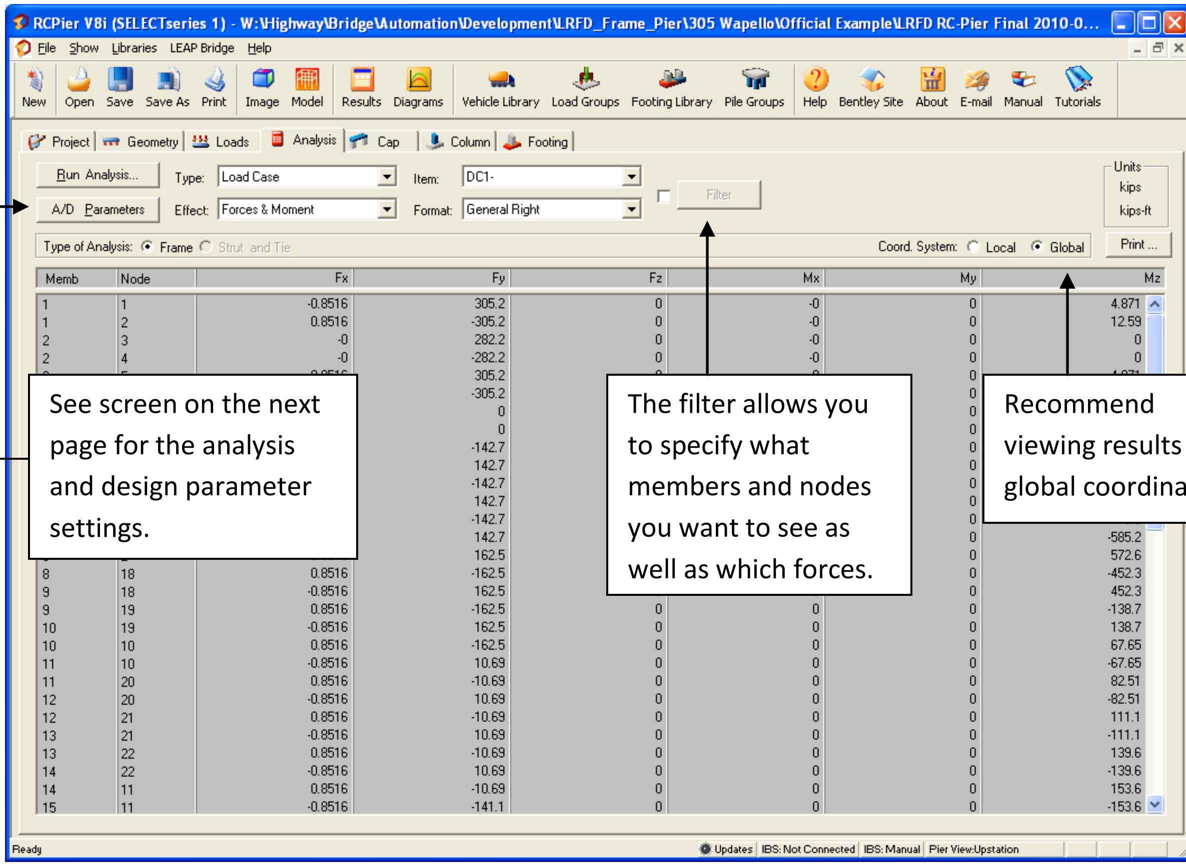
Additionally, the in-plane temperature effects can be modeled with shrinkage effects in the SH load case because the load factors for both loads are the same and the loads occur in the same combinations. [One exception is Extreme Event 2, but we are not using that load case in this design.]

SH2



SH2 is used to model the in-plane pier contraction due to shrinkage and a temperature decrease of 50 degrees F. This is done in RC-Pier through the use of a strain load.

Run the Analysis



See screen on the next page for the analysis and design parameter settings.

The filter allows you to specify what members and nodes you want to see as well as which forces.

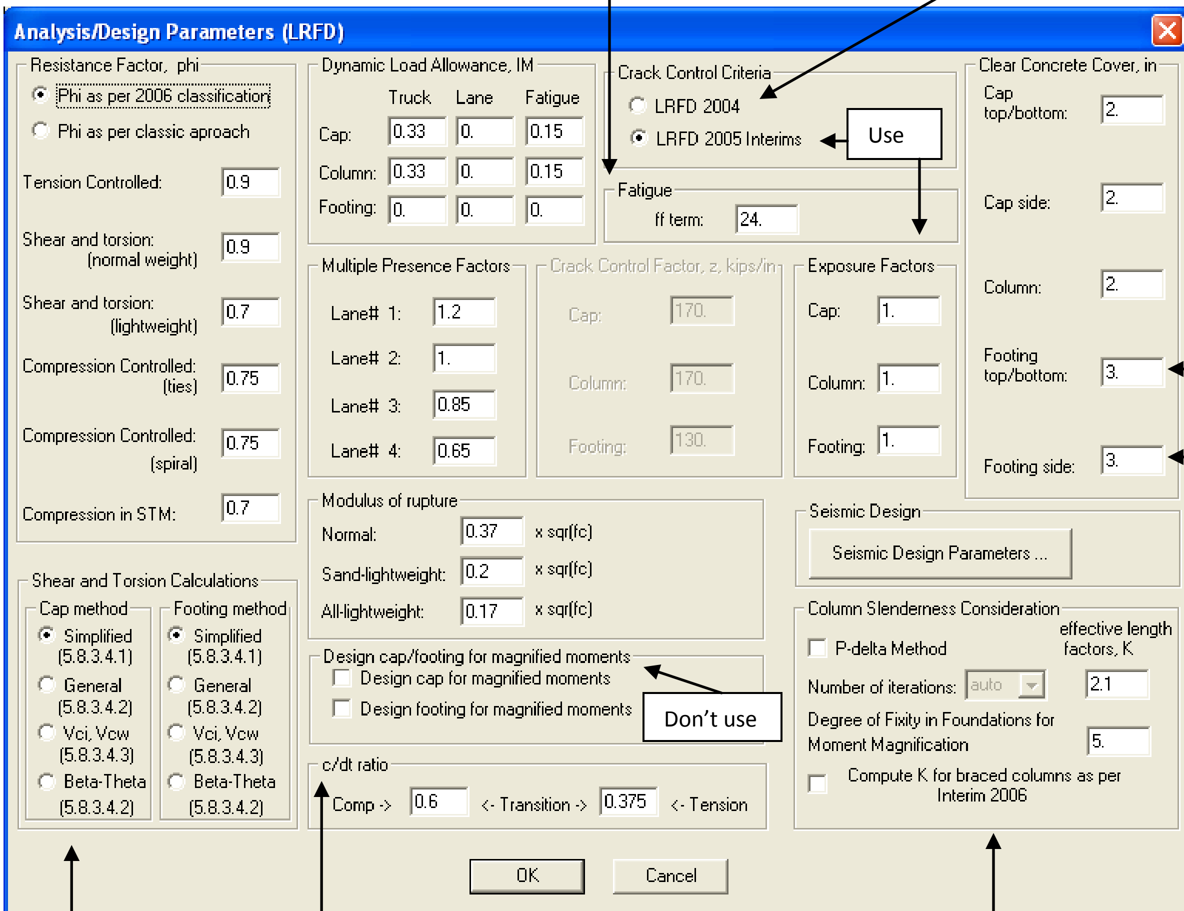
Recommend viewing results in global coordinates.

This screen is useful for checking reasonableness of results and for resolving differences among designers and checkers since you can look at load cases, load combinations, and envelopes.

Analysis and Design Parameters

Typically fatigue is not considered.

z-check



Use

Don't use

Not of interest for cap and column design.

We will use "Phi as per 2006 classification". This means we look at sections as being Tension Controlled, In-Transition, or Compression Controlled. See Aashto Lrfd Fig. C5.5.4.2.1-1 for information on how the "c/dt ratio" used in the calculation of the flexural resistance factors in transition.

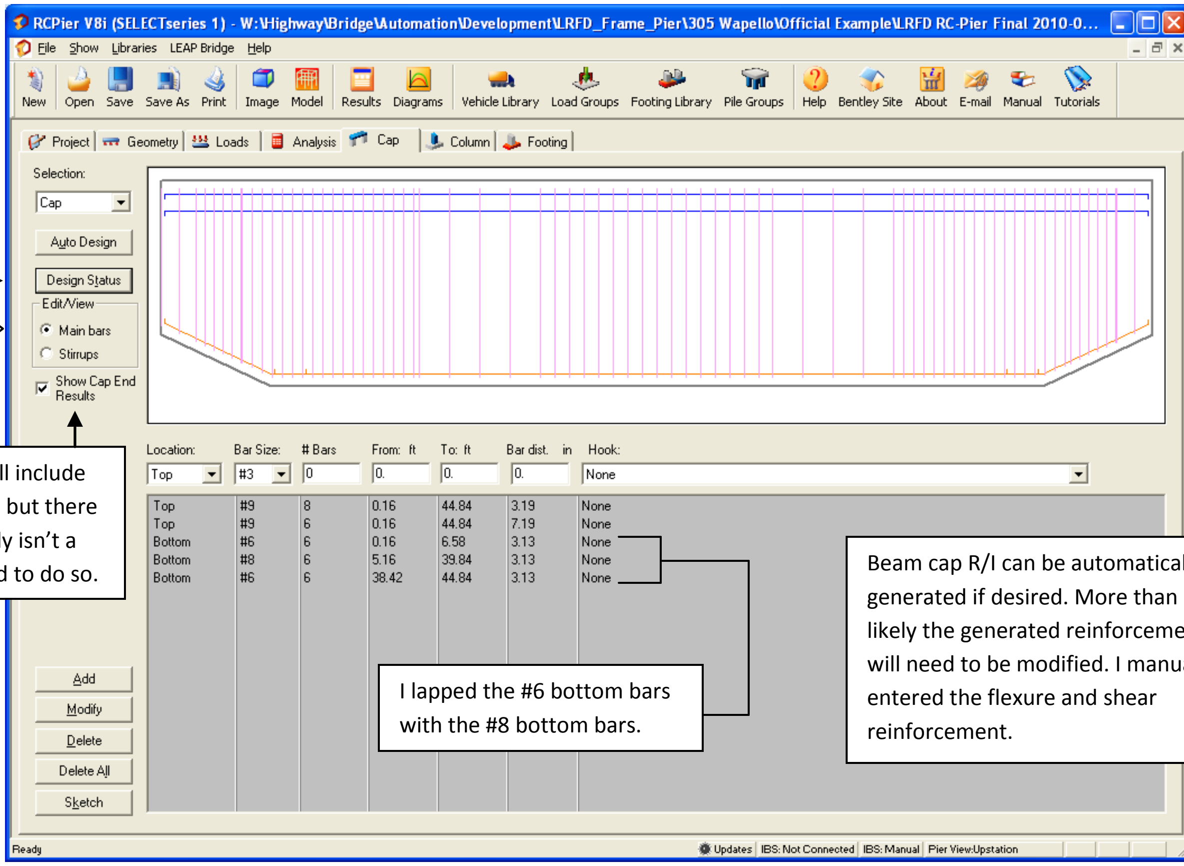
We typically use moment magnification for column design. The parameters we use can be entered on the column design screen.

The Bridge Manual does not specify which shear method to use for cap design. A general recommendation is to base shear design for the cap on the "Simplified" method or the "Beta-Theta" method. The "Beta-Theta" method is actually the procedure listed in Aashto Lrfd 5.8.3.4.2 (General Procedure with closed form solution) rather than that found in Aahto Lrfd Appendix B5 (General Procedure with tables). The "General" method listed on the screen above is the one found in Aashto Lrfd Appendix B5.

RC-Pier does not check column shear. Designers may want to verify that column shear is OK.

Pier Cap Design: RC-Pier

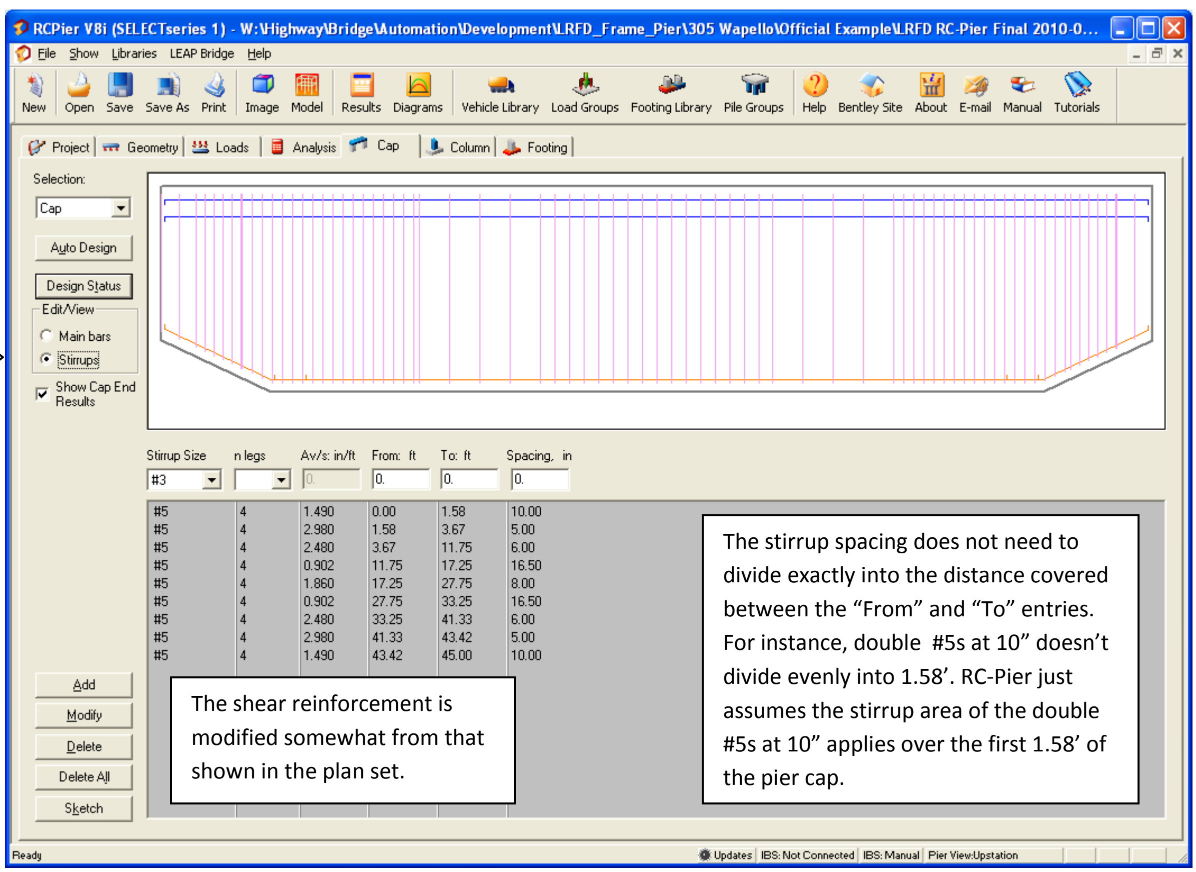
Design status report is printed out a few pages over.



We'll include this, but there really isn't a need to do so.

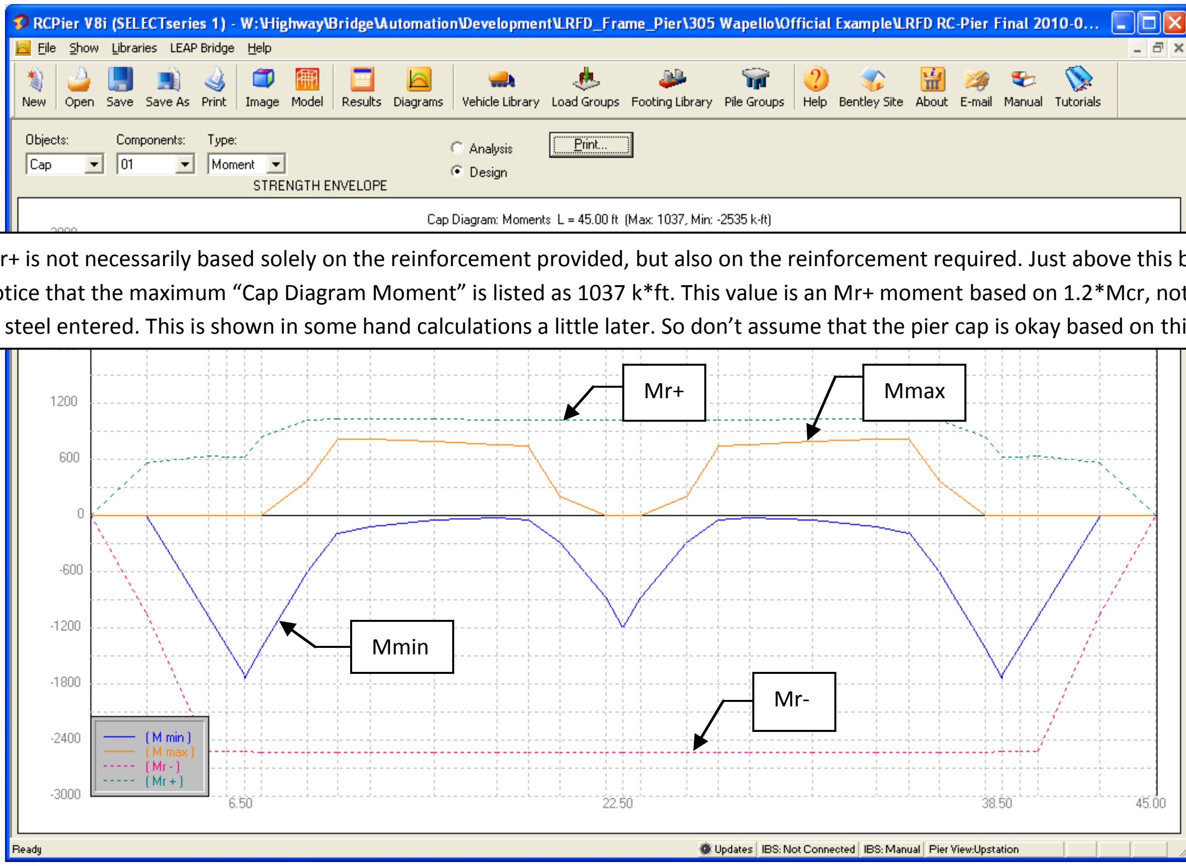
I lapped the #6 bottom bars with the #8 bottom bars.

Beam cap R/I can be automatically generated if desired. More than likely the generated reinforcement will need to be modified. I manually entered the flexure and shear reinforcement.

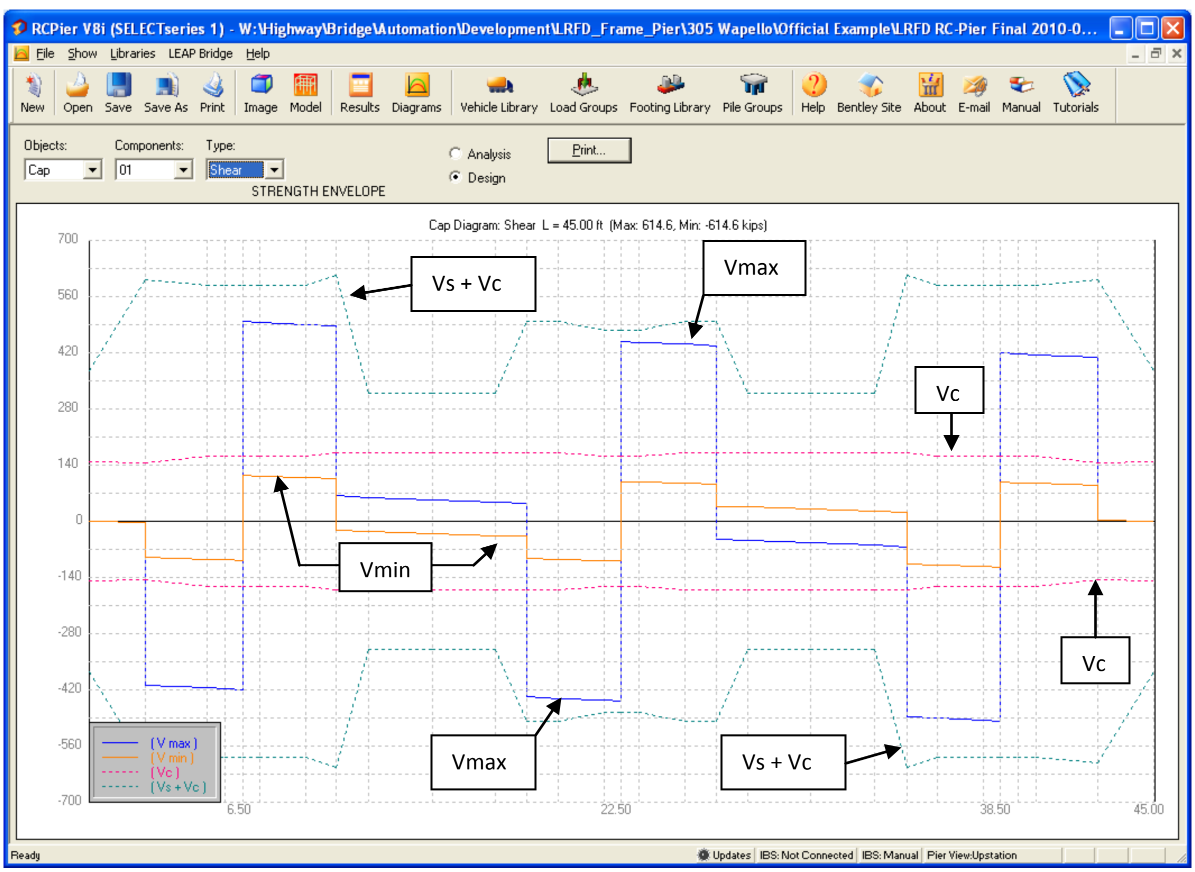


The shear reinforcement is modified somewhat from that shown in the plan set.

The stirrup spacing does not need to divide exactly into the distance covered between the "From" and "To" entries. For instance, double #5s at 10" doesn't divide evenly into 1.58'. RC-Pier just assumes the stirrup area of the double #5s at 10" applies over the first 1.58' of the pier cap.



Mr+ is not necessarily based solely on the reinforcement provided, but also on the reinforcement required. Just above this box you'll notice that the maximum "Cap Diagram Moment" is listed as 1037 k*ft. This value is an Mr+ moment based on 1.2*Mcr, not the area of steel entered. This is shown in some hand calculations a little later. So don't assume that the pier cap is okay based on this graph.



CAP DESIGN

I've included some portions of RC-Pier's output for the cap design.

CAP DESIGN
Code: AASHTO LRFD 2007 (with Interims)
Units: US
Pier View: Upstation.

DESIGN PARAMETERS	
f'c = 3500.0 psi	
Fy flex = 60000.0 psi	Fy shear = 60000.0 psi
phi tens = 0.90	
phi comp = 0.75	phi shear = 0.90
Tens below = 0.375	Comp Above = 0.600
Ec = 3586.6 ksi	Es = 29000.0 ksi
Crack check as per 2005 Interims	
Crack control Exposure = 1.00	
Concrete Type : Normal Weight.	
Design of cap at centerline of column.	

