

# **EXAMPLE NO.1: PRESTRESSED CONCRETE GIRDER BRIDGE DESIGN**

**Prepared for:**



**Prepared by:**



**Date: July 15, 2011**

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## 1. INTRODUCTION

This example illustrates New Mexico Department of Transportation (NMDOT) design procedures for a three-span prestressed concrete girder bridge. Site location is assumed to be near Socorro, New Mexico, with the bridge crossing a waterway on a normal (perpendicular) alignment. The bridge consists of 43.75 ft., 88.0 ft. and 43.75 ft. spans, with a 50 ft. wide bridge. The figures on pages 5 and 6 show the elevation and typical section for the bridge.

The superstructure is supported by AASHTO Type III girders, which are continuous for live load. The substructure consists of three-column piers and abutment bents supported directly by drilled shafts. The abutment is of the semi-integral (floating) type.

The following design steps are included in this example:

- Concrete deck design
- AASHTO Type III girder design
- Bearing pad design
- Pier and abutment cap design
- Pier column design
- Drilled shaft design
- Seismic design

Load and Resistance Factor Design (LRFD) methods are used throughout, except where a suitable LRFD procedure does not exist. Note that acceptable design methods are not limited to those shown here. Other methods that comply with NMDOT requirements are also acceptable.

It is assumed that those using this example are familiar with general bridge design procedures and the *AASHTO LRFD Bridge Design Specifications*, hereinafter referred to as *LRFD Specifications*. References in parentheses refer to the applicable section or equation from the above specifications.

Reference to and use of proprietary computer programs in this example does not constitute an endorsement by the NMDOT.

The NMDOT makes no guarantee regarding the accuracy of example calculations. The reader is cautioned to verify all calculations before duplicating.

Comments or suggestions – send to:

New Mexico Department of Transportation  
Bridge Design Bureau, Room 214  
P.O. Box 1149  
Santa Fe, NM 87504

## 2. DESIGN DATA

Specifications:	<i>AASHTO LRFD Bridge Design Specifications</i> , Fifth Edition, 2010 * <i>AASHTO Guide Specifications for LRFD Seismic Bridge Design</i> , First Edition, 2009
Design Method:	Load and Resistance Factor Design
Design Live Load:	The Design Live Load (HL93) consists of a design truck or design tandem and a design lane load, and a NM permit design live load P327-13
Dead Loads:	150 pcf is assumed for concrete unit weight. 15 psf is assumed for steel stay-in-place forms. 30 psf is assumed for a future wearing surface.
Seismic Design:	Seismic Zone: Socorro, NM Consider bridge to be “Essential”**
Waterway Data:	100 -Year Flood: $V_{100} = 10$ ft./s (average velocity), High water elevation is 1 ft. below the bottom of the pier cap  500 -Year Flood: $V_{500} = 12$ ft./s (average velocity), High water elevation is at the bottom of the pier cap
Bridge Barrier:	NMDOT 42 in. Single Slope Bridge Barrier Railing, Volume = 3.83 ft. <sup>3</sup> / ft.
Construction Method:	Unshored Construction
Prestressed Girder Concrete:	Initial Compressive Strength: $f'_{ci} = 7.0$ ksi Final Compressive Strength: $f'_c = 9.5$ ksi $E_c = 5908.98$ ksi
Prestressing Steel:	½ in. Dia., 270 ksi, Low Relaxation, Seven-Wire Strand $f_s = 240.0$ ksi $E_p = 29,000$ ksi
Superstructure Concrete:	NMDOT Superstructure Concrete $f'_c = 4.0$ ksi $E_c = 3834.25$ ksi

Substructure Concrete:	NMDOT Substructure Concrete Class A $f'_c = 3.0$ ksi $E_c = 3320.56$ ksi
Reinforcing Steel:	Grade 60 $F_y = 60.0$ ksi $E_s = 29,000$ ksi
Drilled Shafts:	NMDOT Class G Concrete $f'_c = 3.0$ ksi $E_c = 3320.56$ ksi

\* Note: The LEAP Bridge computer program, version 9.00.03.02, used for this example uses the Fourth Edition of the *AASHTO LRFD Bridge Design Specifications*.

\*\* Note: Critical/essential bridges are not specifically addressed in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. NMDOT does not have any additional requirements beyond these specifications for critical and essential bridges.

### 3. GENERAL

An outline for basic steps for concrete bridge design is given in Appendix A5 of the *LRFD Specifications*. This design example tries to follow this outline as closely as is relevant.

#### Design Philosophy (1.3.1)

Bridges shall be designed for specified limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics, as specified in Article 2.5.

Regardless of the type of analysis used, the following equation shall be satisfied for all specified force effects and combinations thereof.

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

#### Limit State (1.3.2)

Each component and connection shall satisfy the above equation for each limit state, unless otherwise specified. All limit states shall be considered of equal importance.

The Limit States are:

- Service Limit State

- Fatigue and Fracture Limit State
- Strength Limit State
- Extreme Event Limit State

#### **4. SUPERSTRUCTURE DESIGN**