CAP GEOMETRY
Tapered Cap : Length(X) = 45.00 ft Depth(Z) = 39.00 in

Cap Section Properties

Sec.	Area ft ^2	Ixx in ^4	Izz in ^4
1	13.00	359424.00	237276.00

MAIN REINFORCEMENT

	Bar size	Quantity	Bar dist. in	As total in^2	From ft	To ft	Hook
TOP							
	# 9	8	3.19	8.000	0.16	44.84	None
	# 9	6	7.19	6.000	0.16	44.84	None
BOTTOM							
	# 6	6	3.13	2.640	0.16	6.58	None
	# 8	6	3.13	4.740	5.16	39.84	None
	# 6	6	3.13	2.640	38.42	44.84	None

STIRRUPS

From ft	To ft	Stirrup Size	n legs	Spacing in	Apr/s in^2 / ft
0.00	1.58	# 5	4	10.00	1.49
1.58	3.67	# 5	4	5.00	2.98
3.67	11.75	# 5	4	6.00	2.48
11.75	17.25	# 5	4	16.50	0.90
17.25	27.75	# 5	4	8.00	1.86
27.75	33.25	# 5	4	16.50	0.90
33.25	41.33	# 5	4	6.00	2.48
41.33	43.42	# 5	4	5.00	2.98
43.42	45.00	# 5	4	10.00	1.49

FLEXURE DESIGN

Bottom R/I | Top R/I

Span 1: From 0.00 ft To 6.50 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.0	0.0	36	0.0	0.0	0	T	0.66	0.00*	0.00*	0.66	0.00*	0.00*
			0.0	-0.0	0	T	0.66	0.00*	0.00*	0.66	0.00*	0.00*
2.4	2.4	42	10.6	573.1	481	T	0.71	2.64	2.64	0.71	14.00	14.00
			-20.4	-1060.3	427	T	0.71	2.64	2.64	0.71	14.00	6.70
5.0	5.0	48	0.0	647.2	0	T	0.76	2.64	2.64	0.76	14.00	14.00
			-1086.5	-2520.5	14	T	0.76	2.64	2.64	5.80	14.00	14.00
5.8	5.8	48	0.0	624.9	0	T	0.76	7.38	2.52	0.76	14.00	14.00
			-1403.8	-2524.0	14	T	0.76	7.38	3.40	7.58	14.00	14.00
6.5	6.5	48	0.0	629.8	0	T	0.76	7.38	2.55	0.76	14.00	14.00
			-1714.2	-2531.0	14	T	0.76	7.38	3.94	9.36	14.00	14.00

Span 2: From 6.50 ft To 22.50 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.0	6.5	48	0.0	629.8	0	T	0.76	7.38	2.55	0.76	14.00	14.00
			-1745.8	-2531.0	14	T	0.76	7.38	3.94	9.55	14.00	14.00
0.7	7.2	48	0.0	851.5	0	T	0.76	4.74	3.74	0.76	14.00	14.00
			-1423.9	-2534.8	14	T	0.76	4.74	4.74	7.70	14.00	14.00
2.7	9.2	48	373.8	1029.3	3719	T	2.50	4.74	4.74	0.76	14.00	14.00
			-609.0	-2534.8	727	T	0.76	4.74	4.74	4.29	14.00	14.00
3.9	10.4	48	822.6	1036.6	81	T	5.30	4.74*	4.74*	0.76	14.00	14.00
			-196.9	-2534.8	527	T	0.76	4.74	4.74	1.36	14.00	14.00
5.3	11.8	48	817.4	1036.6	81	T	5.30	4.74*	4.74*	0.76	14.00	14.00
			-129.3	-2534.8	527	T	0.76	4.74	4.74	0.89	14.00	14.00
8.0	14.5	48	794.2	1036.6	81	T	5.30	4.74*	4.74*	0.76	14.00	14.00
			-54.0	-2534.8	3226	T	0.76	4.74	4.74	0.76	14.00	14.00
10.7	17.2	48	756.1	1029.3	94	T	5.13	4.74*	4.74*	0.76	14.00	14.00
			-35.9	-2534.8	316	T	0.76	4.74	4.74	0.76	14.00	14.00
12.0	18.5	48	748.7	1029.3	42	T	5.08	4.74*	4.74*	0.76	14.00	14.00
			-55.2	-2534.8	368	T	0.76	4.74	4.74	0.76	14.00	14.00
13.3	19.8	48	214.0	1029.3	328	T	1.42	4.74	4.74	0.76	14.00	14.00
			-298.8	-2534.8	82	T	0.76	4.74	4.74	2.07	14.00	14.00
15.3	21.8	48	0.0	1029.3	0	T	0.76	4.74	4.74	0.76	14.00	14.00
			-886.8	-2534.8	91	T	0.76	4.74	4.74	5.53	14.00	14.00
16.0	22.5	48	0.0	1029.3	0	T	0.76	4.74	4.74	0.76	14.00	14.00
			-1204.2	-2534.8	91	T	0.76	4.74	4.74	6.46	14.00	14.00

Iowa has special loading requirements for the pier cap overhang design which were not included in this RC-Pier run. Further on in this example is a spreadsheet design addresses these requirements.

The cap end results could have been excluded if we had left the “Show Cap End Results” check box unchecked.

Ideally the C.L. of the column would be excluded when the user specifies offsets from the C.L. of columns as we did.

This is where the two sets of bars lap each other:

$2.64 + 4.74 = 7.38$ in^2

I will show some hand calculations for the bottom reinforcement at this location (3.9’) a little later in the example.

Span 3: From 22.50 ft To 38.50 ft													
Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.0	22.5	48	0.0	1029.3	0	T	0.76	4.74	4.74	0.76	14.00	14.00
			-1204.2	-2534.8	52	T	0.76	4.74	4.74	6.46	14.00	14.00
0.7	23.2	48	0.0	1029.3	0	T	0.76	4.74	4.74	0.76	14.00	14.00
			-886.8	-2534.8	52	T	0.76	4.74	4.74	5.53	14.00	14.00
2.7	25.2	48	214.0	1029.3	368	T	1.42	4.74	4.74	0.76	14.00	14.00
			-298.8	-2534.8	42	T	0.76	4.74	4.74	2.07	14.00	14.00
4.0	26.5	48	748.7	1029.3	82	T	5.08	4.74*	4.74*	0.76	14.00	14.00
			-55.2	-2534.8	328	T	0.76	4.74	4.74	0.76	14.00	14.00
5.3	27.8	48	756.1	1029.3	30	T	5.13	4.74*	4.74*	0.76	14.00	14.00
			-35.9	-2534.8	380	T	0.76	4.74	4.74	0.76	14.00	14.00
8.0	30.5	48	794.2	1036.6	43	T	5.30	4.74*	4.74*	0.76	14.00	14.00
			-54.0	-2534.8	3511	T	0.76	4.74	4.74	0.76	14.00	14.00
10.7	33.2	48	817.4	1036.6	43	T	5.30	4.74*	4.74*	0.76	14.00	14.00
			-129.3	-2534.8	517	T	0.76	4.74	4.74	0.89	14.00	14.00
12.1	34.6	48	822.6	1036.6	43	T	5.30	4.74*	4.74*	0.76	14.00	14.00
			-196.9	-2534.8	517	T	0.76	4.74	4.74	1.36	14.00	14.00
13.3	35.8	48	373.8	1029.3	3434	T	2.50	4.74	4.74	0.76	14.00	14.00
			-609.0	-2534.8	1014	T	0.76	4.74	4.74	4.29	14.00	14.00
15.3	37.8	48	0.0	851.5	0	T	0.76	4.74	3.74	0.76	14.00	14.00
			-1423.9	-2534.8	54	T	0.76	4.74	4.74	7.70	14.00	14.00
16.0	38.5	48	0.0	630.9	0	T	0.76	7.38	2.55	0.76	14.00	14.00
			-1745.8	-2531.1	54	T	0.76	7.38	3.95	9.55	14.00	14.00

Span 4: From 38.50 ft To 45.00 ft													
Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.0	38.5	48	0.0	630.9	0	T	0.76	7.38	2.55	0.76	14.00	14.00
			-1714.2	-2531.1	54	T	0.76	7.38	3.95	9.36	14.00	14.00
0.7	39.2	48	0.0	625.9	0	T	0.76	7.38	2.53	0.76	14.00	14.00
			-1403.8	-2524.0	54	T	0.76	7.38	3.41	7.58	14.00	14.00
1.5	40.0	48	0.0	647.2	0	T	0.76	2.64	2.64	0.76	14.00	14.00
			-1086.5	-2520.5	54	T	0.76	2.64	2.64	5.80	14.00	14.00
4.1	42.6	42	10.6	573.1	491	T	0.71	2.64	2.64	0.71	14.00	14.00
			-20.4	-1060.3	417	T	0.71	2.64	2.64	0.71	14.00	6.70
6.5	45.0	36	0.0	0.0	0	T	0.66	0.00*	0.00*	0.66	0.00*	0.00*
			0.0	-0.0	0	T	0.66	0.00*	0.00*	0.66	0.00*	0.00*

Flexure Design : Notes
CL: Section classification as per LRFD 2006 interims for provided reinforcement.
C = Compression controlled, I = In-Transition, T = Tension controlled.
* The provided reinforcement is not adequate, either less than required or larger than maximum allowed.

Required Shear R/I

Provided Shear R/I

Additional longitudinal reinforcement required based on Aashto Lrfd 5.8.3.5. As of the printing of this example the Iowa DOT Bridge Design Manual requires that this provision be met all along the cap for pier cap design.

SHEAR AND TORSION DESIGN

Span 1: From 0.00 ft To 6.50 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2
0.00	0.00	R	0.0	0	0.0	0	149.5	57.6	0.00	0.00	0.00	1.49	0.00
2.40	2.40	L	4.7	1	0.0	0	145.9	72.0	0.00	0.00	0.00	2.98	0.00
		R	410.7	14	9.8	662	145.9	72.0	1.68	0.00	1.68	2.98	0.00
5.00	5.00	L	416.6	14	9.8	1000	164.0	88.3	1.42	0.00	1.42	2.48	0.00
		R	416.6	14	9.8	662	164.0	88.3	1.42	0.00	1.42	2.48	0.00
5.76	5.76	L	418.5	14	9.8	1000	164.0	88.3	1.43	0.00	1.43	2.48	0.00
		R	418.5	14	9.8	662	164.0	88.3	1.43	0.00	1.43	2.48	0.00
6.50	6.50	L	420.3	14	9.8	1000	164.0	88.3	1.44	0.00	1.44	2.48	0.00

Span 2: From 6.50 ft To 22.50 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2
0.00	6.50	R	498.7	19	9.3	1005	164.0	88.3	1.88	0.00	1.88	2.48	0.00
0.74	7.24	L	496.9	19	9.3	667	164.0	88.3	1.87	0.00	1.87	2.48	0.00
		R	496.9	19	9.3	1005	164.0	88.3	1.87	0.00	1.87	2.48	0.00
2.67	9.17	L	492.2	19	9.3	667	164.0	88.3	1.85	0.00	1.85	2.48	0.00
		R	492.2	19	9.3	1005	164.0	88.3	1.85	0.00	1.85	2.48	0.00
3.94	10.44	L	489.1	19	9.3	667	171.3	88.3	1.71	0.00	1.71	2.48	3.50
		R	64.7	731	0.5	666	171.3	88.3	0.00	0.00	0.00	2.48	0.00
5.33	11.83	L	61.4	731	0.5	1004	171.3	88.3	0.00	0.00	0.00	0.90	0.00
		R	61.4	731	0.5	666	171.3	88.3	0.00	0.00	0.00	0.90	0.00
8.00	14.50	L	54.8	731	0.5	1004	171.3	88.3	0.00	0.00	0.00	0.90	0.00
		R	54.8	731	0.5	666	171.3	88.3	0.00	0.00	0.00	0.90	0.00
10.67	17.17	L	48.3	731	0.5	1004	171.3	88.3	0.00	0.00	0.00	0.90	0.00
		R	48.3	731	0.5	666	171.3	88.3	0.00	0.00	0.00	0.90	0.00
11.98	18.48	L	45.1	731	0.5	1004	171.3	88.3	0.00	0.00	0.00	1.86	0.00
		R	439.1	84	10.3	875	171.3	88.3	1.44	0.00	1.44	1.86	3.98
13.33	19.83	L	442.4	84	10.3	1213	171.3	88.3	1.46	0.00	1.46	1.86	1.50
		R	442.4	84	10.3	875	171.3	88.3	1.46	0.00	1.46	1.86	1.50
15.26	21.76	L	447.1	84	10.3	1213	164.0	88.3	1.59	0.00	1.59	1.86	0.00
		R	447.1	84	10.3	875	164.0	88.3	1.59	0.00	1.59	1.86	0.00
16.00	22.50	L	448.9	84	10.3	1213	164.0	88.3	1.60	0.00	1.60	1.86	0.00

Torsion is usually not a factor for a typical frame pier design. There are essentially two reasons for this. The first reason is that we often use a single line of bearings centered on the pier cap. Modeling the pier this way reduces Mx cap moments. The second reason is that we generally assume lateral superstructure loading does not generate Mx cap moments between the superstructure and the top of the cap. The user needs to determine if these modeling assumptions are appropriate for their design.

Span 3: From 22.50 ft To 38.50 ft													
Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2
0.00	22.50	R	448.9	47	10.3	1215	164.0	88.3	1.60	0.00	1.60	1.86	0.00
0.74	23.24	L	447.1	47	10.3	877	164.0	88.3	1.59	0.00	1.59	1.86	0.00
		R	447.1	47	10.3	1215	164.0	88.3	1.59	0.00	1.59	1.86	0.00
2.67	25.17	L	442.4	47	10.3	877	171.3	88.3	1.46	0.00	1.46	1.86	1.52
		R	442.4	47	10.3	1215	171.3	88.3	1.46	0.00	1.46	1.86	1.50
4.02	26.52	L	439.1	47	10.3	877	171.3	88.3	1.44	0.00	1.44	1.86	3.98
		R	45.1	1019	0.5	1006	171.3	88.3	0.00	0.00	0.00	1.86	0.00
5.33	27.83	L	48.3	1019	0.5	720	171.3	88.3	0.00	0.00	0.00	0.90	0.00
		R	48.3	1019	0.5	1006	171.3	88.3	0.00	0.00	0.00	0.90	0.00
8.00	30.50	L	54.8	1019	0.5	720	171.3	88.3	0.00	0.00	0.00	0.90	0.00
		R	54.8	1019	0.5	1006	171.3	88.3	0.00	0.00	0.00	0.90	0.00
10.67	33.17	L	61.4	1019	0.5	720	171.3	88.3	0.00	0.00	0.00	0.90	0.00
		R	61.4	1019	0.5	1006	171.3	88.3	0.00	0.00	0.00	0.90	0.00
12.06	34.56	L	64.7	1019	0.5	720	171.3	88.3	0.00	0.00	0.00	2.48	0.00
		R	489.1	60	9.3	669	171.3	88.3	1.71	0.00	1.71	2.48	3.38
13.33	35.83	L	492.2	60	9.3	1007	164.0	88.3	1.85	0.00	1.85	2.48	0.00
		R	492.2	60	9.3	669	164.0	88.3	1.85	0.00	1.85	2.48	0.00
15.26	37.76	L	496.9	60	9.3	1007	164.0	88.3	1.87	0.00	1.87	2.48	0.00
		R	496.9	60	9.3	669	164.0	88.3	1.87	0.00	1.87	2.48	0.00
16.00	38.50	L	498.7	60	9.3	1007	164.0	88.3	1.88	0.00	1.88	2.48	0.00

Span 4: From 38.50 ft To 45.00 ft													
Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2
0.00	38.50	R	420.3	54	9.8	1001	164.0	88.3	1.44	0.00	1.44	2.48	0.00
0.74	39.24	L	418.5	54	9.8	715	164.0	88.3	1.43	0.00	1.43	2.48	0.00
		R	418.5	54	9.8	1001	164.0	88.3	1.43	0.00	1.43	2.48	0.00
1.50	40.00	L	416.6	54	9.8	715	164.0	88.3	1.42	0.00	1.42	2.48	0.00
		R	416.6	54	9.8	1001	164.0	88.3	1.42	0.00	1.42	2.48	0.00
4.10	42.60	L	410.7	54	9.8	715	145.9	72.0	1.68	0.00	1.68	2.98	0.00
		R	4.7	1	0.0	0	145.9	72.0	0.00	0.00	0.00	2.98	0.00
6.50	45.00	L	0.0	0	0.0	0	149.5	57.6	0.00	0.00	0.00	1.49	0.00

Shear and Torsion Design : Notes
- Pos is the design position. L suggests the calculation is done at immediate left of "Loc" and R suggests at immediate right of it.
- T-lim is the limiting value of torsion for the concrete section. If actual torsion is higher than this value, torsional steel has to be provided.
- Avs/s is the required area of steel per unit length for shear force.
- 2Ats/s is the required area of steel per unit length for two legs of torsional reinforcement.
- Av/s is the total required area of steel per unit length due to shear plus torsion.
- Aprvs/s is the total provided area of steel per unit length due to shear (stirrups).
- Alx is the EFFECTIVE longitudinal steel required in addition to the PROVIDED EFFECTIVE flexural steel.

CRACKING/FATIGUE CHECK

Span 1: From 0.00 ft To 6.50 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b	Fatigue Comb
2.40	2.4	41.8	4003	0.9	3.2	39.0	4.7	0.0	0.00	0
			4289	0.9	3.1	39.0	6.6	0.0	0.00	0
5.00	5.0	48.0	4055	16.7	3.2	31.6	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0
5.76	5.8	48.0	4055	21.5	3.2	23.2	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	3.0	0.0	0.00	0
6.50	6.5	48.0	4055	26.2	3.2	17.9	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	3.0	0.0	0.00	0

Span 2: From 6.50 ft To 22.50 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b	Fatigue Comb
0.00	6.5	48.0	4055	30.1	3.2	14.7	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	3.0	0.0	0.00	0
0.74	7.2	48.0	4055	24.9	3.2	19.1	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0
2.67	9.2	48.0	4055	11.7	3.2	39.0	4.7	0.0	0.00	0
			4551	19.1	3.1	27.1	6.6	0.0	0.00	0
3.94	10.4	48.0	4055	3.3	3.2	39.0	4.7	0.0	0.00	0
			4551	39.6	3.1	9.8	6.6	0.0	0.00	0
5.33	11.8	48.0	4058	2.2	3.2	39.0	4.7	0.0	0.00	0
			4551	38.0	3.1	10.5	6.6	0.0	0.00	0
8.00	14.5	48.0	4058	0.4	3.2	39.0	4.7	0.0	0.00	0
			4551	34.3	3.1	12.3	6.6	0.0	0.00	0
10.67	17.2	48.0	0	0.0	3.2	39.0	4.7	0.0	0.00	0
			4655	30.0	3.1	15.0	6.6	0.0	0.00	0
11.98	18.5	48.0	0	0.0	3.2	39.0	4.7	0.0	0.00	0
			4213	30.2	3.1	14.9	6.6	0.0	0.00	0
13.33	19.8	48.0	4500	4.8	3.2	39.0	4.7	0.0	0.00	0
			4005	8.1	3.1	39.0	6.6	0.0	0.00	0
15.26	21.8	48.0	4509	14.9	3.2	36.3	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0
16.00	22.5	48.0	4509	19.9	3.2	25.5	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0

Span 3: From 22.50 ft To 38.50 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b	Fatigue Comb
0.00	22.5	48.0	4223	19.9	3.2	25.5	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0
0.74	23.2	48.0	4223	14.9	3.2	36.3	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0
2.67	25.2	48.0	4213	4.8	3.2	39.0	4.7	0.0	0.00	0
			4292	8.1	3.1	39.0	6.6	0.0	0.00	0
4.02	26.5	48.0	0	0.0	3.2	39.0	4.7	0.0	0.00	0
			4500	30.2	3.1	14.9	6.6	0.0	0.00	0
5.33	27.8	48.0	0	0.0	3.2	39.0	4.7	0.0	0.00	0
			4162	30.0	3.1	15.0	6.6	0.0	0.00	0
8.00	30.5	48.0	4343	0.4	3.2	39.0	4.7	0.0	0.00	0
			4266	34.3	3.1	12.3	6.6	0.0	0.00	0
10.67	33.2	48.0	4343	2.2	3.2	39.0	4.7	0.0	0.00	0
			4266	38.0	3.1	10.5	6.6	0.0	0.00	0
12.06	34.6	48.0	4342	3.3	3.2	39.0	4.7	0.0	0.00	0
			4266	39.6	3.1	9.8	6.6	0.0	0.00	0
13.33	35.8	48.0	4342	11.7	3.2	39.0	4.7	0.0	0.00	0
			4266	19.1	3.1	27.1	6.6	0.0	0.00	0
15.26	37.8	48.0	4342	24.9	3.2	19.1	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0
16.00	38.5	48.0	4342	30.1	3.2	14.7	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	3.0	0.0	0.00	0

Span 4: From 38.50 ft To 45.00 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b	Fatigue Comb
0.00	38.5	48.0	4342	26.2	3.2	17.9	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	3.0	0.0	0.00	0
0.74	39.2	48.0	4342	21.5	3.2	23.2	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	3.0	0.0	0.00	0
1.50	40.0	48.0	4342	16.7	3.2	31.6	4.7	0.0	0.00	0
			0	0.0	3.1	39.0	6.6	0.0	0.00	0
4.10	42.6	41.8	4289	0.9	3.2	39.0	4.7	0.0	0.00	0
			4003	0.9	3.1	39.0	6.6	0.0	0.00	0

Hand Calculations for Bottom Reinforcement in Span 2 at Location 3.9’

M_{max} = 822.6 k*ft
$\rho = A_s/(b*d) = (4.74\ in^2)/[(39”)*(44.875”)] = 0.0027084$
$\rho_b = [(0.85*B_1*f'_c)/f_y]*[87/(87+f_y)] = [(0.85*0.85*(3.5\ ksi))/(60\ ksi)]*[87/(87 + 60\ ksi)] = 0.02494345$
$\rho_{max} = 0.634*\rho_b = 0.0158141$ (See derivation below.)

$\rho_{max} > \rho$ which means the section is tension-controlled: $f_s = f_y$ and $\phi = 0.90$

$a = A_s*f_y/[(0.85)*(f'_c)*(b)] = (4.74\ in^2)*(60\ ksi)/[(0.85)*(3.5\ ksi)*(39”)] = 2.4512”$
$\phi M_n = \phi A_s*f_y*(d – a/2) = (0.9)*(4.74\ in^2)*(60\ ksi)*[(44.875”) – (2.4512”)/2]/(12\ in/ft) = 931.04\ k*ft$

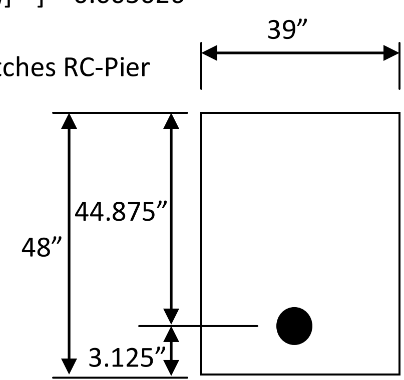
$f_r = 0.37*(f'_c)^{0.5} = 0.37*(3.5\ ksi)^{0.5} = 0.692207\ ksi$
$M_{cr} = f_r*I/c = [(0.692207\ ksi)*(1/12)*(39”)*(48”)^3]/[(0.5)*(48”)*(12\ in/ft)] = 863.87\ k*ft$

$1.2M_{cr} = (1.2)*(863.87\ k*ft) = 1036.65\ k*ft > \phi Mn = 931.04\ k*ft$
$1.2M_{cr} = 1036.65\ k*ft < (4/3)*M_{max} = (4/3)*(822.6\ k*ft) = 1096.8\ k*ft$

So $1.2M_{cr}$ = 1036.65 k*ft controls the design – Find required A_s based on $1.2*M_{cr}$

$R_n = R_u/\phi = M_u/(\phi*b*d^2) = (1036.65\ k*ft)*(12\ in/ft)/[(0.9)*(39”)*(44.875”)^2] = 0.176\ ksi$
$\rho = (0.85*f'_c/f_y)*[1 – (1 – (2*R_n)/(0.85*f'_c))^{0.5}]$
$\quad = [(0.85)*(3.5\ ksi)/(60\ ksi)]*[1 – [1 – ((2)*(0.176\ ksi))/((0.85)*(3.5\ ksi))]^{0.5}] = 0.003026$

Required $A_s = \rho*b*d = (0.003026)*(39”)*(44.875”) = 5.295\ in^2$ ← Matches RC-Pier



6 - #8s
$A_s = 4.74\ in^2$

Derivation of $\rho_{max} = 0.634*\rho_b$ [Ensuring tension controlled sections for singly-reinforced concrete beams.]

Compression Controlled		Transition		Tension Controlled
0.75	<=	$\phi = 0.65 + 0.15*(d_t/c – 1)$	<=	0.90
$\varepsilon_t <= 0.002$				$\varepsilon_t >= 0.005$
$d_t/c <= 1.667$				$d_t/c >= 2.667$
$c/d_t >= 0.600$				$c/d_t <= 0.375$

To get $\phi = 0.90$ the $\varepsilon_t >= 0.005$ and $c/d_t <= 0.375$ Note: $c/d_t = \varepsilon_u/(\varepsilon_u + \varepsilon_s)$

At yield: $\rho_b = [(0.85*B_1*f'_c)/f_y]*[87/(87+f_y)] = [(0.85*B_1*f'_c)/f_y]*[\varepsilon_u/(\varepsilon_u + \varepsilon_y)]$
$= [(0.85*B_1*f'_c)/f_y]*[0.003/(0.003 +(60\ ksi)/(29,000\ ksi))]$ $\varepsilon_y = 0.00207$ in/in
$= [(0.85*B_1*f'_c)/f_y]*(0.5918367)$
$= [(0.85*B_1*f'_c)/f_y]*(3/5)$ approximately

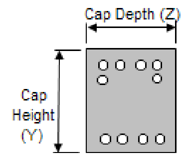
At $\varepsilon_s = 0.005$: $\rho_{max} = [(0.85*B_1*f'_c)/f_y]*[0.003/(0.003 + 0.005)]$
$= [(0.85*B_1*f'_c)/f_y]*(0.375)$
$= [(0.85*B_1*f'_c)/f_y]*(3/8)$ approximately

So $\rho_{max} = C_1*\rho_b$ ⇔ ~~$[(0.85*B_1*f'_c)/f_y]$~~$*(0.375) = C_1*$~~$[(0.85*B_1*f'_c)/f_y]$~~$*(0.5918367)$
$C_1 = 0.375/0.5918367 = 0.634$ which is approximately $[(3/8)/(3/5)] = 5/8 = 0.625$

So in order to ensure $\phi = 0.90$ the value of ρ must be less than the new $\rho_{max} = 0.634*p_b$

Spreadsheet for Pier Cap Flexure Design – Consider Discrete Point along Cap
Enter Moments from RC-Pier

DISCRETE FLEXURE DESIGN Aashto Lrfd 5.7
Design Top and Bottom R/I for One Discrete Location



Cap Height (Y)	4.000	ft	
Cap Depth (Z)	3.250	ft	
Concrete Strength, f'c	3.500	ksi	Aashto Lrfd 5.4.2.1
Reinforcement Yield Strength, fy	60.000	ksi	Aashto Lrfd 5.4.3.1
Reinforcement Modulus of Elasticity, Es	29000.000	ksi	Aashto Lrfd 5.4.3.2
Flexure Resistance Phi Factor, ϕ_f	0.900	Begin by assuming a tension-controlled section, ϕ_f= 0.9 -- Aashto Lrfd 5.5.4.2.	
Shear Resistance Phi Factor, ϕ_v	0.900	ϕ_v = 0.9 -- Aashto Lrfd 5.5.4.2.	

Design Bottom Cap Reinforcement

Factored Applied Positive Moment, Mu_z	822.600	k*ft

Factored applied shear will be used in calculation of additional longitudinal reinforcement. Vu would typically be concurrent with Mu.

Factored Applied Shear, Vu	489.100	k

Note: Use positive value for shears.

Rough Estimate of A_s required		4.162	in^2
Estimate of the number of bars required for bar sizes:	#7	7	
	#8	6	
	#9	5	
	#10	4	
	#11	3	

Bar Size for Flexural Reinforcement	8

Reinforcement Layer	d' (in) by Layer, Measured from **Bottom** of Cap	No. of Bars per Layer	d_s (in) by Layer
1	3.125	6	44.875
2	7.125		40.875
3	11.125		36.875
4	15.125		32.875

Total Bar Area Input, A_s provided	4.740	in^2
Distance from Top of Cap to C.G. of Bar Group, d_s	44.875	in

Note: Total bar area is lumped at its center of gravity.

Design Top Cap Reinforcement

Factored Applied Negative Moment, Mu_z	1423.900	k*ft

Note: Use positive value for negative moments.

Factored Applied Shear, Vu	496.900	k

Note: Use positive value for shears.

Rough Estimate of A_s required		7.341	in^2
Estimate of the number of bars required for bar sizes:	#7	13	
	#8	10	
	#9	8	
	#10	6	
	#11	5	

Bar Size for Flexural Reinforcement	9

Reinforcement Layer	d' (in) by Layer, Measured from **Top** of Cap	No. of Bars per Layer	d_s (in) by Layer
1	3.1875	8	44.8125
2	7.1875	6	40.8125
3	11.1875		36.8125
4	15.1875		32.8125

Total Bar Area Input, A_s provided	14.000	in^2
Distance from Bottom of Cap to C.G. of Bar Group, d_s	43.098	in

Note: Total bar area is lumped at its center of gravity.

Flexural Capacity Check Aashto Lrfd 5.7.3.2

Factored Applied Moment, Mu_z	822.600	k*ft
Depth of Equivalent Stress Block, a	2.451	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	931.042	k*ft
Is Mu_z <= Mr_z ?	**Yes**	

Factored Applied Moment, Mu_z	1423.900	k*ft
Depth of Equivalent Stress Block, a	7.240	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	2487.133	k*ft
Is Mu_z <= Mr_z ?	**Yes**	

Minimum Reinforcement Check Aashto Lrfd 5.4.2.6 and 5.7.3.3.2

Modulus of Rupture, f_r	0.692	ksi
Section Modulus of Cap, S_z	8.667	ft^3
120% of the Cracking Moment, 1.2^*Mcr_z	1036.649	k*ft
Is 1.2^*Mcr_z <= Mr_z ?	**No**	
---- OR ----		
A_s required	4.174	in^2
Is A_s prov'd >= 1.33^*A_s req'd ?	**No**	

Modulus of Rupture, f_r	0.692	ksi
Section Modulus of Cap, S_z	8.667	ft^3
120% of the Cracking Moment, 1.2^*Mcr_x	1036.649	k*ft
Is 1.2^*Mcr_x <= Mr_x ?	**Yes**	
---- OR ----		
A_s required	7.697	in^2
Is A_s prov'd >= 1.33^*A_s req'd ?	**Yes**	

Matches RC-Pier results on previous page.

Continued on next page.

Maximum Reinforcement Check Aashto Lrfd 5.5.4.2, 5.7.2.1, and 5.7.3.3

Stress Block Factor, $\beta1$	0.850			Stress Block Factor, $\beta1$	0.850	
Location of Neutral Axis, c	2.884	in		Location of Neutral Axis, c	8.517	in
Net Tensile Strain in the Extreme Tension Steel, ε_s	0.044	in/in		Net Tensile Strain in the Extreme Tension Steel, ε_s	0.013	in/in
Is Section Tension Controlled? ε_s >= 0.005	**Yes**			**Is Section Tension Controlled? ε_s >= 0.005**	**Yes**	
Is Section Compression Controlled? ε_s <= 0.002	**No**			**Is Section Compression Controlled? ε_s <= 0.002**	**No**	
Is Section in Transition? 0.005 > ε_s > 0.002	**No**			**Is Section in Transition? 0.005 > ε_s > 0.002**	**No**	
Flexure Phi Factor, ϕ, for Design	0.900			Flexure Phi Factor, ϕ, for Design	0.900	

NOTE: If section is in Transition, then the user must adjust the Flexural Phi Factor, ϕ, in cell G7.
If section is Compression Controlled, then the user can not use this spreadsheet, but must do a strain compatability analysis.

Additional Longit. R/I Check Aashto Lrfd 5.8.3.5

Stirrup Bar Size (i.e. #4, #5, etc.)	5			Stirrup Bar Size (i.e. #4, #5, etc.)	5	
Number of Stirrup Legs (Typ. 4 legs for double hoops)	4			Number of Stirrup Legs (Typ. 4 legs for double hoops)	4	
Stirrup Spacing at Location of Interest (i.e. at Mu)	6.000	in		Stirrup Spacing at Location of Interest (i.e. at Mu)	6.000	in

See Aashto Lrfd 5.8.3.4.1 for Simplified Shear Procedure or 5.8.3.4.2 for the General Modified Compression Field Theory

Beta, β	2.000			Beta, β	2.000	
Theta, θ	45.000	deg		Theta, θ	45.000	deg
Effective Shear Depth, dv	43.649	in		Effective Shear Depth, dv	39.478	in
Concrete Shear Resistance, V_c	201.277	k		Concrete Shear Resistance, V_c	182.043	k
Stirrup Shear Resistance, V_s	541.253	k		Stirrup Shear Resistance, V_s	489.531	k
Vn_{cs} = Vc + Vs	742.530	k		Vn_{cs} = Vc + Vs	671.574	k
Vn_{max} = 0.25*f'c*bv*dv	1489.536	k		Vn_{max} = 0.25*f'c*bv*dv	1347.197	k
Vn = minimum of Vn_{cs} and Vn_{max}	742.530	k		Vn = minimum of Vn_{cs} and Vn_{max}	671.574	k
ϕ_v*Vn	668.277	k		ϕ_v*Vn	604.417	k
Factored Applied Shear, Vu	489.100	k		Factored Applied Shear, Vu	496.900	k
Is Shear Design Adequate? ϕ_v*Vn > Vu	**Yes**			Is Shear Design Adequate? ϕ_v*Vn > Vu	**Yes**	

Note: Shear design must be adequate to continue. | Note: Shear design must be adequate to continue.

Total Longitudinal Reinforcement Needed	8.735	in^2		Total Longitudinal Reinforcement Needed	13.138	in^2
Additional Longitudinal Reinforcement Needed	3.995	in^2		Additional Longitudinal Reinforcement Needed	-0.862	in^2

If value for additional longitudinal reinforcement is zero or negative it means that this provision of the code is satisfied.

Is Additional Longitudinal R/I Requirement Met?	**No**		Is Additional Longitudinal R/I Requirement Met?	**Yes**

Note: Flexural and/or shear R/I should be increased until this requirement is met. | Note: Flexural and/or shear R/I should be increased until this requirement is met.

Is Flexural R/I Adequate?	**NO, More R/I is Required.**		**Is Flexural R/I Adequate?**	**YES, Flexural R/I is Adequate.**

Current policy is that this requirement must be met.

Continued on next page.

Crack Control: Flexure R/I Aashto Lrfd 5.7.3.4. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

Exposure Factor, γ_e (Typically 1.00)	1.000	

Class 1 and 2 exposure factors are 1.00 and 0.75 respectively.

Concrete Cover Thickness to R/I Center, d_c	3.125	in
βs	1.099	

User Value for Positive Service M_z	651.500	k*ft

See Aashto Lrfd 5.4.2.4 and 5.7.1 for E_c and n

Reinforcment Ratio, ρ	0.00271	
Concrete Modulus of Elasticity, E_c	3586.616	ksi
Modular Ratio, n	8.000	
Factor for Distance to Neutral Axis, k	0.188	
Reinforcement Stress at Service Level	39.207	ksi

Max. Spacing of Bot Layer of Pos. Flexural R/I, s	9.989	in

Exposure Factor, γ_e (Typically 1.00)	1.000	

Class 1 and 2 exposure factors are 1.00 and 0.75 respectively.

Concrete Cover Thickness to R/I Center, d_c	3.188	in
βs	1.102	

User Value for Negative Service M_z	1141.000	k*ft

Note: Use positive value for negative moments.

Reinforcment Ratio, ρ	0.00833	
Concrete Modulus of Elasticity, E_c	3586.616	ksi
Modular Ratio, n	8.000	
Factor for Distance to Neutral Axis, k	0.304	
Reinforcement Stress at Service Level	25.255	ksi

Max. Spacing of Top Layer of Neg. Flexural R/I, s	18.785	in

Crack Control: Skin R/I Aashto Lrfd 5.7.3.4. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

Is Skin R/I Required ? (Is $d_e = d_s > 3.00'$?)	**Yes**	
Area of Skin R/I Required per Face, A_{sk}	0.179	in^2 per ft
Max Spacing of Skin R/I Required	7.479	in

Is Skin R/I Required ? (Is $d_e = d_s > 3.00'$?)	**Yes**	
Area of Skin R/I Required per Face, A_{sk}	0.157	in^2 per ft
Max Spacing of Skin R/I Required	7.183	in

Shrinkage and Temp. R/I Aashto Lrfd 5.10.8. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

Area of Skin R/I Required per Face, A_{sk}	0.233	in^2 per ft
Max Spacing of Skin R/I Required	12.000	in

Area of Skin R/I Required per Face, A_{sk}	0.233	in^2 per ft
Max Spacing of Skin R/I Required	12.000	in

Fatigue in R/I Aashto Lrfd 5.5.3

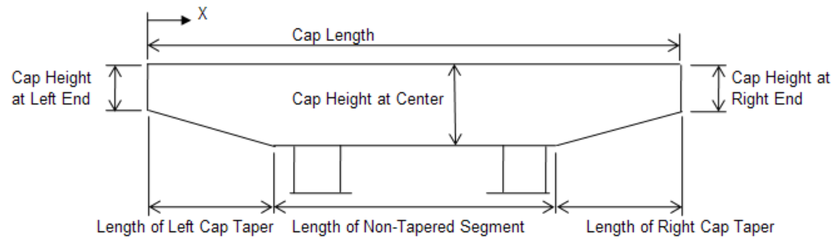
Office policy is to neglect checking fatigue.

Spreadsheet for Pier Cap Shear Design – Multiple Points along Cap
Enter Forces from RC-Pier

SIMPLIFIED SHEAR AND TORSION DESIGN See Aashto Lrfd 5.8 and specifically 5.8.3.4.1, β = 2.0 and θ = 45 degrees

Enter the basic pier cap dimensions in order to handle geometric variations.

Cap Depth (Z)	3.250	ft
Cap Height at Center	4.000	ft
Cap Height at Left End	3.000	ft
Cap Height at Right End	3.000	ft
Cap Length (X dir'n)	45.000	ft
Length of Left Cap Taper	5.000	ft
Length of Right Cap Taper	5.000	ft
Length of Non-Tapered Segment	35.000	ft
Concrete Strength, f'c	3.500	ksi
Reinforcement Yield Strength, fy	60.000	ksi
Reinforcement Modulus of Elasticity, Es	29000.000	ksi
Shear and Torsion Phi Factor, ϕ_v	0.900	
Modulus of Rupture, f_r	0.692	ksi
Bar Size for Shear Stirrups	5	
Number of Stirrup Legs	4	
Total Area of Shear Stirrups	1.240	in^2



Note: 1.) Aashto Lrfd 5.10.8 limits spacing of temperature and shrinkage R/I to 12" for components greater than 36" thick -- this will be the case for all pier caps, thus this spreadsheet limits the maximum stirrup spacing to 12".
2.) Final stirrup spacing is rounded down to the nearest 0.5" increment.
3.) The requirement for considering torsion is checked; however, torsion is not considered in this spreadsheet even when it is required.
4.) β = 2.0 and θ = 45 degrees according to Aashto Lrfd 5.8.3.4.1. Since the cap must have at least the minimum reinforcement specified in Aashto Lrfd 5.8.2.5, then Aashto Lrfd 5.8.2.4 does not need to be considered.

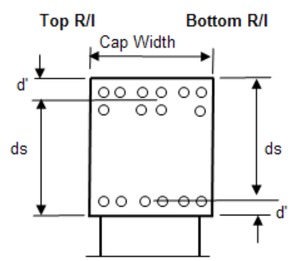
Bars in Top of Cap

Bar Size for Flexural R/I (i.e. #9, #10, etc.)	9

Reinforcement Layer Layers go from the top to the bottom	**d' (in) by Layer** Measured from cap top to each individual layer	**No. of Bars per Layer**
1	3.1875	8
2	7.1875	6
3	11.1875	
4	15.1875	

Total Bar Area Input, A_s provided	14.000	in^2
Dist. from Top of Cap to C.G. of Bar Group, d'	4.902	in

Total bar area is lumped at its center of gravity.



Bars in Bottom of Cap

Bar Size for Flexural R/I (i.e. #9, #10, etc.)	8

Reinforcement Layer Layers go from the bottom to the top	**d' (in) by Layer** Measured from cap bottom to each individual layer	**No. of Bars per Layer**
1	3.125	6
2	7.125	
3	11.125	
4	15.125	

Total Bar Area Input, A_s provided	4.740	in^2
Dist. from Bottom of Cap to C.G. of Bar Group, d'	3.125	in

Total bar area is lumped at its center of gravity.

Continued on the next page.

* If desired the user can reduce As provided by red'n factor, R, in order to account for bars not fully developed: Ase = R*As

** This factor will typically be 0.900 for tension-controlled sections
This spreadsheet is not valid for compression-controlled sections.

*** This is for information only.

Sequence	Dist. From Left End of Pier Cap, X (ft)	Cap Height at Section, H (ft)	Max Abs Moment Mu, (k*ft)	Max Abs Shear, Vu (k)	Max Abs Torsion Tu, (k*ft)	Flex. R/I: T = Top B = Bottom	d' (in)	ds (in)	Total Bar Area Provided, As (in^2)	* Reduction Factor for As, R	Calculate Factored Flexural Resistance for Effective Bar Area: Effective Bar Area Provided, Ase = R*As (in^2)	Depth of Equiv. Stress Block, a (in)	** Flexure Phi Factor, ϕ_f	*** Factored Flexural Resistance, Mr = ϕ_fMn (k*ft)
1	2.400	3.480	20.400	410.700	9.800	T	4.902	36.858	14.000	1.000	14.000	7.240	0.900	2094.013
2	5.000	4.000	1086.500	416.600	9.800	T	4.902	43.098	14.000	1.000	14.000	7.240	0.900	2487.133
3	5.760	4.000	1403.800	418.500	9.800	T	4.902	43.098	14.000	1.000	14.000	7.240	0.900	2487.133
4	7.240	4.000	1423.900	496.900	9.300	T	4.902	43.098	14.000	1.000	14.000	7.240	0.900	2487.133
5	9.170	4.000	609.000	492.200	9.300	T	4.902	43.098	14.000	1.000	14.000	7.240	0.900	2487.133
6	10.440	4.000	822.600	489.100	9.300	B	3.125	44.875	4.740	1.000	4.740	2.451	0.900	931.042
7	11.830	4.000	817.400	61.400	0.500	B	3.125	44.875	4.740	1.000	4.740	2.451	0.900	931.042
8	14.500	4.000	794.200	54.800	0.500	B	3.125	44.875	4.740	1.000	4.740	2.451	0.900	931.042
9	17.170	4.000	756.100	48.300	0.500	B	3.125	44.875	4.740	1.000	4.740	2.451	0.900	931.042
10	18.480	4.000	748.700	439.100	10.300	B	3.125	44.875	4.740	1.000	4.740	2.451	0.900	931.042
11	19.830	4.000	298.800	442.400	10.300	T	4.902	43.098	14.000	1.000	14.000	7.240	0.900	2487.133
12	21.760	4.000	886.800	447.100	10.300	T	4.902	43.098	14.000	1.000	14.000	7.240	0.900	2487.133

Continued

§ The effective shear depth, dv, is based on the maximum of: 0.9*ds, 0.72*h, ds - a/2 -- see Aashto Lrfd 5.8.2.9

§ Eff. Shear Depth Used, dv (in)
33.238
39.478
39.478
39.478
39.478
43.649
43.649
43.649
43.649
43.649
39.478
39.478

Check if Torsion Must Be Considered: Aashto Lrfd 5.8.2.1

Cap Area, Acp (in^2)	Cap Perimeter, pc (in)	Torsional Cracking Moment, Tcr (k*ft)	Limit for Consideration 0.25*ϕ*Tcr (k*ft)	Consider Torsion? Tu > 0.25*ϕ*Tcr (Y or N)
1628.640	161.520	320.027	72.006	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N
1872.000	174.000	392.487	88.310	N

Continued on the next page.

! If this limiting condition fails then the required stirrup spacing will read, "Increase Cap Size".

!! This spacing is used for checking additional long. R/I requirements.

Determine Stirrup Spacing:

! Aashto Lrfd Eq. 5.8.3.3-2 Max. Perm'ble Factored Shear Resistance, $Vr_{max} = \phi$*Vn = ϕ*0.25*f'c*dv*bv	Aashto Lrfd Eq. 5.8.3.3-3 Factored Concrete Shear Resistance, ϕ*Vc	Required Factored Reinforcement Shear Resistance, ϕ*Vs = Vu -ϕ*Vc	Aashto Lrfd Equation 5.8.3.3-4 Spacing, s = Av*fy*dv* … … cot(θ)/Vs	Aashto Lrfd Equation 5.8.2.5-1 Spacing, s = Av*fy/(0.0316* … … bv*f'c^0.5)	Aashto Lrfd Equation 5.8.2.7-1 & 5.8.2.7-2 Spacing, s	Required Stirrup Spacing, s	!! User Entered Stirrup Spacing, s
(k)	(k)	(k)	(in)	(in)	(in)	(in)	(in)
1020.831	137.942	272.758	8.160	32.269	24.000	**8.000**	**5.000**
1212.477	163.839	252.761	10.458	32.269	24.000	**10.000**	**6.000**
1212.477	163.839	254.661	10.380	32.269	24.000	**10.000**	**6.000**
1212.477	163.839	333.061	7.937	32.269	24.000	**7.500**	**6.000**
1212.477	163.839	328.361	8.050	32.269	24.000	**8.000**	**6.000**
1340.582	181.149	307.951	9.491	32.269	24.000	**9.000**	**6.000**
1340.582	181.149	0.000	N/A	32.269	24.000	**12.000**	**16.500**
1340.582	181.149	0.000	N/A	32.269	24.000	**12.000**	**16.500**
1340.582	181.149	0.000	N/A	32.269	24.000	**12.000**	**16.500**
1340.582	181.149	257.951	11.331	32.269	24.000	**11.000**	**8.000**
1212.477	163.839	278.561	9.490	32.269	24.000	**9.000**	**8.000**
1212.477	163.839	283.261	9.332	32.269	24.000	**9.000**	**8.000**

Check Additional Longit. R/I Requirements, Eq. 5.8.3.5-1

Mu/(dv*ϕ_f)	Vu/ϕ_v	0.5*Vs	As*fy	Total Longit. Reinf., As	Additional Longit. Reinf., As
(k)	(k)	(k)	(k)	(in^2)	(Y or N)
8.183	456.333	228.167	236.350	3.939	0.000
366.953	462.889	231.444	598.397	9.973	0.000
474.117	465.000	232.500	706.617	11.777	0.000
480.905	552.111	244.765	788.251	13.138	0.000
205.683	546.889	244.765	507.806	8.463	0.000
251.275	543.444	270.626	524.093	8.735	3.995
249.687	68.222	34.111	283.798	4.730	0.000
242.600	60.889	30.444	273.044	4.551	0.000
230.962	53.667	26.833	257.795	4.297	0.000
228.701	487.889	202.970	513.620	8.560	3.820
100.916	491.556	183.574	408.898	6.815	0.000
299.506	496.778	183.574	612.710	10.212	0.000

Pier Cantilever Design

There is only one beam on the cantilever.

DC Loads	Exterior Beam	Loads were calculated earlier in example.
Beam	55.558 k	
Slab	60.525 k	
Haunch	1.740 k	
Intermediate Diaphragm	1.574 k	
Pier Diaphragm	8.799 k	
Pier Steps	-------	
SBC	43.590 k	One entire SBC distributed to exterior beam.

DW Loads	Exterior Beam
FWS	13.643 k

Live Load

Max. LL+I Reaction = 185.558 k   Dual Truck Train + Lane Controls

Interior Beam Spacing = 7.401'   Perpendicular to C.L. roadway
Slab Overhang = 3.083'

Since the slab overhang is less than half of the interior beam spacing, then the shear distribution factor should be based on the interior beam shear distribution factor according to Bridge Design Manual 5.4.2.2.2.

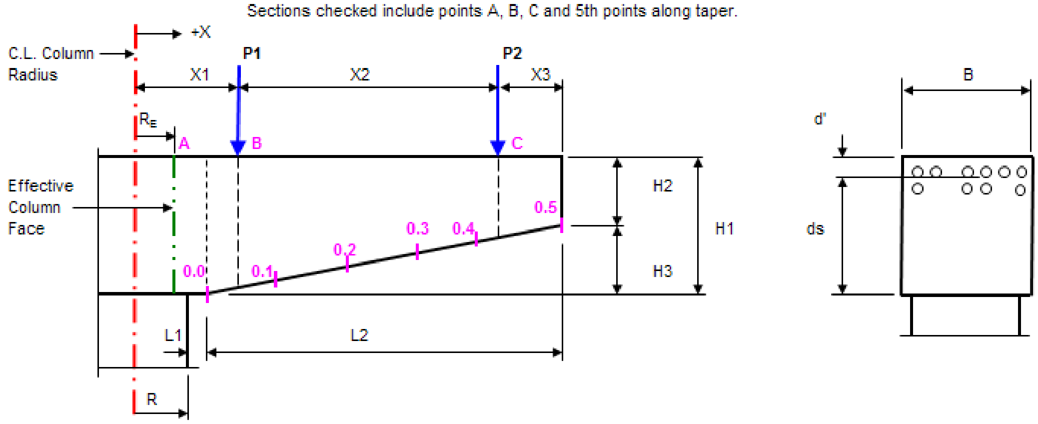
Shear Distribution Factor = 0.821   With skew correction factor

Live Load on Exterior Beam = (185.558 k)*(0.821) = 152.343 k

Spreadsheet for Pier Cap Overhang Design

PIER CAP OVERHANG DESIGN

Includes T-Piers or Frame Piers with either one or two beams on the overhang.
Sections checked include points A, B, C and 5th points along taper.



General Input

Note: X1, H3, and R_E are calculated based on other dimensions

All dimensions are along skew of pier cap

Is this a T-Pier? Y or N	N	This entry only affects the calculation of R_E	
Edge Dist. bt. Column and Cap Taper, L1 (Typ. 3")	3.000	in	
Taper Length, L2	5.000	ft	
Column Radius, R	1.250	ft	
Cap Height, H1	4.000	ft	Note: Input items P1 and X2 shall be set to 0 if there is only one beam on the overhang. [P1 shall only be considered to be on the overhang if it falls to the right of the effective column face.]
Cap Height at End, H2	3.000	ft	
Beam Spa, X2 -- Enter 0 if only one beam on overhar	0.000	ft	
Distance bt. Exterior Beam and Cap End, X3	2.401	ft	
Pier Cap Depth, B	3.250	ft	
Estimated Dist. from Cap Top to C.G. of Top R/I, d'	5.000	in	

Concentrated Beam Loads Enter 0 kips for all Int. Beam Loads (P1 Loads) if only one beam is on the overhang.	Unfactored Loads Interior Beam, P1	 Exterior Beam, P2	Hover for extended comment	Load Factors	Factored Loads Interior Beam, P1	 Exterior Beam, P2	
DC Load (DC1 and DC2)	0.000	171.786	k	1.250	0.000	214.733	k
DW Load	0.000	13.643	k	1.500	0.000	20.465	k
LL+IM Load (Truck with Impact and Lane)	0.000	152.343	k	1.750	0.000	266.600	k
Total Load	0.000	337.772	k		0.000	501.797	k

	Unfactored Load		Load Factors	Factored Load	
Distributed Pier Diaphragm Weight	0.001	k/ft	1.250	0.001	k/ft

Bar Size for the Side Reinforcing Bars	5	Currently not used

Concrete Strength, f'c	3.500	ksi
Reinforcement Yield Strength, fy	60.000	ksi
Reinforcement Modulus of Elasticity, Es	29000.000	ksi
Flexure Resistance Factor, ϕ_f	0.900	Begin by assuming a tension-controlled section, ϕ = 0.9 -- Aashto Lrfd 5.5.4.2
Shear Resistance Factor, ϕ_v	0.900	
Exposure Factor, γ_e (Typically 1.00)	1.000	Class 1 and 2 exposure factors are 1.00 and 0.75 respectively
Concrete Unit Weight, γ_c	0.150	kcf

Intermediate Calculations

Modular Ratio, n	8.000		
Height of Tapered Section, H3	1.000	ft	
Distance from C.L. Column to Interior Beam, X1	4.099	ft	If only one beam is on the overhang then this is the distance to the exterior beam
Dist. from C.L. Column to Effective Column Face, R_E	0.739	ft	Calculation is slightly different depending on pier type

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Dist. from C.L. Column to Point of Interest, X	0.739	4.099	4.099	1.500	2.500	3.500	4.500	5.500	6.500	ft
Dist. from Cap End to Point of Interest, X1+X2+X3-X	5.761	2.401	2.401	5.000	4.000	3.000	2.000	1.000	0.000	ft
Section Height, Hx	4.000	3.480	3.480	4.000	3.800	3.600	3.400	3.200	3.000	ft
Estimat'd Dist. from Cap Bot. to C.G. of Bar Group, d_s	3.583	3.064	3.064	3.583	3.383	3.183	2.983	2.783	2.583	ft
Factored Shear, Vu, due to P1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	k
Factored Shear, Vu, due to P2	501.80	501.80	501.80	501.80	501.80	501.80	0.00	0.00	0.00	k
Factored Shear, Vu, due to Diaphragm	0.01	0.00	0.00	0.01	0.01	0.00	0.00	0.00	0.00	k
Factored Shear, Vu, due to Pier Cap	12.52	4.74	4.74	10.66	8.29	6.03	3.90	1.89	0.00	k
Total Factored Shear, Vu	514.32	506.54	506.54	512.47	510.09	507.83	3.90	1.89	0.00	k
Factored Moment, Mu due to P1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	k*ft
Factored Moment, Mu due to P2	1686.28	0.00	0.00	1304.17	802.37	300.58	0.00	0.00	0.00	k*ft
Factored Moment, Mu due to Diaphragm	0.02	0.00	0.00	0.02	0.01	0.01	0.00	0.00	0.00	k*ft
Factored Moment, Mu due to Pier Cap	34.22	5.55	5.55	25.39	15.93	8.78	3.82	0.93	0.00	k*ft
Total Factored Moment, Mu	1720.52	5.55	5.55	1329.58	818.31	309.36	3.82	0.94	0.00	k*ft

Estimate Flexural R/I Required

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Rough Estimate of A_s required at each section	9.426	0.034	0.034	7.181	4.615	1.822	0.024	0.006	0.000	in^2

Rough Estimate of Maximum A_s required		9.426	in^2
Estimate of the number of bars required for bar sizes:	#7	16	
	#8	12	
	#9	10	
	#10	8	
	#11	7	

Note: The flexural reinforcement information on the left is an estimate of what is required for the overhang. In the next section of the spreadsheet the user can enter the actual reinforcement to be used in the design checks.

Enter Flexural and Shear R/I for Design Checks

Stirrup Bar Size (i.e. #4, #5, etc.)	5		
Number of Stirrup Legs (Typ. 4 legs for double hoops	4		
Total Area of Shear Stirrups	1.240	in^2	Stirrup spacing is entered later.

Bar Size for Flexural Reinforcement (i.e. #9, #10, etc	9

Layer	d' (in) by Layer	No. of Bars per Layer
1	3.1875	8
2	7.1875	6
3	11.1875	
4	15.1875	

Total Bar Area Input, A_s provided	14.000	in^2	
Distance from Top of Cap to C.G. of Bar Group, d'	4.902	in	Total bar area is lumped at its center of gravity.

Enter Flexural R/I Development Lengths

Aashto Lrfd 5.11.2

Available Cap Length for # 9 Development Length	67.138	in

Measured from critical sxn and assumes 2" end clearance.

Straight Bar Development Length (Non-Epoxy Coated R/I)		
Basic Development Length for a # 9 Bar	40.089	in
Development Length for a # 9 Bar	37.787	in
Enter Develop Length Used for a # 9 Bar	38.000	in
Is Cap Long Enough for Develop Length?	YES	

The 0.8 factor from Aashto Lrfd 5.11.2.1.3 is not applied.
The ratio of As required to As provided is applied to the develop length as G91/G113 -- see Aashto Lrfd 5.11.2.1.3.

Standard Hook Development Length (Non-Epoxy Coated R/I)		
Basic Development Length for a # 9 Bar	22.912	in
Development Length for a # 9 Bar	10.798	in
Enter Develop Length Used for a # 9 Bar	11.000	in
Is Cap Long Enough for Develop Length?	YES	

The 0.7 factor from Aashto Lrfd 5.11.2.4.2 is applied.
The 0.8 factor from Aashto Lrfd 5.11.2.4.2 is not applied.
The ratio of As required to As provided is applied to the develop length as G91/G113 -- see Aashto Lrfd 5.11.2.4.2.

Enter Bar End Type for Development (S = straight, H = hook)	S	
Development Length Used for a # 9 Bar	38.000	in

This is used to determine effective area of flexural R/I.

Check Flexural R/I

Flexural Capacity Check Aashto Lrfd 5.7.3.2

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Bar Area Provided, A_s	14.000	14.000	14.000	14.000	14.000	14.000	14.000	14.000	0.000	in^2
Fraction of Bar Area that is Effective	1.000	0.706	0.706	1.000	1.000	0.895	0.579	0.263	0.000	
Effective Bar Area Provided, A_{se}	14.000	9.878	9.878	14.000	14.000	12.526	8.105	3.684	0.000	in^2
Factored Applied Moment, Mu_z	1720.517	5.554	5.554	1329.577	818.309	309.357	3.821	0.935	0.000	k*ft
Depth of Equivalent Stress Block, a	7.240	5.108	5.108	7.240	7.240	6.478	4.191	1.905	0.000	in
Distance from Cap Bottom to C.G. of Bar Group, d_s	43.098	36.861	36.861	43.098	40.698	38.298	35.898	33.498	31.098	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	2487.133	1524.974	1524.974	2487.133	2335.933	1976.240	1232.901	539.572	0.000	k*ft
Is Mr_z >= Mu_z ?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	

Minimum Reinforcement Check Aashto Lrfd 5.4.2.6 and 5.7.3.3.2

Modulus of Rupture, f_r	0.692	ksi

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	2487.133	1524.974	1524.974	2487.133	2335.933	1976.240	1232.901	539.572	0.000	k*ft
Section Modulus of Cap, S_z	8.667	6.561	6.561	8.667	7.822	7.020	6.262	5.547	4.875	ft^3
120% of the Cracking Moment, 1.2^*Mcr_z	1036.649	784.730	784.730	1036.649	935.575	839.685	748.979	663.455	583.115	k*ft
Is Mr_z >= 1.2^*Mrc_z ?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No	

---- OR ----

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Effective A_{se} provided	14.000	9.878	9.878	14.000	14.000	12.526	8.105	3.684	0.000	in^2
A_s required	9.402	0.033	0.033	7.163	4.603	1.817	0.024	0.006	0.000	in^2
1.33^*A_s required	12.504	0.045	0.045	9.527	6.122	2.417	0.031	0.008	0.000	in^2
Is Effective A_{se} prov'd >= 1.33^*A_s req'd ?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	

Maximum Reinforcement Check Aashto Lrfd 5.5.4.2, 5.7.2.1, and 5.7.3.3

If section is in Transition, then the user may adjust the Flexural Phi Factor, ϕ, in cell G54. If the section is compression-controlled then do not use this sheet.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Stress Block Factor, $\beta 1$	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	
Location of Neutral Axis, c	8.517	6.010	6.010	8.517	8.517	7.621	4.931	2.241	0.000	in
Net Tensile Strain in the Extreme Tension Steel, ε_s	0.013	0.016	0.016	0.013	0.012	0.013	0.020	0.044	0.000	in/in
Is Section Tension Controlled? ε_s >= 0.005	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	n/a	
Is Section Compression Controlled? ε_s <= 0.00	No	No	No	No	No	No	No	No	n/a	
Is Section in Transition? 0.005 > ε_s > 0.002	No	No	No	No	No	No	No	No	n/a	
Flexure Phi Factor, ϕ, for Design	0.900	0.900	0.900	0.900	0.900	0.900	0.900	0.900	0.900	

	Critical Points			Fifth Points Along Taper					
	A	B	C	0	1	2	3	4	5
Is Flexural R/I Adequate?	YES	YES	YES	YES	YES	YES	YES	YES	YES

Check Shear R/I

To increase size of shear reinforcement bars go to cell G101.
Maximum stirrup spacing is 12" based on the Bridge Design Manual.

Simplified Shear Design Aashto Lrfd 5.8 and specifically 5.8.3.4.1, β = 2.0 and θ = 45 degrees

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Total Factored Shear, Vu	514.325	506.541	506.541	512.468	510.090	507.834	3.903	1.890	0.000	k
Effective Shear Depth, dv	39.478	34.306	34.306	39.478	37.078	35.059	33.802	32.546	31.098	in
Max. Permissible Factored Shear Resistance, Vr_{max}	1212.477	1053.638	1053.638	1212.477	1138.767	1076.760	1038.159	999.557	955.104	k
Factored Concrete Shear Resistance, ϕVc	163.839	142.375	142.375	163.839	153.879	145.500	140.284	135.067	129.061	k
Req'd Factored Shear Reinforcement Resistance, ϕV	350.486	364.166	364.166	348.629	356.211	362.334	0.000	0.000	0.000	k
Stirrup Spacing, s, Aashto Lrfd Eq. 5.8.3.3-4	7.542	6.308	6.308	7.582	6.970	6.479	N/A	N/A	N/A	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.8.2.5-1	32.269	32.269	32.269	32.269	32.269	32.269	32.269	32.269	32.269	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.8.2.7-1 & 2	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	in
Final Stirrup Spacing, s	**7.500**	**6.000**	**6.000**	**7.500**	**6.500**	**6.000**	**12.000**	**12.000**	**12.000**	in

General Shear Design Aashto Lrfd 5.8 and specifically 5.8.3.4.2, variable β and θ

In Aashto Lrfd 5.8.3.4.2 there are a number of bulleted items that should be considerd when using this method for shear design. This spreadsheet does reduce A_s for the section under consideration based on development length. This spreadsheet does not base ε_s on the calculated value at d_v when the section under consideration is closer than d_v to the face of the support. Axial forces in the cap are not considered.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Total Factored Shear, Vu	514.325	506.541	506.541	512.468	510.090	507.834	3.903	1.890	0.000	k
Total Factored Moment, Mu	1720.517	5.554	5.554	1329.577	818.309	309.357	3.821	0.935	0.000	k*ft
Effective Shear Depth, dv	39.478	34.306	34.306	39.478	37.078	35.059	33.802	32.546	31.098	in
Max. Permissible Factored Shear Resistance, Vr_{max}	1212.477	1053.638	1053.638	1212.477	1138.767	1076.760	1038.159	999.557	955.104	k
Flexural R/I Strain, ε_s, Aashto Lrfd Eq. 5.8.3.4.2-4	0.00255	0.00354	0.00354	0.00252	0.00251	0.00280	0.00003	0.00004	0.00000	in/in
$\beta = 4.8/(1 + 750*\varepsilon_s)$, Aashto Lrfd Eq. 5.8.3.4.2-1	1.646	1.314	1.314	1.659	1.664	1.550	4.683	4.676	4.800	
$\theta = 29 + 3500*\varepsilon_s$, Aashto Lrfd Eq. 5.8.3.4.2-3	37.942	41.378	41.378	37.836	37.795	38.786	29.116	29.124	29.000	deg
Factored Concrete Shear Resistance, ϕVc	134.838	93.556	93.556	135.902	128.029	112.755	328.500	315.781	309.745	k
Req'd Factored Shear Reinforcement Resistance, ϕV	379.487	412.985	412.985	376.565	382.061	395.079	0.000	0.000	0.000	k
Stirrup Spacing, s, Aashto Lrfd Eq. 5.8.3.3-4	8.934	6.314	6.314	9.038	8.379	7.394	N/A	N/A	N/A	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.8.2.5-1	32.269	32.269	32.269	32.269	32.269	32.269	32.269	32.269	32.269	in
Stirrup Spacing, s, Aashto Lrfd Eq. 5.8.2.7-1 & 2	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	24.000	in
Final Stirrup Spacing, s	**8.500**	**6.000**	**6.000**	**9.000**	**8.000**	**7.000**	**12.000**	**12.000**	**12.000**	in

Additional Longitudinal Reinforcement

To increase the amount of longitudinal R/I go to cell G105 and following.
Aashto Lrfd 5.8.3.5

Base θ off of Shear Method 1 or 2	1	Method 1 = Simplified Shear, 2 = General Shear Design

The user has the opportunity to enter a stirrup spacing.	Critical Points			Fifth Points Along Taper						
The stirrup size and number of legs remain the same	A	B	C	0	1	2	3	4	5	
Stirrup Spacing Used	6.000	5.000	5.000	6.000	6.000	5.000	5.000	10.000	10.000	in
θ based on Shear Method 1	45.000	45.000	45.000	45.000	45.000	45.000	45.000	45.000	45.000	deg
$Mu/(dv*\phi_f)$	581.084	2.159	2.159	449.049	294.263	117.651	1.507	0.383	0.000	k
Vu	514.325	506.541	506.541	512.468	510.090	507.834	3.903	1.890	0.000	k
Vu/ϕ_v	571.472	562.823	562.823	569.408	566.766	564.260	4.336	2.100	0.000	k
0.5*Vs	244.765	255.240	255.240	244.765	229.885	260.842	2.168	1.050	0.000	k
$As*fy = Mu/(dv*\phi_f) + (Vu/\phi_v - 0.5*Vs)*\cot\theta$	907.791	309.742	309.742	773.692	631.144	421.069	3.675	1.433	0.000	k
Total As Needed	15.130	5.162	5.162	12.895	10.519	7.018	0.061	0.024	0.000	in^2
Additional Longitudinal R/I Needed, Alx	**1.130**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	in^2

Crack Control and S&T Reinforcement

Crack Control: Flexure R/I Aashto Lrfd 5.7.3.4. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.
See cell G56 to change the Exposure Factor, γ_e, which is typically set to 1.00 (Class 1) for pier caps.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Concrete Cover Thickness to R/I Center, d_c	3.188	3.188	3.188	3.188	3.188	3.188	3.188	3.188	3.188	in
βs	1.102	1.118	1.118	1.102	1.107	1.114	1.121	1.129	1.139	
Service Moment, Ms due to P1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	k*ft
Service Moment, Ms due to P2	1135.075	0.000	0.000	877.869	540.097	202.325	0.000	0.000	0.000	k*ft
Service Moment, Ms due to Diaphragm	0.017	0.003	0.003	0.013	0.008	0.005	0.002	0.001	0.000	k*ft
Service Moment, Ms due to Pier Cap	27.374	4.440	4.440	20.313	12.740	7.020	3.055	0.748	0.000	k*ft
Total Service Moment, Ms	1162.466	4.443	4.443	898.194	552.845	209.350	3.057	0.748	0.000	k*ft
Reinforcment Ratio, ρ	0.00833	0.00974	0.00974	0.00833	0.00882	0.00937	0.01000	0.01072	0.01154	
Factor for Distance to Neutral Axis, k	0.304	0.324	0.324	0.304	0.312	0.319	0.328	0.337	0.347	
Reinforcement Stress at Service Level	25.731	0.116	0.116	19.881	12.993	5.244	0.082	0.022	0.000	ksi
Max. Spacing of Bot Layer of Pos. Flexural R/I, s	**18.3**	**5397.9**	**5397.9**	**25.6**	**42.3**	**113.5**	**7613.0**	**28739.7**	**n/a**	in

Crack Control: Skin R/I Aashto Lrfd 5.7.3.4. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

	Critical Points			Fifth Points Along Taper						
	A	B	C	0	1	2	3	4	5	
Is Skin R/I Required ? (Is $d_e = d_s > 3.00'$?)	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	**No**	**No**	**No**	
Area of Skin R/I Required per Face, A_{sk}	**0.157**	**0.082**	**0.082**	**0.157**	**0.128**	**0.100**	**0.000**	**0.000**	**0.000**	in^2/ft
Max Spacing of Skin R/I Required	**7.183**	**6.143**	**6.143**	**7.183**	**6.783**	**6.383**	**5.983**	**5.583**	**5.183**	in

Shrinkage and Temp. R/I Aashto Lrfd 5.10.8. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.

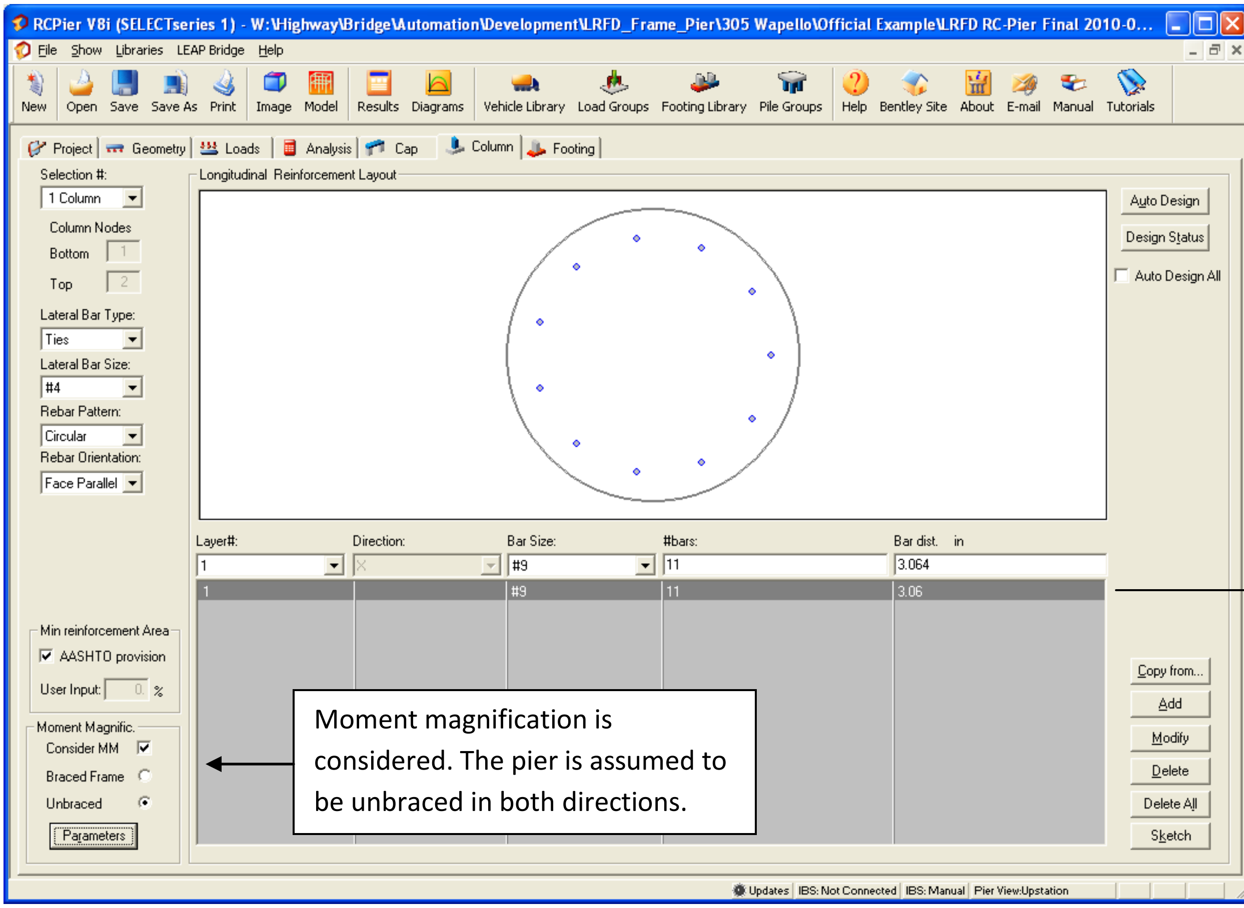
Area of Skin R/I Required per Face, A_{sk}	**0.233**	**0.218**	**0.218**	**0.233**	**0.228**	**0.222**	**0.216**	**0.210**	**0.203**	in^2/ft
Max Spacing of Skin R/I Required	**12.000**	**12.000**	**12.000**	**12.000**	**12.000**	**12.000**	**12.000**	**12.000**	**12.000**	in

Fatigue in Reinforcement

Aashto Lrfd 5.5.3

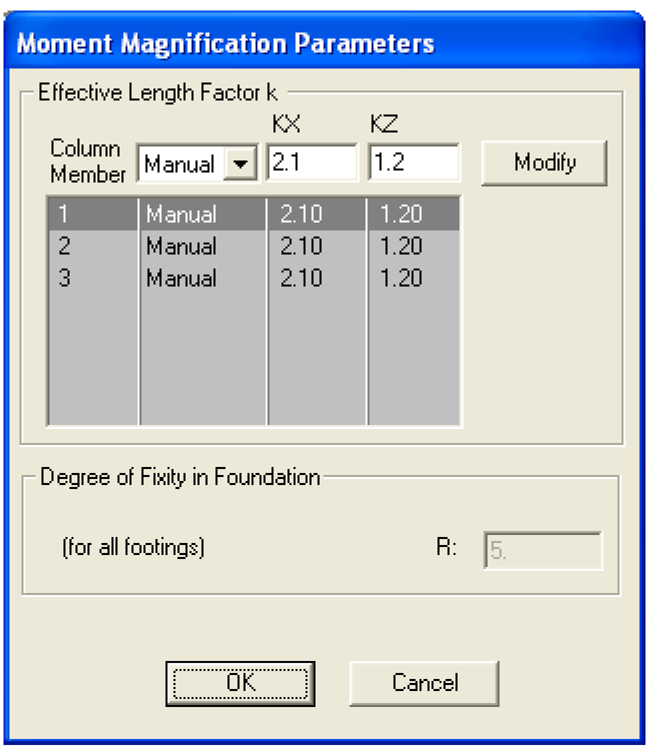
Office policy is to neglect checking fatigue.

Column Design: RC-Pier



Moment magnification is considered. The pier is assumed to be unbraced in both directions.

Column R/I can be auto-designed; however, I manually entered the column R/I that was specified on the plans.



The effective length factors were entered manually.

RC-Pier calculates magnification factors a little differently than Aashto. For instance, only the sidesway term is used for unbraced frames.

Column Design

I've included some portions of RC-Pier's output for the column

COLUMN DESIGN - Column: 1
Code: AASHTO LRFD 2007
Units: US
Pier View: Upstation.
Design/Analysis Method: Moment Magnification - Unbraced Column.

Column Type: Round D = 30.00 in

Column Section Properties

Sec.	Area ft^2	Ixx in^4	Izz in^4
1	4.91	39760.78	39760.78

DESIGN PARAMETERS	
f'c = 3500.0 psi	fy = 60000.0 psi
phi tens = 0.90	phi comp = 0.75
Tens below = 0.375	Comp Above = 0.600
Ec = 3586.6 ksi	Es = 29000 ksi
Concrete Type : Normal Weight.	

Reinforcement
Rebar Pattern: Circular
Rebar Orientation: Face Parallel

Reinforcement Schedule

Layer	Dir	Size	No. bars	Bar Dist. in
1	X	9	11	3.06

Reinforcement summary
Main bars summary:
11 # 9 bars
Total number of bars in the column: 11
Ties size: # 4

$r_g = (I/A)^{0.5} = [(0.25*\pi*r^4)/(\pi*r^2)] = 0.5*r$
$= (0.5)*(30''/2) = 7.5''$ Aashto Lrfd 5.7.4.3

$KL/r_g = [(2.1)*(20.5')*(12\ in/ft)]/(7.5'') = 68.88 > 22$
< 100

Magnification factors are fairly large. This typically occurs when the slenderness ratio approaches 70. See appendix for calculation of moment magnification factors for bottom of column 1. The Bridge Manual states that if magnification factors exceed 1.25 then the footing and piling shall be designed for magnification as well. This won't be done in this example. Typically it may be better to increase column diameter in order to reduce magnification factors below 1.25.

Moment Magnification calculation - Mx (global)

Loc ft	Comb	K	Cm	Beta	Delta B	Delta S	pPc kips
0.00	88	2.1000	1.0000	0.2469	—	2.1522	1268.88
20.50	88	2.1000	1.0000	0.5494	—	3.1849	1021.15
0.00	1010	2.1000	1.0000	0.1722	—	2.0043	1349.69
20.50	1010	2.1000	1.0000	0.5837	—	3.0941	999.01

Moment Magnification calculation - Mz (global)

Loc ft	Comb	K	Cm	Beta	Delta B	Delta S	pPc kips
0.00	88	1.2000	1.0000	0.3995	—	1.2787	3462.32
20.50	88	1.2000	1.0000	0.2638	—	1.2486	3834.02
0.00	1010	1.2000	1.0000	0.5932	—	1.2950	3041.27
20.50	1010	1.2000	1.0000	0.5837	—	1.2837	3059.47

Design values used after Moment Magnification (e-min effect included).

Loc ft	Comb	Fx kips	Fy kips	Fz kips	Mx kips-ft	My kips-ft	Mz kips-ft
0.00	88	-16.2	772.1	10.0	470.3	0.5	172.7
20.50	88	16.2	755.5	-10.0	-300.8	0.5	245.6
0.00	1010	2.2	727.6	14.6	627.8	0.7	-117.8
20.50	1010	-1.7	711.1	-14.0	-275.0	0.7	-114.1

COLUMN DESIGN

Bot/Top Elev. ft	Comb	Pu kips	Mux kips-ft	Muz kips-ft	pMn kips-ft	Incl deg	pPn/Pu	pMn/Mu
0.00	1010	727.6	627.8	117.8	651.3	10.63	1.00	1.01962
20.50	88	755.5	300.8	245.6	647.2	39.23	1.00	1.66680

Column passes. The controlling combination is plotted from RC-Pier a couple of pages over.

COLUMN DESIGN

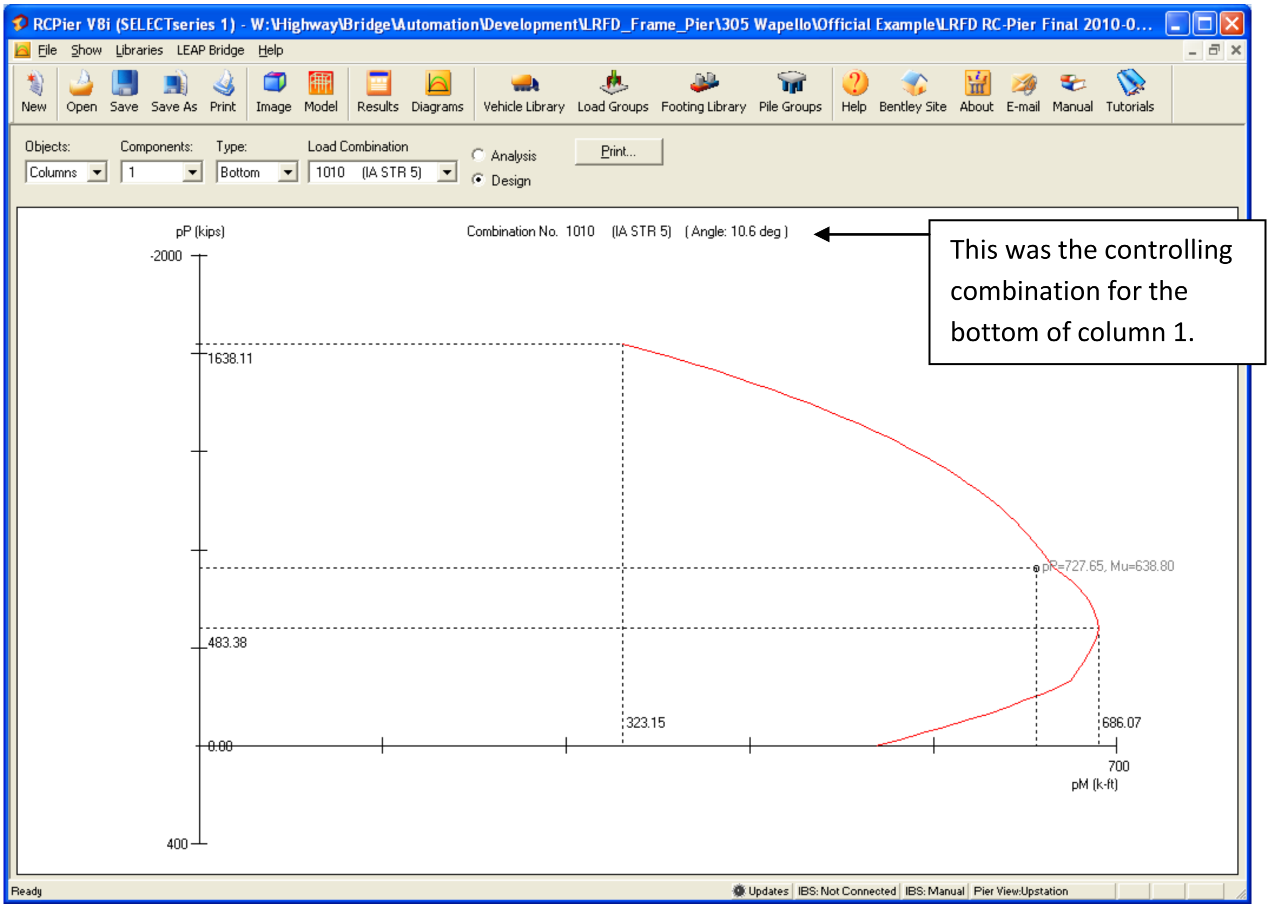
Bot/Top Elev. ft	Comb	As_min in^2	As_max in^2	As_req in^2
0.00	1010	5.57	56.55	11.00
20.50	88	5.57	56.55	11.00

K values for all columns used in unbraced moment magnification

Column	Kx	Kz
1	2.10	1.20
2	2.10	1.20
3	2.10	1.20

Column Design : Notes

Min reinforcement = 0.7875 % of Ag = 5.57 in^2.
For layer 1, direction X: Minimum bar distance is: 1.13 in. Effective bar distance is: 5.79 in.
— Values do not exist at that location as computation is done at the top and bottom of clear length of column.



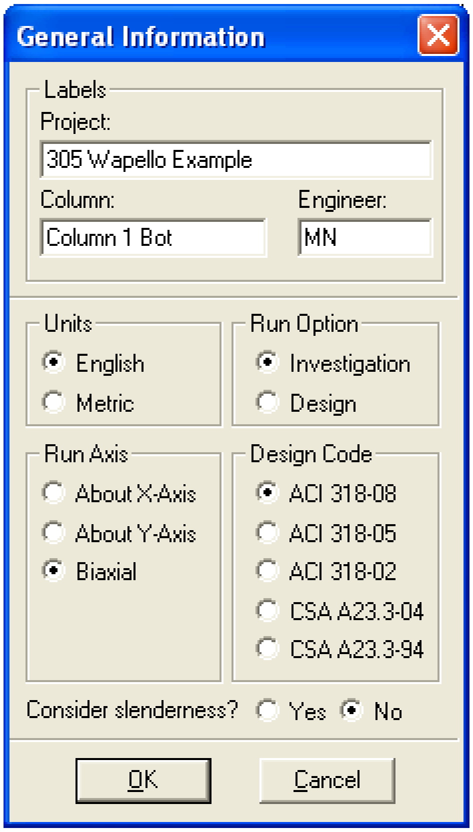
Pu = 727.6 k

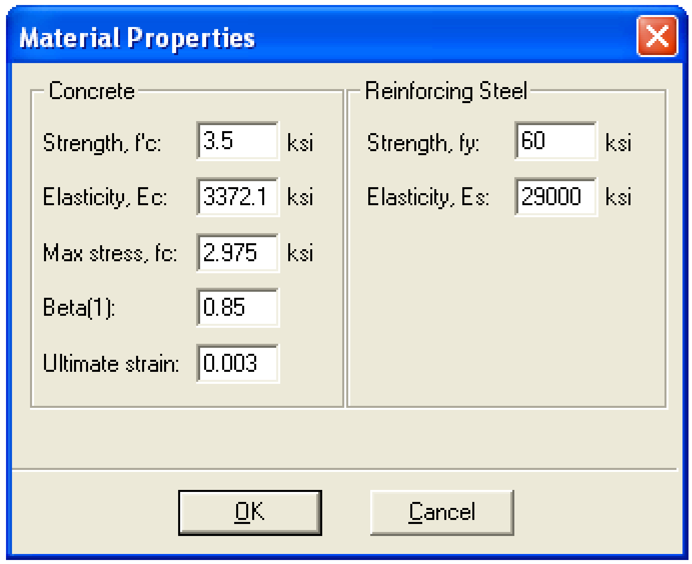
Mu = $[(627.8 \text{ k*ft})^2 + (117.8 \text{ k*ft})^2]^{0.5}$ = 638.8 k*ft

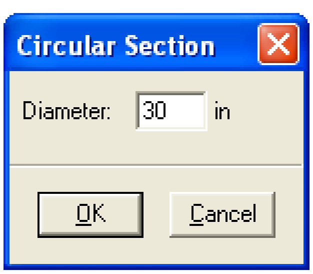
PR = ϕMn/Mu = (651.3 k*ft)/(638.8 k*ft) = 1.0196

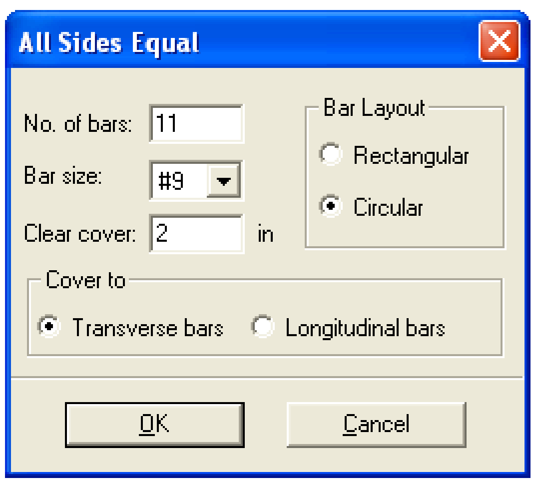
Column Design: spColumn v4.50

spColumn Input Screens

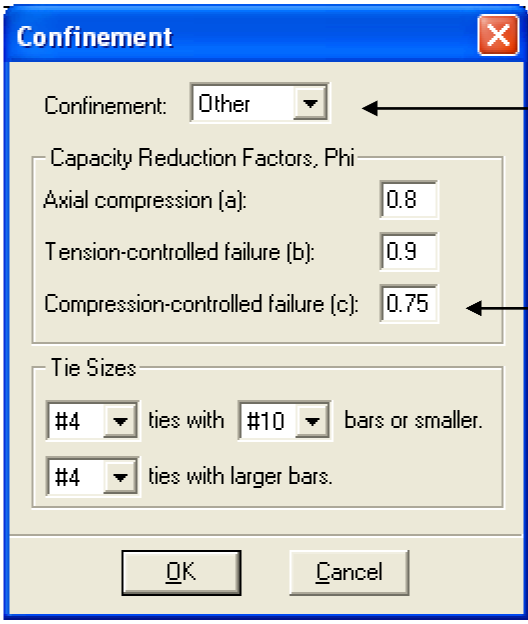




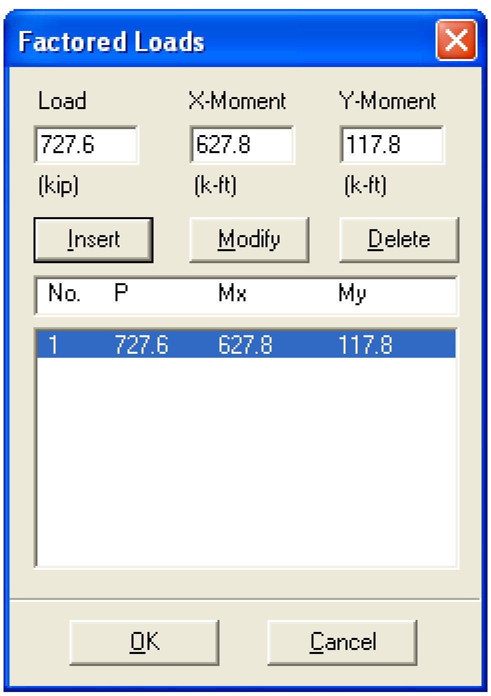




spColumn Input Screens



Confinement was set to "Other" in order to input modify the default Compression-controlled failure value to 0.75 for Aashto requirements.



These are the loads as found in Combination 1010 from RC-Pier.

spColumn Output Screens

```
STRUCTUREPOINT - spColumn v4.50 (TM)                                                          Page   1
Licensed to: Iowa Department of Transportation. License ID: 56461-1020268-4-29625-22F82       08/09/10
W:\Highway\Bridge\Automation\Development\LRFD_Frame_Pier\305 Wapello\Official Exa...\PierCol01.col      02:23 PM
```

```
                         oooooo              o
                        oo    oo            oo
           ooooo    oooooo    oo          ooooo    oo      oo   oo   o oooooooooo    o ooooo
          oo    o   oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
          oo        oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
           ooooo    oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
               oo   oooooo    oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
          o    oo   oo        oo    oo   oo   oo   oo  o   oo   oo    oo   oo   oo    oo   oo
           ooooo    oo         oooooo     ooooo     ooo     ooooo o   oo   oo   oo    oo   oo (TM)
```

===

spColumn v4.50 (TM)

Computer program for the Strength Design of Reinforced Concrete Sections

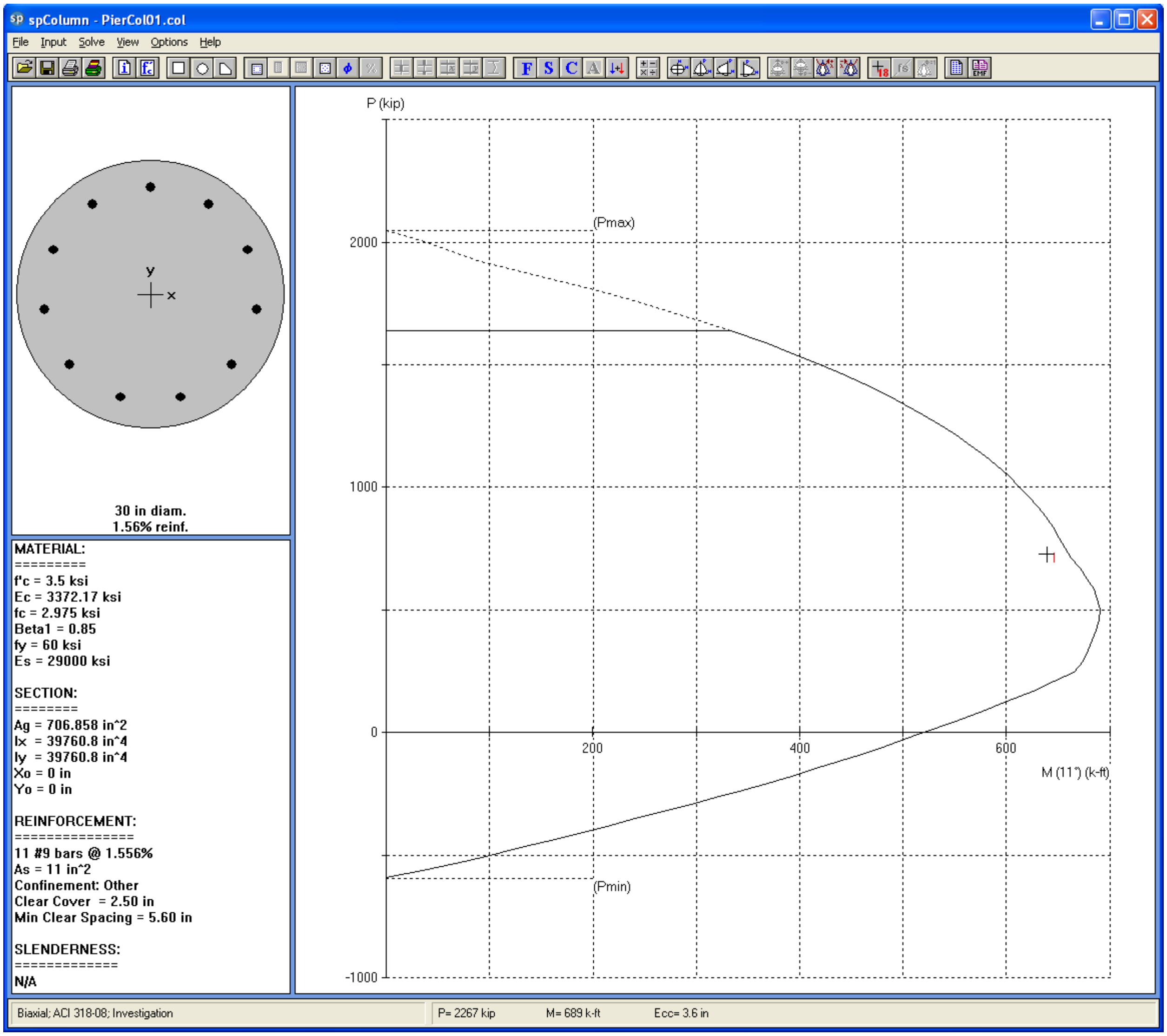
===

spColumn Output Screens

```
STRUCTUREPOINT - spColumn v4.50 (TM)                                                                Page   2
Licensed to: Iowa Department of Transportation. License ID: 56461-1020268-4-29625-22F82             08/09/10
W:\Highway\Bridge\Automation\Development\LRFD_Frame_Pier\305 Wapello\Official Exa...\PierCol01.col      02:23 PM


General Information:
====================
  File Name: W:\Highway\Bridge\Automation\Development\LRFD_Frame_Pier\305 Wapello\Off...\PierCol01.col
  Project:  305 Wapello Example
  Column:   Column 1 Bot                      Engineer: MN
  Code:     ACI 318-08                        Units: English

  Run Option: Investigation                   Slenderness: Not considered
  Run Axis:   Biaxial                         Column Type: Structural

Material Properties:
====================
  f'c   = 3.5 ksi                             fy   = 60 ksi
  Ec    = 3372.17 ksi                         Es   = 29000 ksi
  Ultimate strain = 0.003 in/in
  Beta1 = 0.85

Section:
========
  Circular:    Diameter = 30 in

  Gross section area, Ag =  706.858 in^2
  Ix =  39760.8 in^4                          Iy =  39760.8 in^4
  rx =  7.5 in                                ry =  7.5 in
  Xo =  0 in                                  Yo =  0 in

Reinforcement:
==============
  Bar Set: ASTM A615
  Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
  ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
  #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
  #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
  #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
  # 14      1.69        2.25   # 18      2.26        4.00

  Confinement: Other; #4 ties with #10 bars,  #4 with larger bars.
  phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.75

  Layout: Circular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 11.00 in^2 at rho = 1.56%
  Minimum clear spacing = 5.60 in

  11 #9   Cover = 2 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
               Pu         Mux         Muy        fMnx        fMny   fMn/Mu NA depth Dt depth    eps_t    Phi
  No.         kip        k-ft        k-ft        k-ft        k-ft               in       in
  --- ----------- ----------- ----------- ----------- ----------- -------- -------- -------- -------- ------
    1      727.60      627.80      117.80      648.67      121.72    1.033    16.09    26.75  0.00199  0.750

  *** End of output ***
```

There is a good match between the calculated performance ratios from spColumn and RC-Pier.

spColumn Interaction Diagram



Aashto Lrfd 5.7.4.4 and 5.5.4.2

$P_r = \phi P_n = \phi\phi_a P_o$ where $P_o = 0.85 * f'_c * (A_g - A_{st}) + f_y * A_{st}$

$P_n = 0.80 * [(0.85) * (3.5\ ksi) * (706.858\ in^2 - 11\ in^2) + (60\ ksi) * (11\ in^2)] = 2184.142\ k$

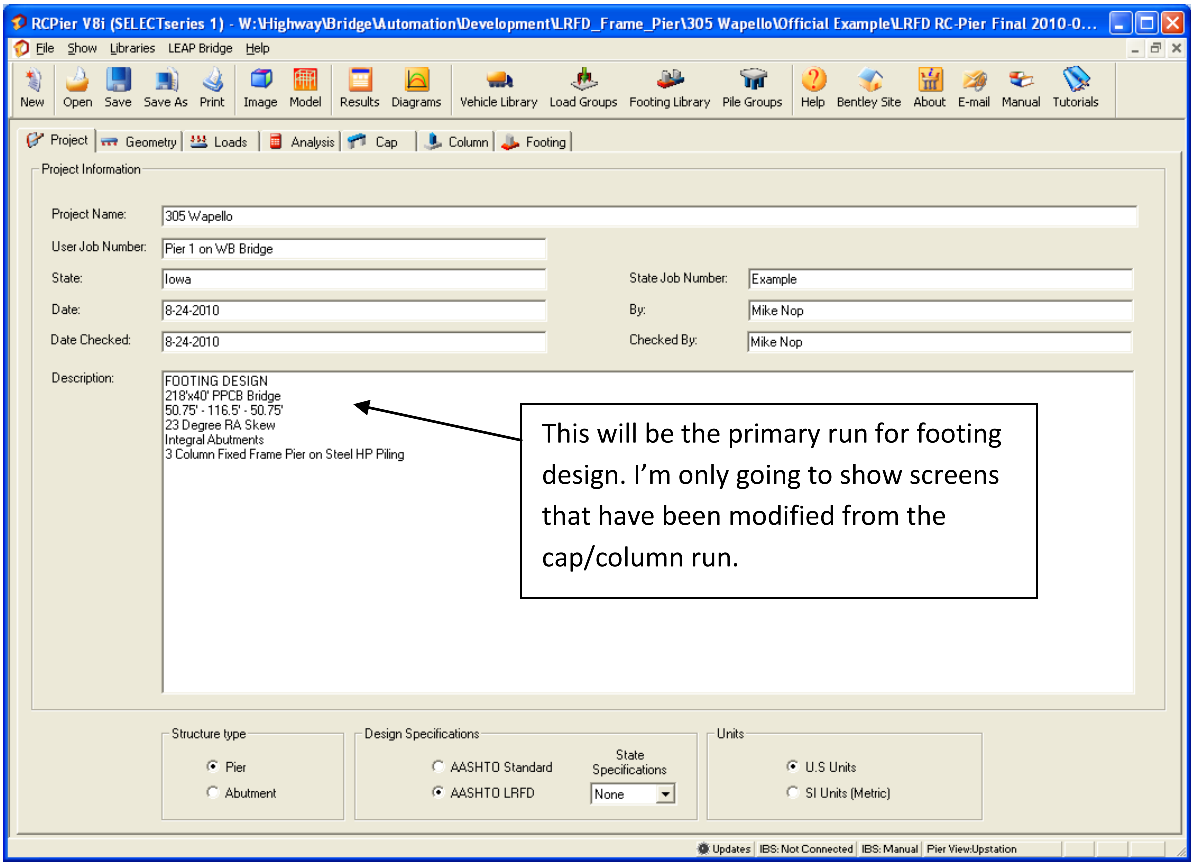
$\phi_b = 0.90$ for tension-controlled

$\phi_c = 0.75$ for compression-controlled

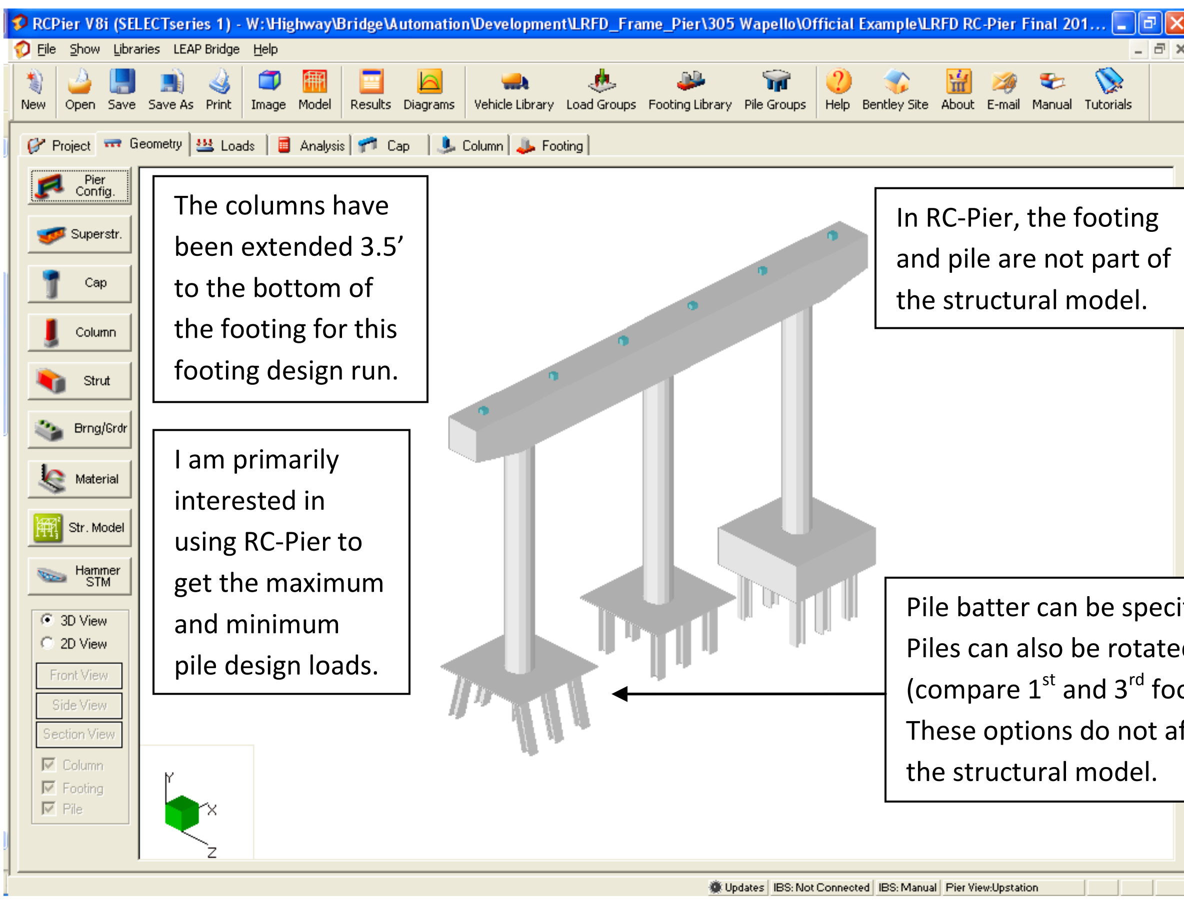
$\phi_a = 0.80$

$Pr = \phi_c P_n = (0.75) * (2184.142\ k) = 1638.107\ k$

Footing Design



This will be the primary run for footing design. I'm only going to show screens that have been modified from the cap/column run.



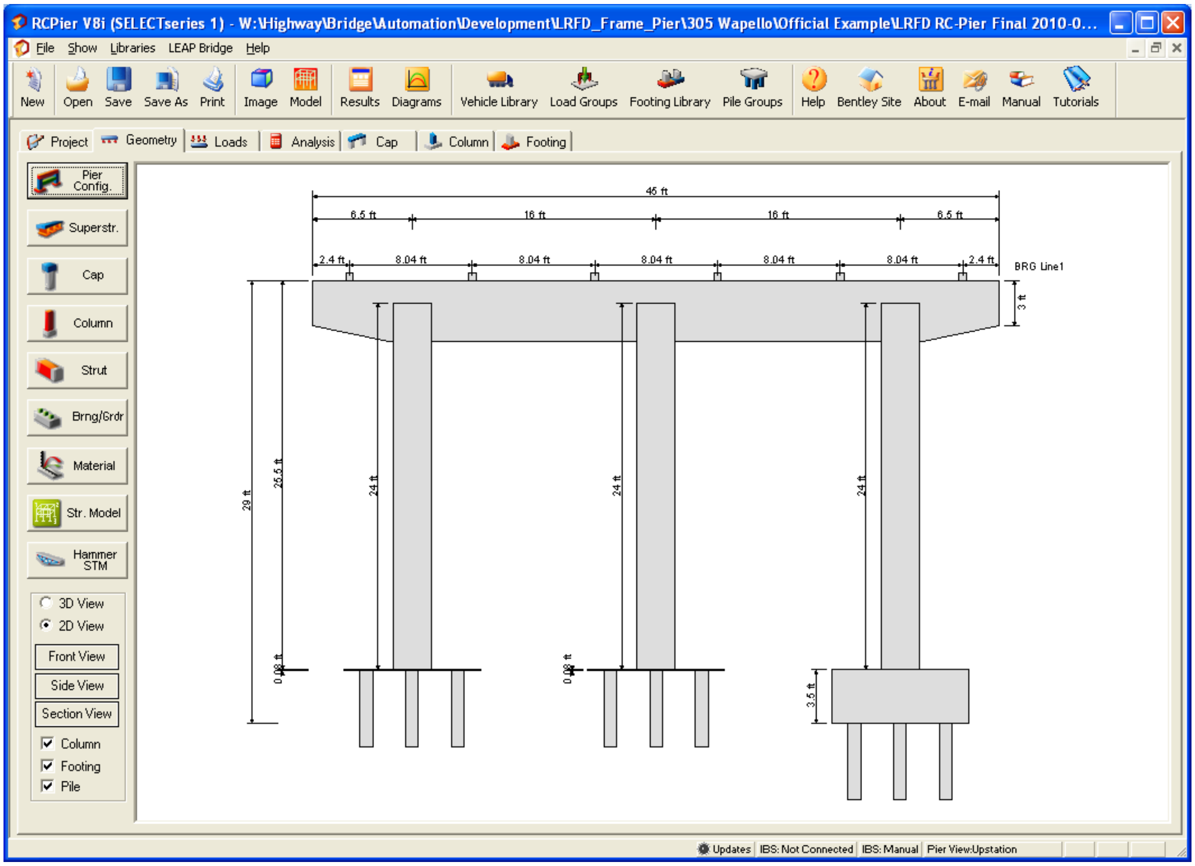
The columns have been extended 3.5' to the bottom of the footing for this footing design run.

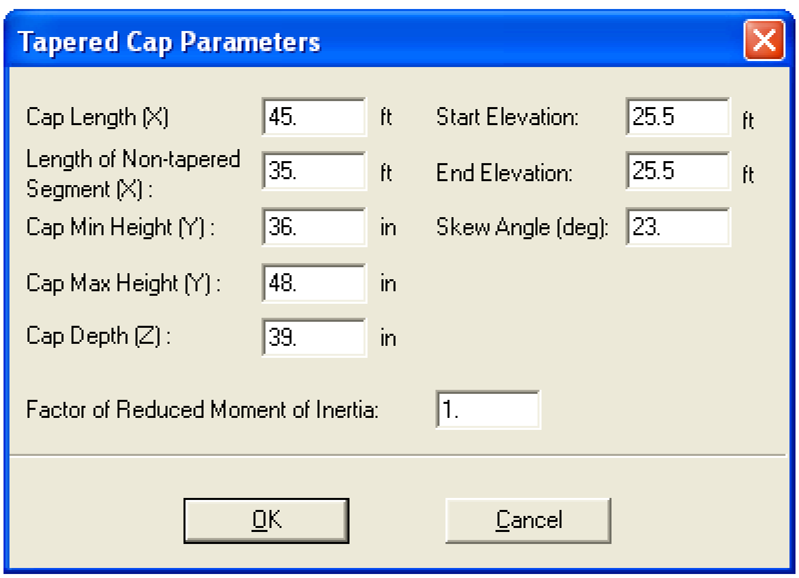
I am primarily interested in using RC-Pier to get the maximum and minimum pile design loads.

In RC-Pier, the footing and pile are not part of the structural model.

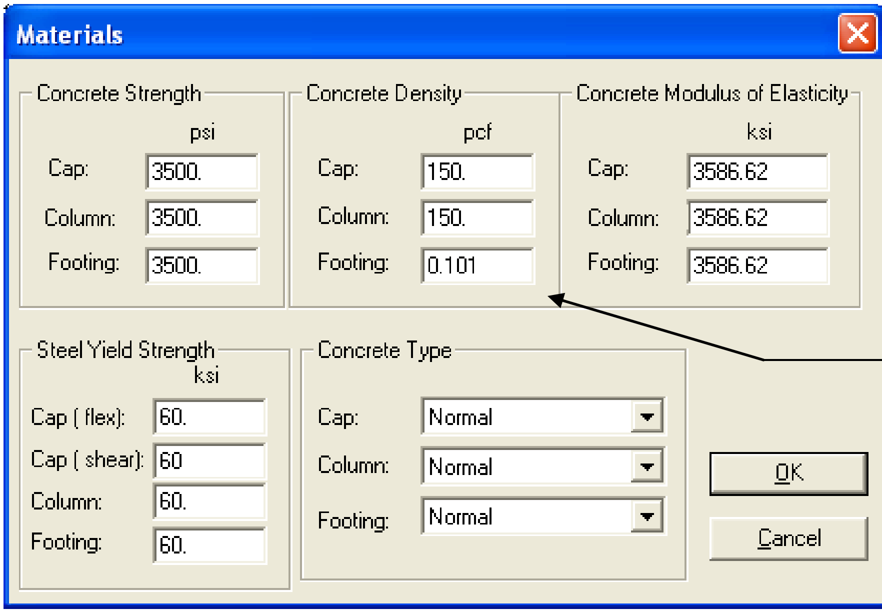
Pile batter can be specified. Piles can also be rotated (compare 1st and 3rd footing). These options do not affect the structural model.

I am going to look at maximum and minimum pile loads for the left most footing only. The footing and fill weight will be placed on the bottom of the column from the "Loads" tab rather than through the entries on the "Footing" tab. Handling loads this way allows me to include these loads in an analysis results file.

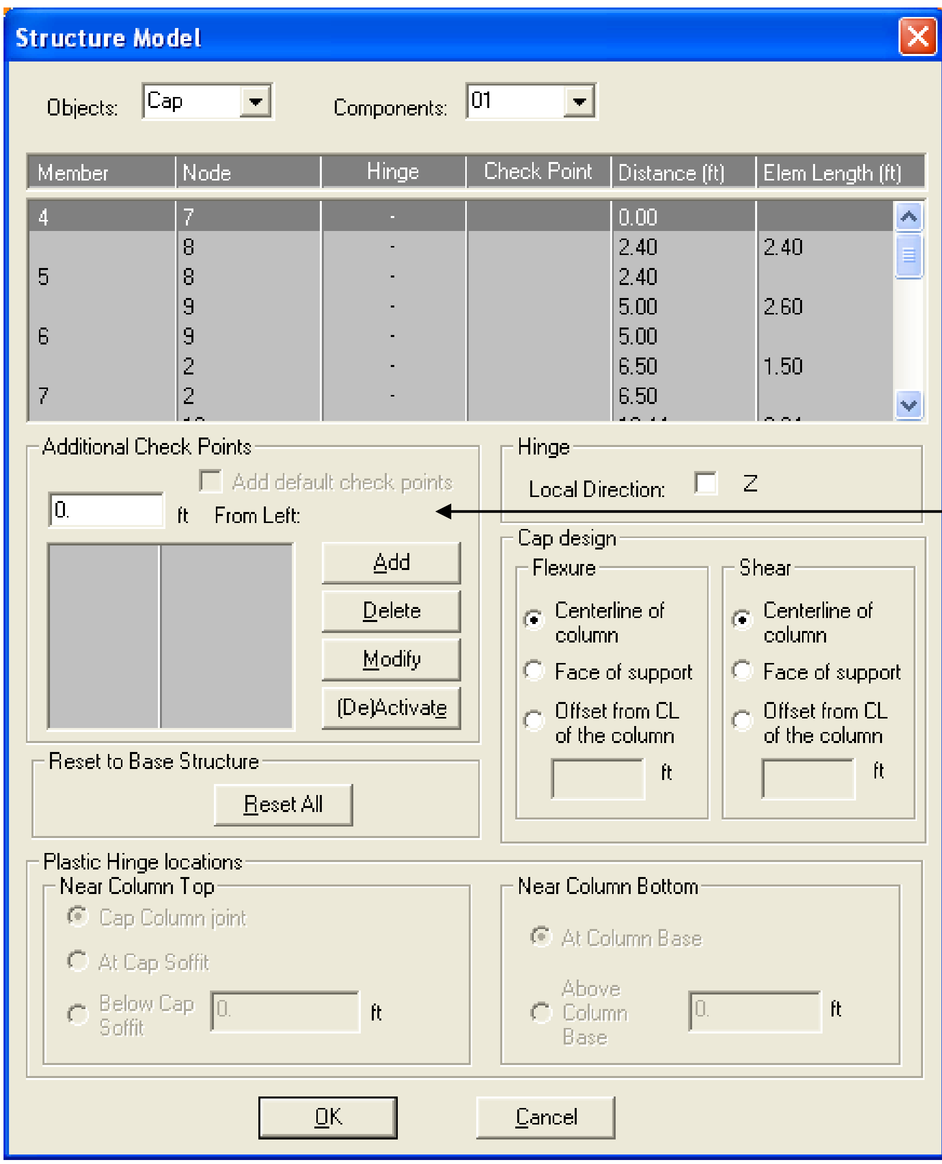




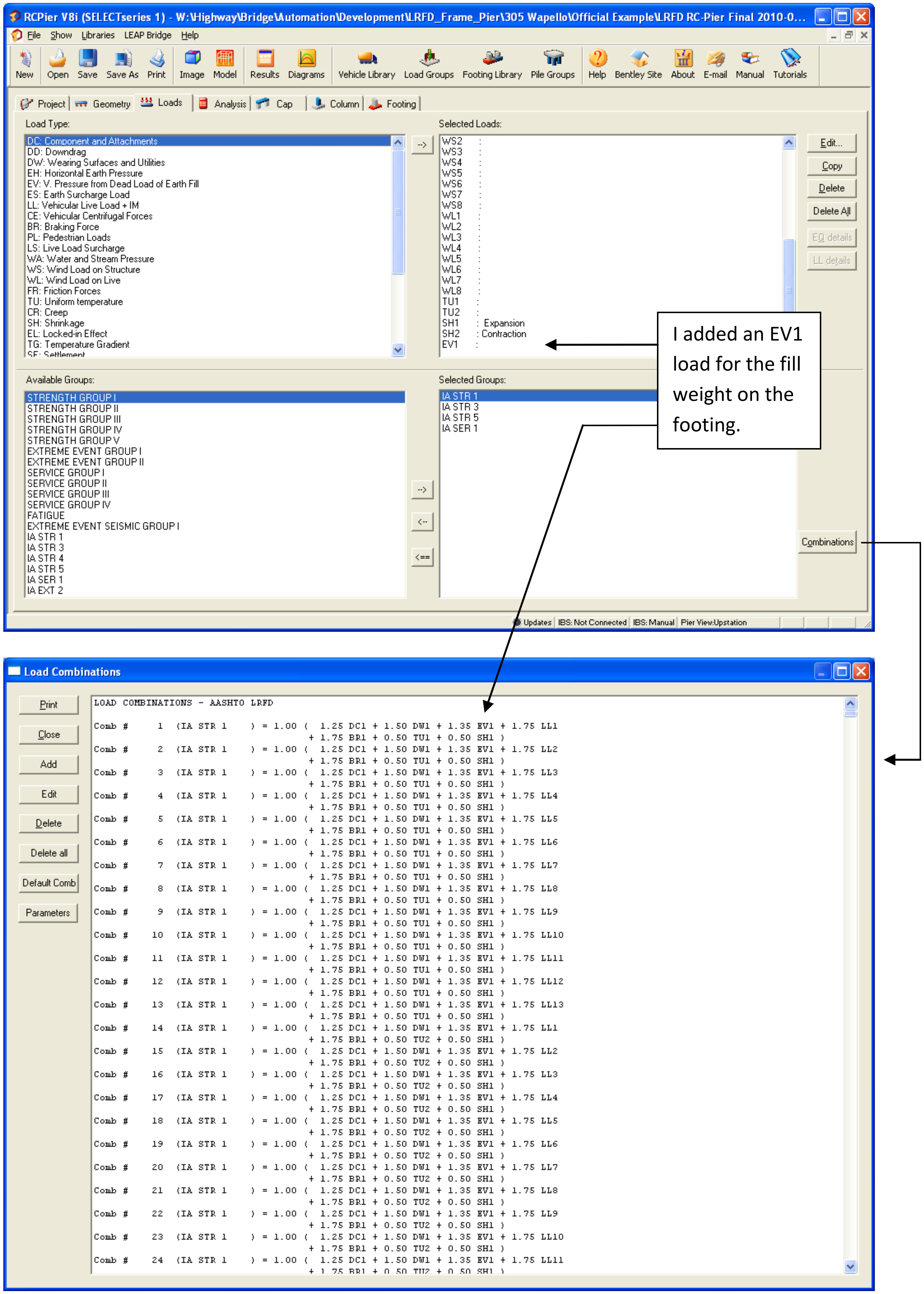
Notice that the column length has been increased by 3.5’ from 22’ to 25.5’.



Set the footing concrete density to a small value so that the self-weight calculated by RC-Pier is negligible.

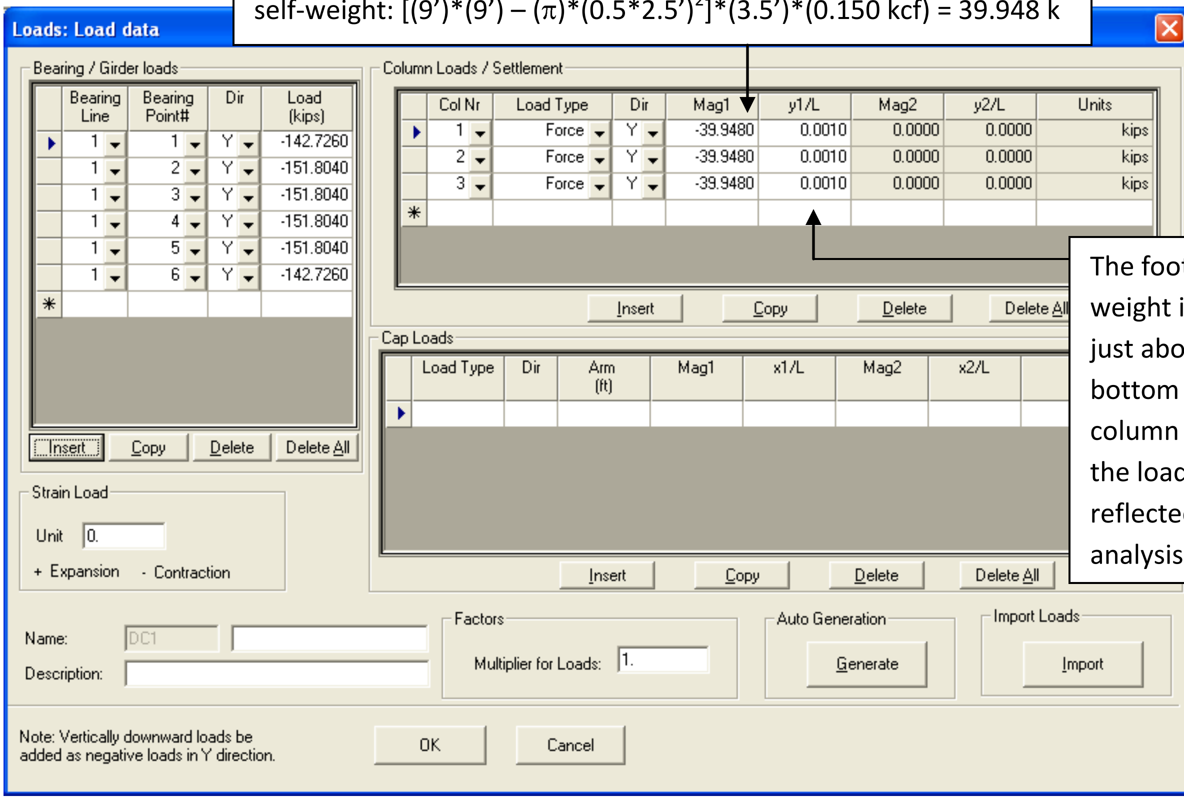


I removed all the additional pier cap check points. This helps speed up the program especially when printing analysis results to a text file.


RCPier V8i (SELECTseries 1) - W:\Highway\Bridge\Automation\Development\LRFD_Frame_Pier\305 Wapello\Official Example\LRFD RC-Pier Final 2010-0...
File Show Libraries LEAP Bridge Help
New Open Save Save As Print Image Model Results Diagrams Vehicle Library Load Groups Footing Library Pile Groups Help Bentley Site About E-mail Manual Tutorials
Project Geometry Loads Analysis Cap Column Footing
Load Type:
DC: Component and Attachments
DD: Downdrag
DW: Wearing Surfaces and Utilities
EH: Horizontal Earth Pressure
EV: V. Pressure from Dead Load of Earth Fill
ES: Earth Surcharge Load
LL: Vehicular Live Load + IM
CE: Vehicular Centrifugal Forces
BR: Braking Force
PL: Pedestrian Loads
LS: Live Load Surcharge
WA: Water and Stream Pressure
WS: Wind Load on Structure
WL: Wind Load on Live
FR: Friction Forces
TU: Uniform temperature
CR: Creep
SH: Shrinkage
EL: Locked-in Effect
TG: Temperature Gradient
Selected Loads:
WS2 :
WS3 :
WS4 :
WS5 :
WS6 :
WS7 :
WS8 :
WL1 :
WL2 :
WL3 :
WL4 :
WL5 :
WL6 :
WL7 :
WL8 :
TU1 :
TU2 :
SH1 : Expansion
SH2 : Contraction
EV1 :
Edit... Copy Delete Delete All EQ details LL details
I added an EV1 load for the fill weight on the footing.
Available Groups:
STRENGTH GROUP I
STRENGTH GROUP II
STRENGTH GROUP III
STRENGTH GROUP IV
STRENGTH GROUP V
EXTREME EVENT GROUP I
EXTREME EVENT GROUP II
SERVICE GROUP I
SERVICE GROUP II
SERVICE GROUP III
SERVICE GROUP IV
FATIGUE
EXTREME EVENT SEISMIC GROUP I
IA STR 1
IA STR 3
IA STR 4
IA STR 5
IA SER 1
IA EXT 2
Selected Groups:
IA STR 1
IA STR 3
IA STR 5
IA SER 1
Combinations
Updates IBS: Not Connected IBS: Manual Pier View:Upstation
Load Combinations
Print Close Add Edit Delete Delete all Default Comb Parameters
LOAD COMBINATIONS - AASHTO LRFD
Comb # 1 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL1 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 2 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL2 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 3 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL3 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 4 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL4 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 5 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL5 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 6 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL6 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 7 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL7 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 8 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL8 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 9 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL9 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 10 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL10 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 11 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL11 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 12 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL12 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 13 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL13 + 1.75 BR1 + 0.50 TU1 + 0.50 SH1)
Comb # 14 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL1 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 15 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL2 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 16 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL3 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 17 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL4 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 18 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL5 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 19 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL6 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 20 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL7 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 21 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL8 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 22 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL9 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 23 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL10 + 1.75 BR1 + 0.50 TU2 + 0.50 SH1)
Comb # 24 (IA STR 1) = 1.00 (1.25 DC1 + 1.50 DW1 + 1.35 EV1 + 1.75 LL11

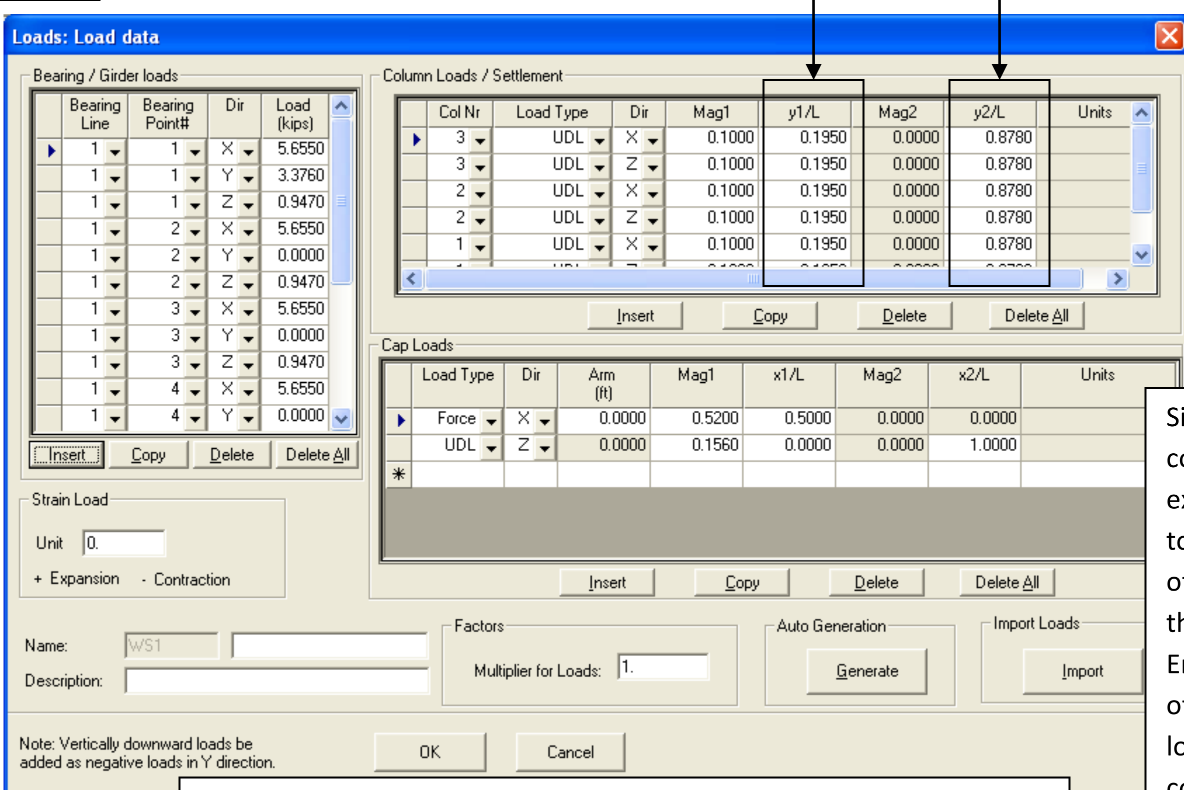
DC1

Added footing weight as a DC1 load, but also removed the weight due to the 3.5' column extension to counteract the addition of its self-weight: $[(9')*(9') – (\pi)*(0.5*2.5')^2]*(3.5')*(0.150\text{ kcf}) = 39.948\text{ k}$



The footing weight is placed just above the bottom of the column so that the load will be reflected in the analysis results.

WS1



Since the columns were extended 3.5' to the bottom of the footing, the Start and End locations of the wind loads on the columns should be modified.

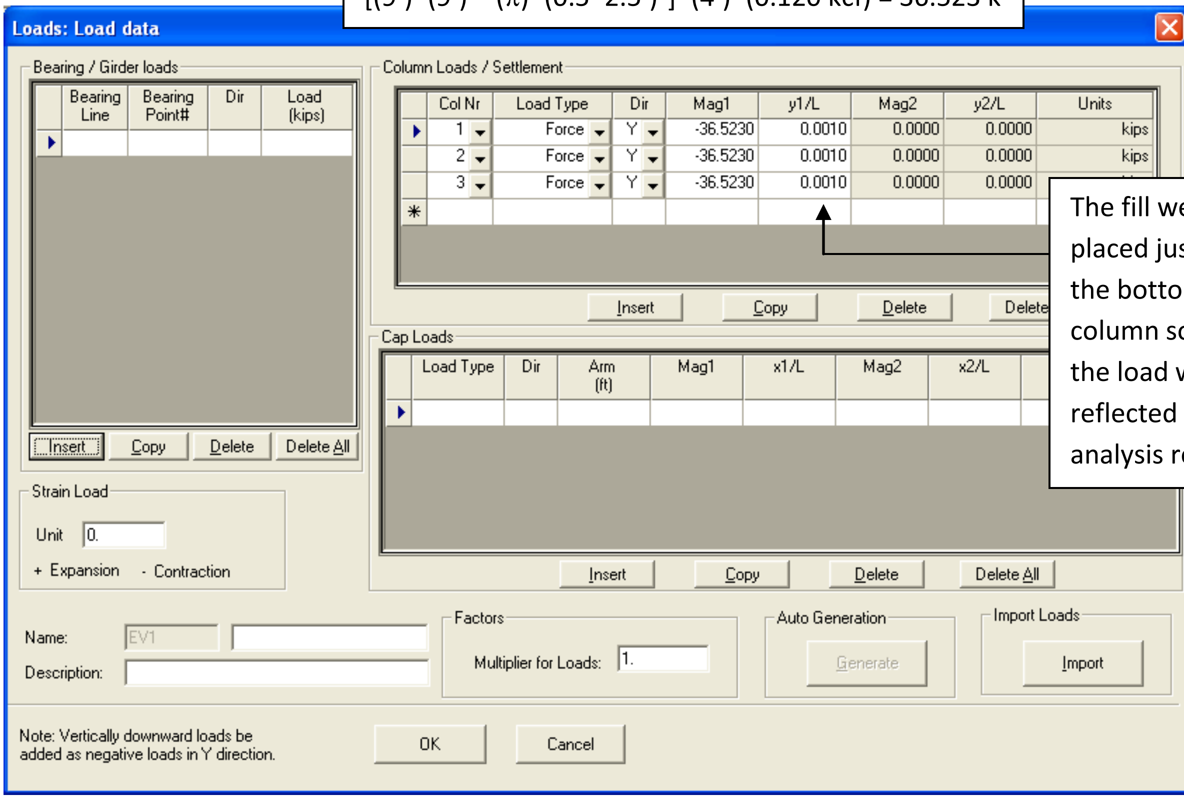
I didn't adjust the locations of the wind loads since the overall effect is not that significant. However, if I had the new values would be:

Start: $y1/L = 7.5'/24' = 0.313$

End: $y2/L = 21.5'/24' = 0.896$

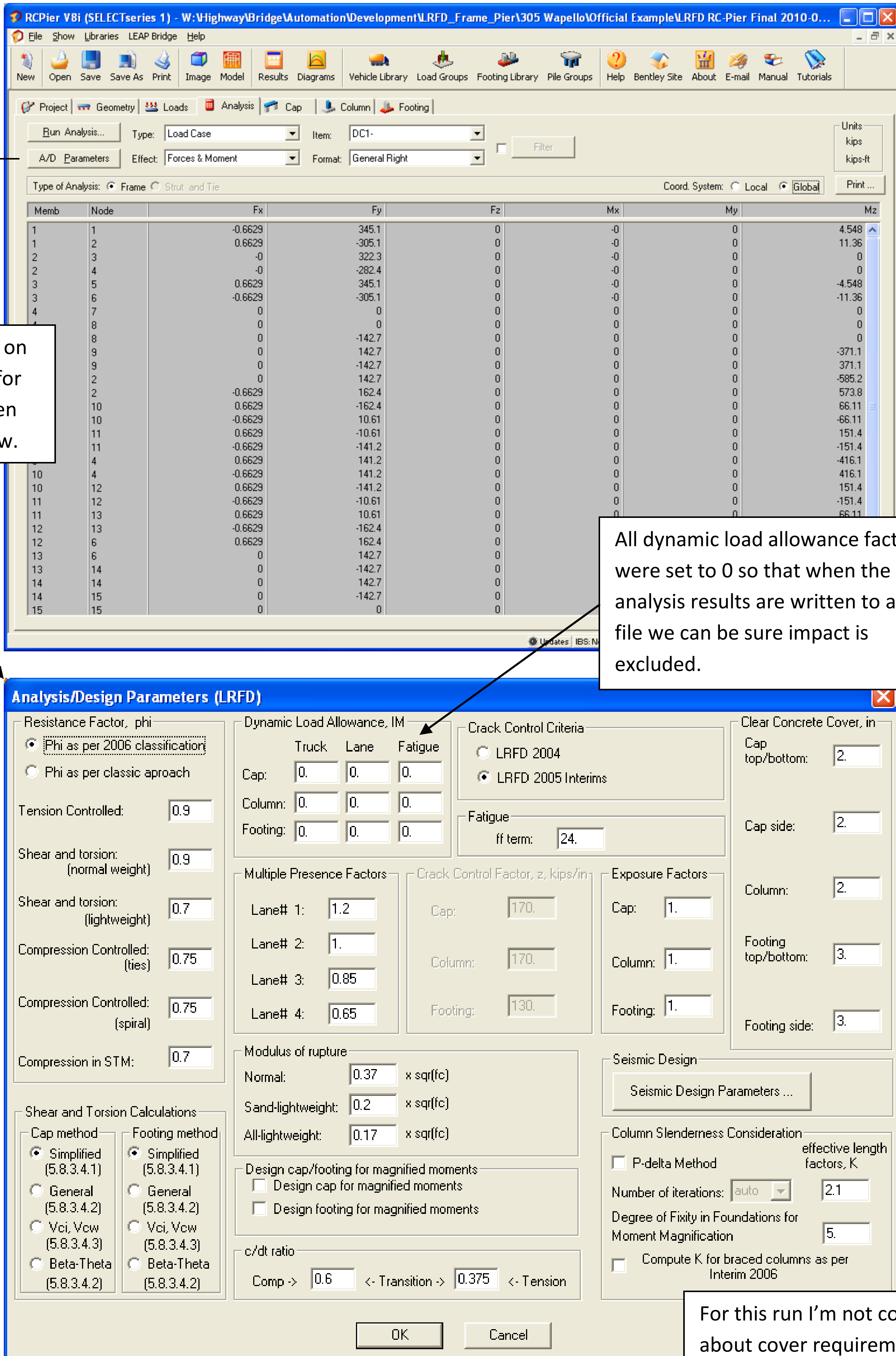
EV1

Adding fill weight on top of footings:
$[(9')*(9') - (\pi)*(0.5*2.5')^2]*(4')*(0.120\text{ kcf}) = 36.523\text{ k}$



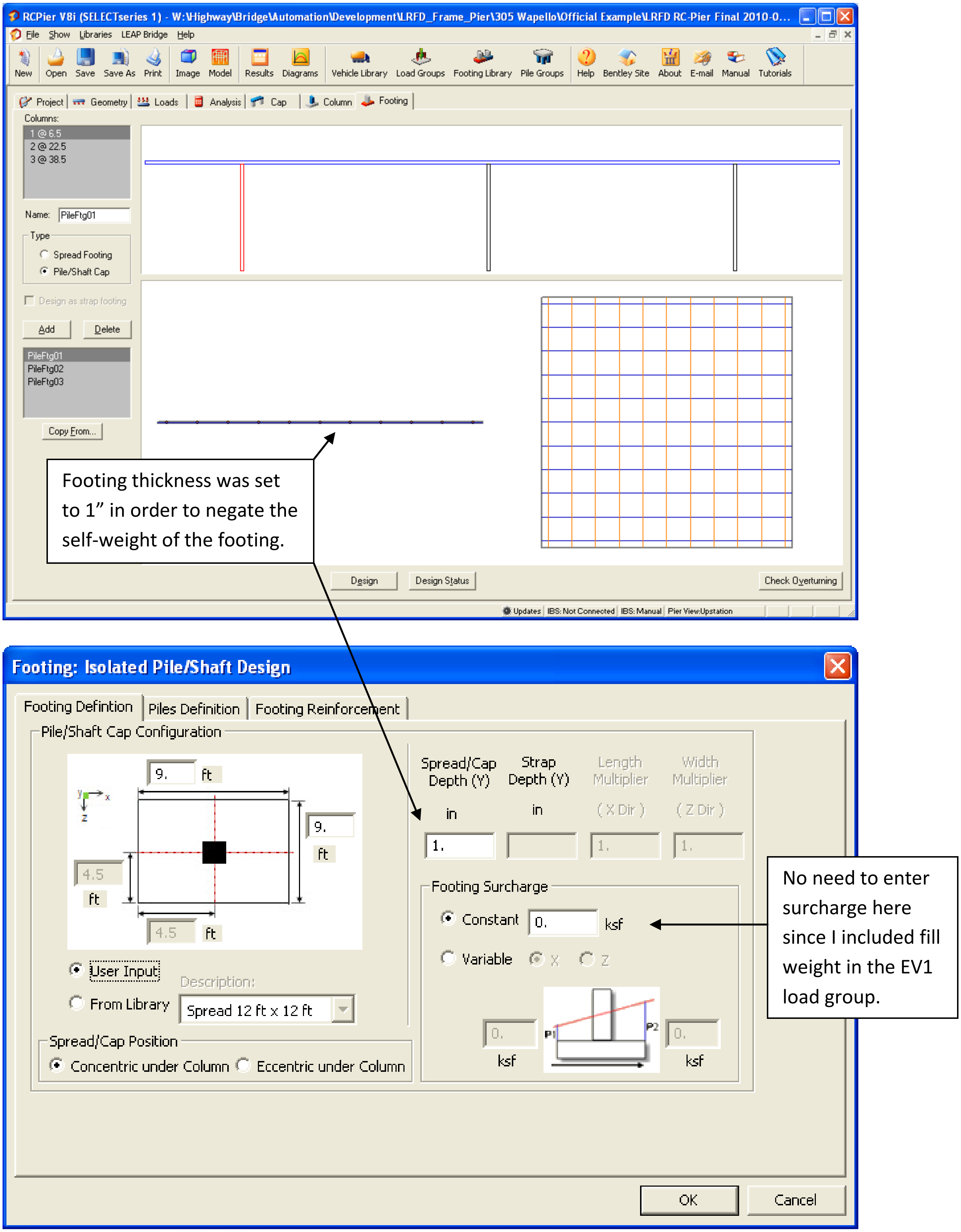
The fill weight is placed just above the bottom of the column so that the load will be reflected in the analysis results.

Click on this for screen below.

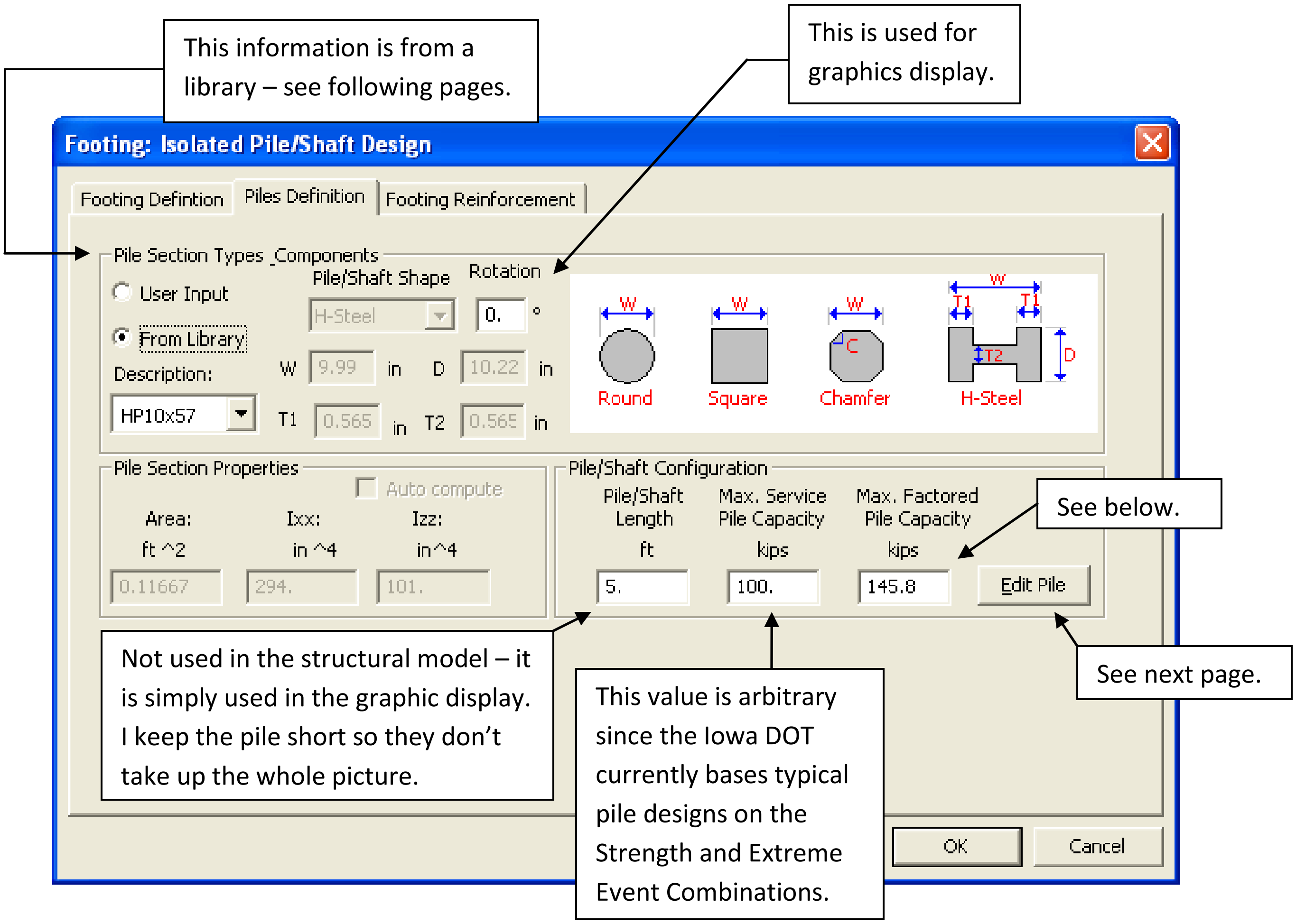


All dynamic load allowance factors were set to 0 so that when the analysis results are written to a file we can be sure impact is excluded.

For this run I'm not concerned about cover requirements since I am not designing footing reinforcement in RC-Pier.



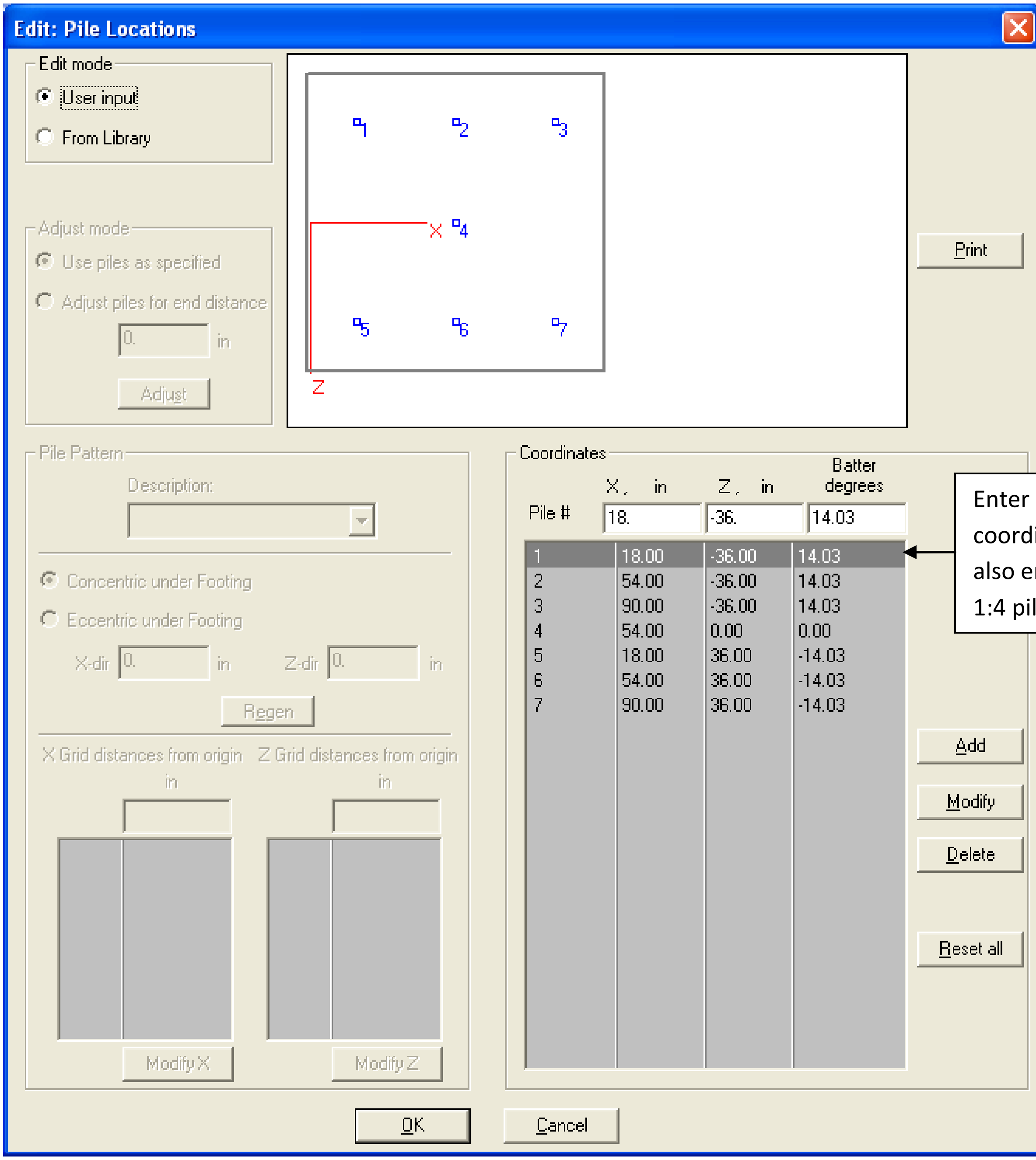
RCPier V8i (SELECTseries 1)
Columns:
1 @ 6.5
2 @ 22.5
3 @ 38.5
Name: PileFtg01
Type
Spread Footing
Pile/Shaft Cap
Design as strap footing
Add
Delete
PileFtg01
PileFtg02
PileFtg03
Copy From...
Design
Design Status
Check Overturning
Footing thickness was set to 1” in order to negate the self-weight of the footing.
Footing: Isolated Pile/Shaft Design
Footing Defintion
Piles Definition
Footing Reinforcement
Pile/Shaft Cap Configuration
Spread/Cap Depth (Y)
Strap Depth (Y)
Length Multiplier
Width Multiplier
in
in
(X Dir)
(Z Dir)
Footing Surcharge
Constant
Variable
ksf
User Input
Description:
From Library
Spread 12 ft x 12 ft
Spread/Cap Position
Concentric under Column
Eccentric under Column
OK
Cancel
No need to enter surcharge here since I included fill weight in the EV1 load group.



<u>HP10x57 Structural Resistance Level 1</u>

Factored Resistance = (6 ksi)*(0.1167 ft^2)*(144 in^2/ft^2)*(1.45) = 146.16 k

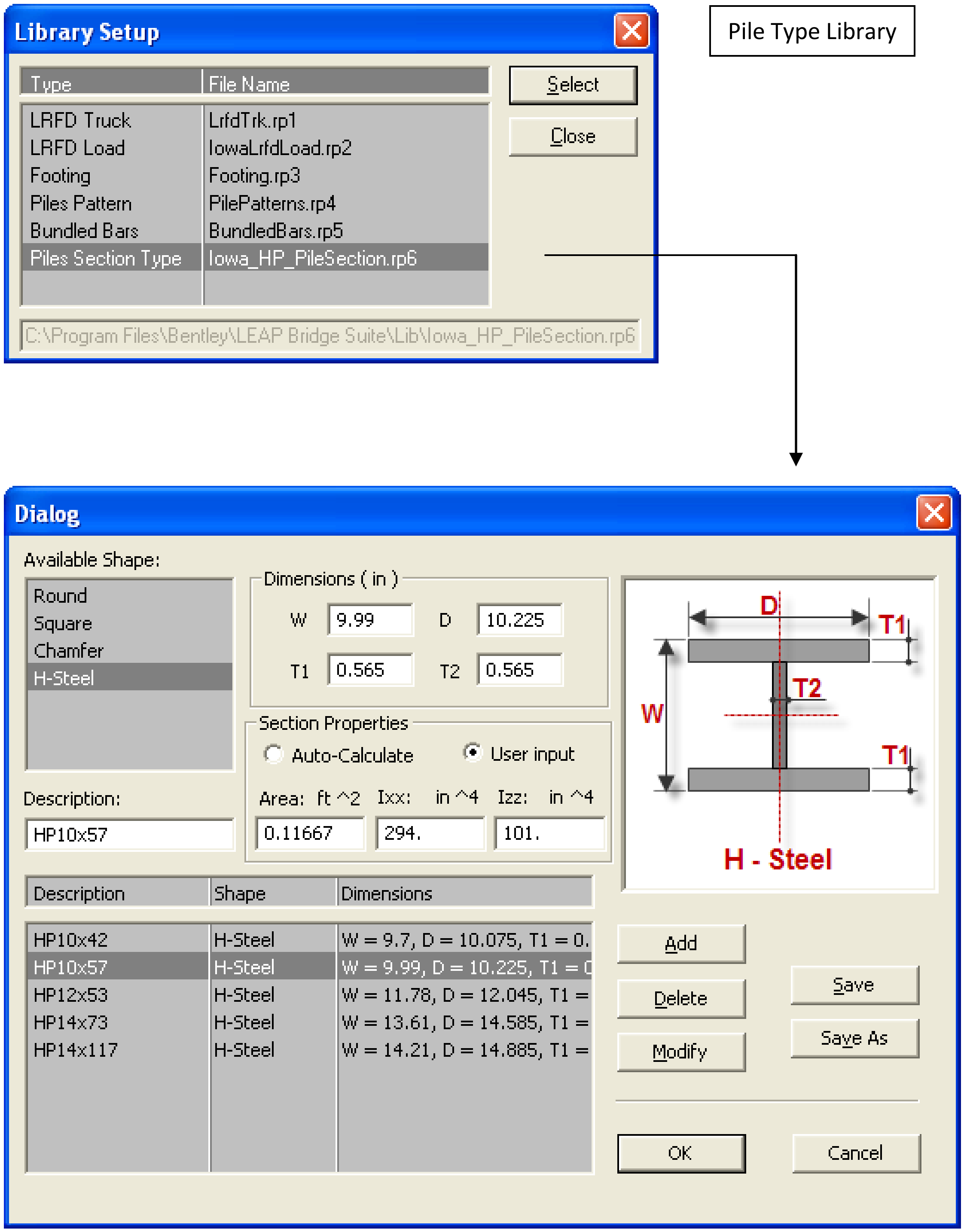
BDM Table 6.2.6.1-1 shows (0.6)*(243 k) = 145.8 k

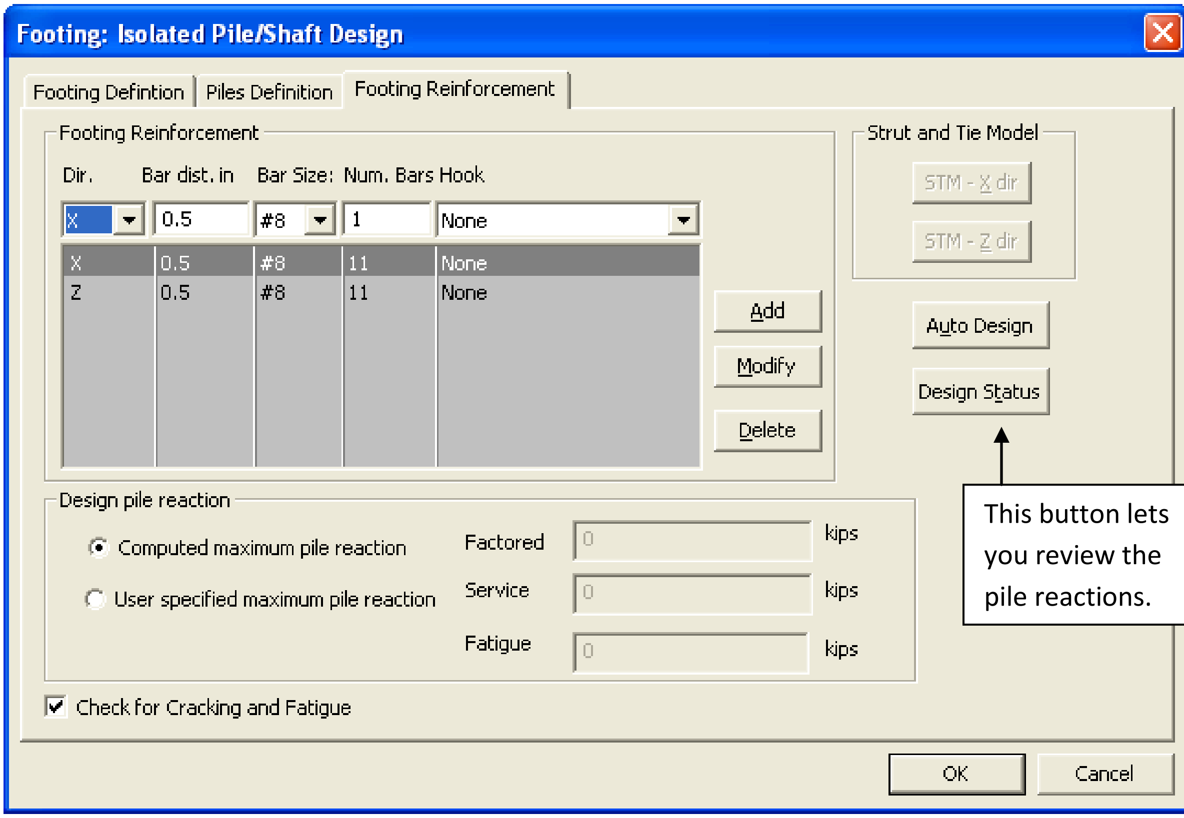


Enter pile coordinates. I also entered a 1:4 pile batter.

Aashto Lrfd 5.13.3.2 talks about piles being out of planned position by 6” or ¼ times the pile diameter and that the center of a group of piles may be 3” from its planned position. This provision does not need to be considered for typical pier designs. [Field construction errors are a separate issue from tolerances.]

According to the Bridge Design Manual a pile battered no more than 1:4 may be assumed to carry the same vertical load as a pile driven vertically; there need be no reduction for angle of the pile.


Library Setup
Pile Type Library
Type
File Name
Select
Close
LRFD Truck
LrfdTrk.rp1
LRFD Load
IowaLrfdLoad.rp2
Footing
Footing.rp3
Piles Pattern
PilePatterns.rp4
Bundled Bars
BundledBars.rp5
Piles Section Type
Iowa_HP_PileSection.rp6
C:\Program Files\Bentley\LEAP Bridge Suite\Lib\Iowa_HP_PileSection.rp6
Dialog
Available Shape:
Round
Square
Chamfer
H-Steel
Dimensions (in)
W
9.99
D
10.225
T1
0.565
T2
0.565
Section Properties
Auto-Calculate
User input
Area: ft ^2
Ixx: in ^4
Izz: in ^4
0.11667
294.
101.
Description:
HP10x57
Description
Shape
Dimensions
HP10x42
H-Steel
W = 9.7, D = 10.075, T1 = 0.
HP10x57
H-Steel
W = 9.99, D = 10.225, T1 = 0
HP12x53
H-Steel
W = 11.78, D = 12.045, T1 =
HP14x73
H-Steel
W = 13.61, D = 14.585, T1 =
HP14x117
H-Steel
W = 14.21, D = 14.885, T1 =
D
T1
T2
W
T1
H - Steel
Add
Delete
Modify
Save
Save As
OK
Cancel



This button lets you review the pile reactions.

There is not much point in entering footing reinforcement since my footing is only 1" thick and I won't be using RC-Pier for the design of the concrete footing. However, RC-Pier requires the entry in order to get the pile reactions.

ISOLATED FOOTING DESIGN

I've included some portions of RC-Pier's output for the footing.

ISOLATED FOOTING DESIGN
Code: AASHTO LRFD 2007 (with Interims)
Units: US
Pier View: Upstation.

GEOMETRY
Name : PileFtg01
Shape : Rectangular, Type : Pile/Shaft Cap
Bf(X) = 9.00 ft, Hf(Z) = 9.00 ft, Thickness(Y) = 1.00 in
Ag = 81.00 ft^2, Ix = 54.00 ft^2, Iz = 36.00 ft^2
Footing concentric.
Columns located on the footing:
Column No. 1 at x = 0.00 ft, Round D = 30.00 in
Surcharge = 0.00 ksf
Piles: H-Steel Size: W = 9.99 in, D = 10.23 in, T1 = 0.56 in, T2 = 0.56 in
Service Capacity: 100.00 kips Factored Capacity: 145.80 kips
Piles Section Properties: Area = 0.12 ft^2 Ix = 294.00 in^4 Iz = 101.00 in^4

DESIGN PARAMETERS	
f'c = 3500.00 psi	fy = 60000.00 psi
phi tens = 0.90	
phi comp = 0.75	phi shear = 0.90
Tens below = 0.375	Comp Above = 0.600
Ec = 3586.6 ksi	Es = 29000.0 ksi
Crack check as per 2005 Interims	
Crack control Exposure = 1.00	
Concrete Type : Normal Weight.	

Not interested in the Service capacity of the piles at this time.

Pile Reactions, Service

Pile	Loc(X) ft	Loc(Z) ft	X in	Z in	Batter degree	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips	Php kips	Plong kips	Plong-Php kips
1	-3.00	-1.50	18.00	-36.0	14	8308	1.000	-659.72	-109.97	159.98	113.69*	28.41	-0.66	27.75 #
						8019	1.000	-411.64	109.97	-197.05	36.28	9.06	0.66	9.72 #
2	0.00	-1.50	54.00	-36.0	14	8204	1.000	-639.33	-365.26	-10.32	111.62*	27.89	-2.09	25.80 #
						8125	1.000	-430.24	365.26	-15.23	41.17	10.29	2.09	12.38 #
3	3.00	-1.50	90.00	-36.0	14	8412	1.000	-631.04	-365.26	-105.22	119.21*	29.79	-2.09	27.70 #
						7917	1.000	-438.53	365.26	79.66	35.72	8.93	2.09	11.01 #
4	0.00	-4.50	54.00	0.0	0	7879	1.000	-671.41	196.58	216.79	95.92	0.00	1.06	1.06 #
						8424	1.000	-398.17	-196.58	-242.34	56.88	0.00	-1.06	-1.06
5	-3.00	-7.50	18.00	36.0	-14	7879	1.000	-671.41	196.58	216.79	124.90*	-31.21	1.06	-30.15
						8422	1.000	-399.95	-196.58	-253.85	25.06	-6.26	-1.06	-7.32
6	0.00	-7.50	54.00	36.0	-14	7866	1.000	-648.16	365.26	81.61	112.89*	-28.21	2.09	-26.12
						8411	1.000	-436.20	-365.26	-107.30	42.02	-10.50	-2.09	-12.59
7	3.00	-7.50	90.00	36.0	-14	8074	1.000	-639.87	365.26	-13.28	112.81*	-28.19	2.09	-26.10
						8203	1.000	-444.49	-365.26	-12.40	44.24	-11.05	-2.09	-13.14

These values appear because I entered pile batter.

Pile Reactions, Factored

Pile	Loc(X) ft	Loc(Z) ft	X in	Z in	Batter degree	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips	Php kips	Plong kips	Plong-Php kips
1	-3.00	-1.50	18.0	-36.0	14	1652	—	-842.94	-236.79	104.78	142.31	35.56	-1.38	34.18 #
						1078	—	-300.38	288.04	-241.31	6.80	1.70	1.80	3.50 #
2	0.00	-1.50	54.0	-36.0	14	57	—	-894.48	-253.16	-57.54	141.85	35.45	-1.42	34.03 #
						1086	—	-310.58	415.69	-156.16	21.27	5.32	2.52	7.84 #
3	3.00	-1.50	90.0	-36.0	14	83	—	-890.33	-253.16	-104.99	150.00*	37.48	-1.42	36.06 #
						1070	—	-314.72	415.69	-108.71	30.93	7.73	2.52	10.25 #
4	0.00	-4.50	54.0	0.0	0	18	—	-919.62	253.16	97.77	131.38	0.00	1.42	1.42 #
						1077	—	-295.38	111.98	-273.96	42.20	0.00	0.82	0.82 #
5	-3.00	-7.50	18.0	36.0	-14	18	—	-919.62	253.16	97.77	153.59*	-38.38	1.42	-36.96
						7590	—	-345.00	-158.07	-205.23	23.40	-5.85	-0.83	-6.68
6	0.00	-7.50	54.0	36.0	-14	18	—	-919.62	253.16	97.77	145.44	-36.34	1.42	-34.92
						7579	—	-389.24	-364.43	-28.70	35.36	-8.84	-2.10	-10.93
7	3.00	-7.50	90.0	36.0	-14	1418	—	-826.90	364.43	-30.66	140.93	-35.22	2.10	-33.12
						1064	—	-418.92	-415.69	137.49	25.30	-6.32	-2.52	-8.84

This is greater than the factored resistance of 145.80 k. So, I should modify my pile arrangement or add more piling. I won't do that at this time.

Footing Design : Notes

* Service Force in pile is greater than service pile capacity.
* Factored Force in pile is greater than factored pile capacity.
# = Pile needs to resist remaining lateral force.
Only max. force in piles is considered for design.
Pile coordinates X and Z are from the most left edge of the footing.
Plong= Lateral load in longitudinal direction at the top of pile, Kips.
Php= Available resisting horizontal component due to batter= batter * Vertical pile reaction, Kips.
Plong-Php= Remaining lateral force required to resist by pile.

Max. Pile Reaction Used in Design: (without selfweight and surcharge)

Factored pile reaction	153.59 kips
Service pile reaction	124.90 kips

Note that the maximum factored pile reaction is still 153.59 kips in this table. This is because the footing and fill weight were input as DC1 and EV1 loads rather than entered on the footing tab.

Reinforcement Schedule

Dir	Quantity	Size	Bar dist. in	As total in^2	Spacing in	Hook
X	11	#8	0.50	8.69	10.10	None
Z	11	#8	0.50	8.69	10.10	None

I've included the rest of the footing output, but I'm not really interested in the concrete footing design portion since I entered a 1" thick footing. Footing design is typically done using a spreadsheet.

Flexure

Dir ft	Loc	d in	Mmax kft	Comb	CL	Asb_req in^2	Asb_prv in^2	Asb_eff in^2	Ast_req in^2	Ast_prv in^2	Ast_eff in^2
X	-1.11	-2.00	581.2	18	T	0.00	0.00	0.00	0.08	8.69	8.69
X	1.11	-2.00	581.2	18	T	0.00	0.00	0.00	0.08	8.69	8.69
Z	-1.11	-2.00	871.9	18	T	0.00	0.00	0.00	0.08	8.69	8.69
Z	1.11	-2.00	871.9	18	T	0.00	0.00	0.00	0.08	8.69	8.69

Flexure Note

CL: Section classification as per LRFD 2006 interims for provided reinforcement.
C = Compression controlled, I = In-Transition, T = Tension controlled.
Required reinforcement is based on phi for tension controlled sections..

Cracking check as per AASHTO LRFD 2007 with Interims (2005)

Cracking/Fatigue

Dir	Loc ft	d in	Cracking Mmax kft	Cracking Comb	Cracking fs ksi	Cracking Srq in	Cracking Spr in	Fatigue Mmax kft	Fatigue Comb	Fatigue fs ksi	Fatigue ratio fs
X	-1.11	-2.00	472.7	7879	0.00	108.0	0.0	0.0	0	0.00	0.00
X	1.11	-2.00	472.7	7879	0.00	108.0	0.0	0.0	0	0.00	0.00
Z	-1.11	-2.00	709.0	7879	0.00	108.0	0.0	0.0	0	0.00	0.00
Z	1.11	-2.00	709.0	7879	0.00	108.0	0.0	0.0	0	0.00	0.00

One Way Shear (Simplified Method)

Col	Dir	Dist ft	Comb	dv in	Vu kips	phi*Vc kips
1	X	-1.17	18	0.72	307.2 *	8.3
	X	1.17	18	0.72	307.2 *	8.3
	Z	-1.17	18	0.72	460.8 *	8.3
	Z	1.17	18	0.72	460.8 *	8.3

One Way Shear Note

* Shear resistance is less than applied shear force. You may increase the footing depth or provide stirrups.

Two Way Shear

#	Bo ft	Ao ft^2	Comb	Avg. dv in	Vu kips	phi*Vc kips
Columns						
1	8.04	5.15	18	0.72	921.5*	14.7
Piles - max						
1	3.57	0.80	18	0.72	153.6*	6.5
Piles - min						
1	3.57	0.80	18	0.72	153.6*	6.5

Two Way Shear Note

* Shear resistance is less than applied punching force.

TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.

Spreadsheet for Footing Design

Application to Design Pile Footings based on RC-Pier Output

Developed on 10/16/2006
Last Updated on 8/30/2010

DOT refers to the Iowa Department of Transportation.
OBS refers to the Iowa DOT Office of Bridges and Structures

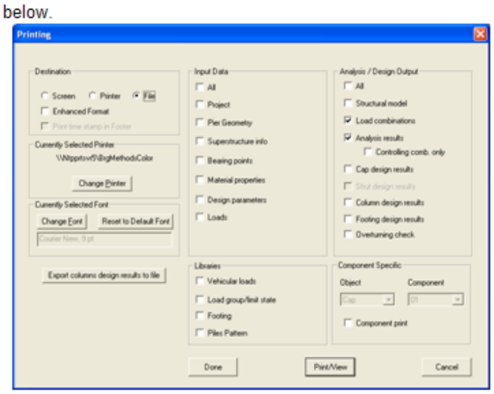
The OBS will only support those persons using this software in connection with Iowa DOT related business.

Please report any spreadsheet errors to the Iowa DOT OBS.

This spreadsheet was developed to aid the design of typical Iowa DOT piers. The Iowa DOT Bridge Design Manual (BDM) should be consulted for the most up-to-date policies.

Description:
This application is designed to read RC-Pier's analysis results and to use them for pile footing design.

Steps:
1.) Run RC-Pier.
2.) Print "Load combinations" and "Analysis results" from RC-Pier to a text file. See figure below.
[Be sure to set the dynamic load allowance, IM, to 0 for the cap, column, and footing on the "A/D Parameters" screen before running the analysis. This will ensure impact is excluded from the results that are sent to the text file. You should only include Strength Combinations in the text file.] It may take awhile for RC-Pier to write all the information to a file.
3.) Fill out Input tab in this spreadsheet.
4.) Click "Load Input" button on the Input tab and select the RC-Pier text file just created. It may take awhile for this spreadsheet to read in the data from the text file.
5.) Pile footings can be designed on the "PileFtgDesign" tab.



Notes:
1.) Loads peculiar to the footing such as footing weight, soil weight, and bouyancy on the footing and soil should be entered in RC-Pier at the base of the column on the "Loads" tab so that they will be included in the analysis results that are written to the text file. [Actually any point loads at the base of the column should be input just above the base in RC-Pier. If they are not, then those loads are excluded from analysis results text file.]
2.) The user must include "Load combinations" and "Analysis results" in the RC-Pier text file.
3.) Only Strength combinations should be included in the RC-Pier text file. Fatigue, Service, and Extreme Event combinations should be excluded.
4.) To save time it is recommended that the user turn of all extra analysis points set up in RC-Pier (e.g. any additional points on the cap) before saving data to a file.
5.) Footing pile arrangements must be symmetrical.
6.) The tabs labeled "Calc1", "Calc2", "Col1Bot" to "Col6Bot", and "Keywords" are used to store data and should not be altered.

The steps to export loads from RC-Pier and to import them into this sheet are not shown.

Pile Footing Design

Aashto Lrfd 5.13.3

Footing Length (X direction)	9.000	feet
Footing Width (Z direction)	9.000	feet
Footing Depth (Y direction)	3.500	feet

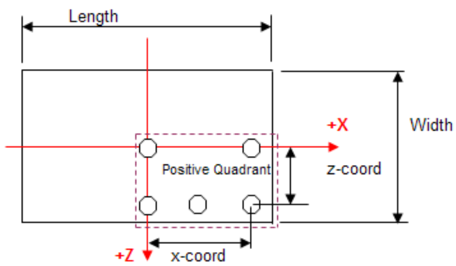
Column Width or Diameter (X direction)	2.500	feet
Column Depth (Z direction)	0.000	feet

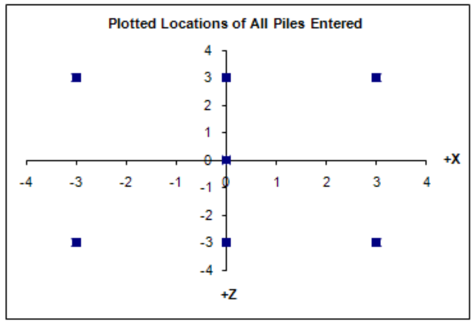
Enter column depth of 0 for round columns.

Pile Diameter, dp	10	inches
28 Day Concrete Strength, f'c in psi	3500	psi
Footing Number to Consider	1	

Only the pile locations in the positive quadrant should be entered since the pile footing is assumed to be symmetrical. The user should include any piles located on the +X and + Z axes and the pile at the center of the footing if present.

Aashto Lrfd 5.13.3.2 makes provision for the tolerance of actual pile location. Office policy is to ignore this provision in footing design.





Pile Number	Positive x-coord (feet)	Positive z-coord (feet)
1	3	3
2	0	3
3	0	0
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
21		
22		
23		
24		
25		
26		
27		
28		
29		
30		

Total Number of Footing Piles	7	
Sx, Pile Section Modulus	18.000	ft
Sz, Pile Section Modulus	12.000	ft

Determine Pile Loads

If any input above the line or on previous tabs is changed, then you will need to re-click on "Determine Pile Loads".

Pile Loads	Load (kips)	Controlling Combo
Fact'd Pile Resist, $R_r = \phi R_n$	145.800	
Max Fact'd Pile Load, Pu_m	153.586	18
Max Fact'd Avg Pile Load, Pu_a	131.374	18
Min Fact'd Pile Load	6.800	1078

"-" means uplift

Loads match RC-Pier.

Pile Capacity Results: Is Pu_m <= Rr ?

NO. Pile is NO GOOD.

Pu_m should be less than or equal to Rr = ϕRn.

User Loads for Footing Design			Used for:
Factored Pile Resistance, $R_r = \phi R_n$	145.800	kips	One-way (beam) shear
Maximum Factored Pile Load, Pu_m	153.586	kips	Flexure reinforcement
Maximum Factored Average Pile Load, Pu_a	131.374	kips	Two-way (punching) shear

Note: The user can, according to office policy, deduct the factored bouyant weight of the footing from the pile loads for flexure and shear design. Be sure to deduct only the portion of the load going to one pile. Soil load may not be deducted.

Flexural Capacity Check Aashto Lrfd 5.7.3.2 and 5.13.3.4

Design Flexural R/I Parallel to X-axis (Mz moments)		
Clearance to Flexural R/I from Bottom of Footing	13.000	in
Number of Bars Required for R/I parallel to X-axis	7	
Flexural R/I Bar Size for bars parallel to X-axis	9	
Bar Diameter for # 9	1.128	in
Bar Area for # 9	1.000	in^2
Total Bar Area	7.000	in^2
Effective Depth for bars parallel to X-axis, de_z or ds_x	28.436	in

Note: de_x corresponds with design for Mz

Design Flexural R/I Parallel to Z-axis (Mx moments)		
Clearance to Flexural R/I from Bottom of Footing	14.128	in
Number of Bars Required for R/I parallel to Z-axis	10	
Flexural R/I Bar Size for bars parallel to Z-axis	9	
Bar Diameter for # 9	1.128	in
Bar Area for # 9	1.000	in^2
Total Bar Area	10.000	in^2
Effective Depth for bars parallel to Z-axis, de_z or ds_z	27.308	in

Note: de_z corresponds with design for Mx.

Flexure Phi Factor, ϕ	0.900	
Factored Applied Moment, Mu_z	581.236	k*ft
Depth of Equivalent Stress Block, a	1.307	in
Factored Flexural Resistance, $Mr_z = \phi Mn_z$	875.146	k*ft
Is Mu_z <= Mr_z ?	**Yes**	

Flexure Phi Factor, ϕ	0.900	
Factored Applied Moment, Mu_x	871.854	k*ft
Depth of Equivalent Stress Block, a	1.867	in
Factored Flexural Resistance, $Mr_x = \phi Mn_x$	1186.843	k*ft
Is Mu_x <= Mr_x ?	**Yes**	

Minimum Reinforcement Check Aashto Lrfd 5.7.3.3.2 and 5.4.2.6

Enter: 1 if de + 2" is to be used to calculate Mcr 2 if the ftg depth is to be used to calculate Mcr	1	If 1, then de + 2" is used to calculate Mcr; otherwise, the footing depth is used.
Modulus of Rupture, fr	0.692	ksi

Section Modulus of Concrete Footing, Sz	9.649	ft^3
120% of the Cracking Moment, $1.2*Mcr_z$	1154.206	k*ft
Is $1.2*Mcr_z$ <= Mr_z ?	**No**	

Section Modulus of Concrete Footing, Sx	8.947	ft^3
120% of the Cracking Moment, $1.2*Mcr_x$	1070.239	k*ft
Is $1.2*Mcr_x$ <= Mr_x ?	**Yes**	

---- OR ----

As required	4.612	in^2
Is As prov'd >= 1.33*As req'd ?	**Yes**	

---- OR ----

As required	7.276	in^2
Is As prov'd >= 1.33*As req'd ?	**Yes**	

Maximum Reinforcement Check Aashto Lrfd 5.5.4.2, 5.7.2.1, and 5.7.3.3

Stress Block Factor, β1	0.850	
Location of Neutral Axis, c	1.538	in
Net Tensile Strain in the Extreme Tension Steel, ε_s	0.052	in/in
Is Section Tension Controlled? ε_s >= 0.005	**Yes**	
Is Section Compression Controlled? ε_s <= 0.002	**No**	
Is Section in Transition? 0.005 > ε_s > 0.002	**No**	
Flexure Phi Factor, ϕ, for Design	0.900	

Stress Block Factor, β1	0.850	
Location of Neutral Axis, c	2.197	in
Net Tensile Strain in the Extreme Tension Steel, ε_s	0.034	in/in
Is Section Tension Controlled? ε_s >= 0.005	**Yes**	
Is Section Compression Controlled? ε_s <= 0.002	**No**	
Is Section in Transition? 0.005 > ε_s > 0.002	**No**	
Flexure Phi Factor, ϕ, for Design	0.900	

NOTE: If section is in Transition, then the user must adjust the Flexural Phi Factor, ϕ, in cell G73 or O73.
If section is Compression Controlled, then do not use this spreadsheet. The user must do a strain compatabilty analysis.

Is Flexural R/I Adequate?	**YES, Flexural R/I is Adequate.**

Is Flexural R/I Adequate?	**YES, Flexural R/I is Adequate.**

Crack Control: Flexure R/I Aashto Lrfd 5.7.3.4.

The requirements of Aashto Lrfd 5.13.3.5 should be included. Spacing should also comply with Aashto Lrfd 5.10.3.1 and 5.10.3.2.
If uplift is present; then, as a minimum, add #5 bars at 12" to the top of the footing in both directions.

Enter: 1 if de + 2" is to be used to calculate cover 2 if the ftg depth is to be used to calculate cover	1	If 1, then de + 2" is to be used to calculate cover; otherwise, the footing depth is used.

Exposure Factor, γ_e	1.000		Exposure Factor, γ_e	1.000	
Concrete Cover Thickness to R/I Center, dc	2.000	in	Concrete Cover Thickness to R/I Center, dc	2.000	in
βs	1.100		βs	1.105	
Maximum Service Pile Load, Ps_m	124.900	kips	Maximum Service Pile Load, Ps_m	124.900	kips
Positive Service Mz	472.676	k*ft	Positive Service Mz	709.013	k*ft

See Aashto Lrfd 5.4.2.4 and 5.7.1 for Ec and n

Reinforcment Ratio, ρ	0.00228		Reinforcment Ratio, ρ	0.00339	
Concrete Modulus of Elasticity, Ec	3586.616	ksi	Concrete Modulus of Elasticity, Ec	3586.616	ksi
Modular Ratio, n	8.000		Modular Ratio, n	8.000	
Factor for Distance to Neutral Axis, k	0.174		Factor for Distance to Neutral Axis, k	0.207	
Reinforcement Stress at Service Level	30.246	ksi	Reinforcement Stress at Service Level	33.470	ksi
Max. Spacing of Bot Layer of Pos. Flex. R/I, s	**17.031**	in	**Max. Spacing of Bot Layer of Pos. Flex. R/I, s**	**14.933**	in

Crack Control: Skin R/I Aashto Lrfd 5.7.3.4

Is Skin R/I Required ? (Is de = ds > 3.00'?)	**No**		**Is Skin R/I Required ? (Is de = ds > 3.00'?)**	**No**	
Area of Skin R/I Required per Face, Ask	**0.000**	in^2 per ft	**Area of Skin R/I Required per Face, Ask**	**0.000**	in^2 per ft
Max Spacing of Skin R/I Required	**4.739**	in	**Max Spacing of Skin R/I Required**	**4.551**	in

Shrinkage and Temp. R/I and Structural Mass Concrete Aashto Lrfd 5.10.8

Area of Skin R/I Required per Face, Ask	**0.328**	in^2 per ft	**Area of Skin R/I Required per Face, Ask**	**0.328**	in^2 per ft
Max Spacing of Skin R/I Required	**12.000**	in	**Max Spacing of Skin R/I Required**	**12.000**	in

Fatigue in R/I Aashto Lrfd 5.5.3

Office policy is to neglect checking fatigue.

Shear Capacity Check Aashto Lrfd 5.8.1.4, 5.13.3.6 and 5.8.3

Enter 1 to check dv = 0.72*h. Enter 2 to exclude it.	2		See Aashto Lrfd 5.8.2.9.
Calculated Effective Shear Depth, dv	26.374	in	
User Entry for Effective Shear Depth, dv	26.374	in	

One Way Shear or Beam Shear Parallel to Z-axis			One Way Shear or Beam Shear Parallel to X-axis		
Distance from Column Center to Critical Section	3.306	ft	Distance from Column Center to Critical Section	3.306	ft
Point of 0 Shear to Equivalent Column Face	2.309	ft	Point of 0 Shear to Equivalent Column Face	2.309	ft
Distance of 3*dv	6.594	ft	Distance of 3*dv	6.594	ft
Is Point of 0 Shear to Equivalent Column Face < 3*dv?	YES		Is Point of 0 Shear to Equivalent Column Face < 3*dv?	YES	
If the above is YES then Aashto Lrfd 5.8.3.4.1 may be applied with β = 2.00.			If the above is YES then Aashto Lrfd 5.8.3.4.1 may be applied with β = 2.00.		
Factor for Tens Trans Diagonally Crack'd Concr, β	2.000		Factor for Tens Trans Diagonally Crack'd Concr, β	2.000	
Aashto Lrfd 5.8.3.3 and 5.8.3.4			Aashto Lrfd 5.8.3.3 and 5.8.3.4		
Factored Applied Shear, Vu_x	38.859	k	Factored Applied Shear, Vu_z	58.288	k
Factored Shear Resistance, $Vr_x = \phi Vn_x = \phi Vc_x$	303.105	k	Factored Shear Resistance, $Vr_z = \phi Vn_z = \phi Vc_z$	303.105	k
Is Beam Shear OK? Vu_x <= Vr_x	**YES.**		**Is Beam Shear OK? Vu_z <= Vr_z**	**YES.**	

Two Way Shear or Punching Shear			
Distance from Column Center to Critical Section	2.207	ft	Parallel to Z-axis
Distance from Column Center to Critical Section	2.207	ft	Parallel to X-axis
Perimeter of the Critical Section, bo	17.654	ft	
Ratio of Long Side to Short Side, βc	1.000		
Factored Applied Shear, Vu_x	788.244	k	
Factored Shear Resistance, $Vr_x = \phi Vn_x = \phi Vc_x$	1185.326	k	
Is Punching Shear OK? Vu_x <= Vr_x	**YES.**		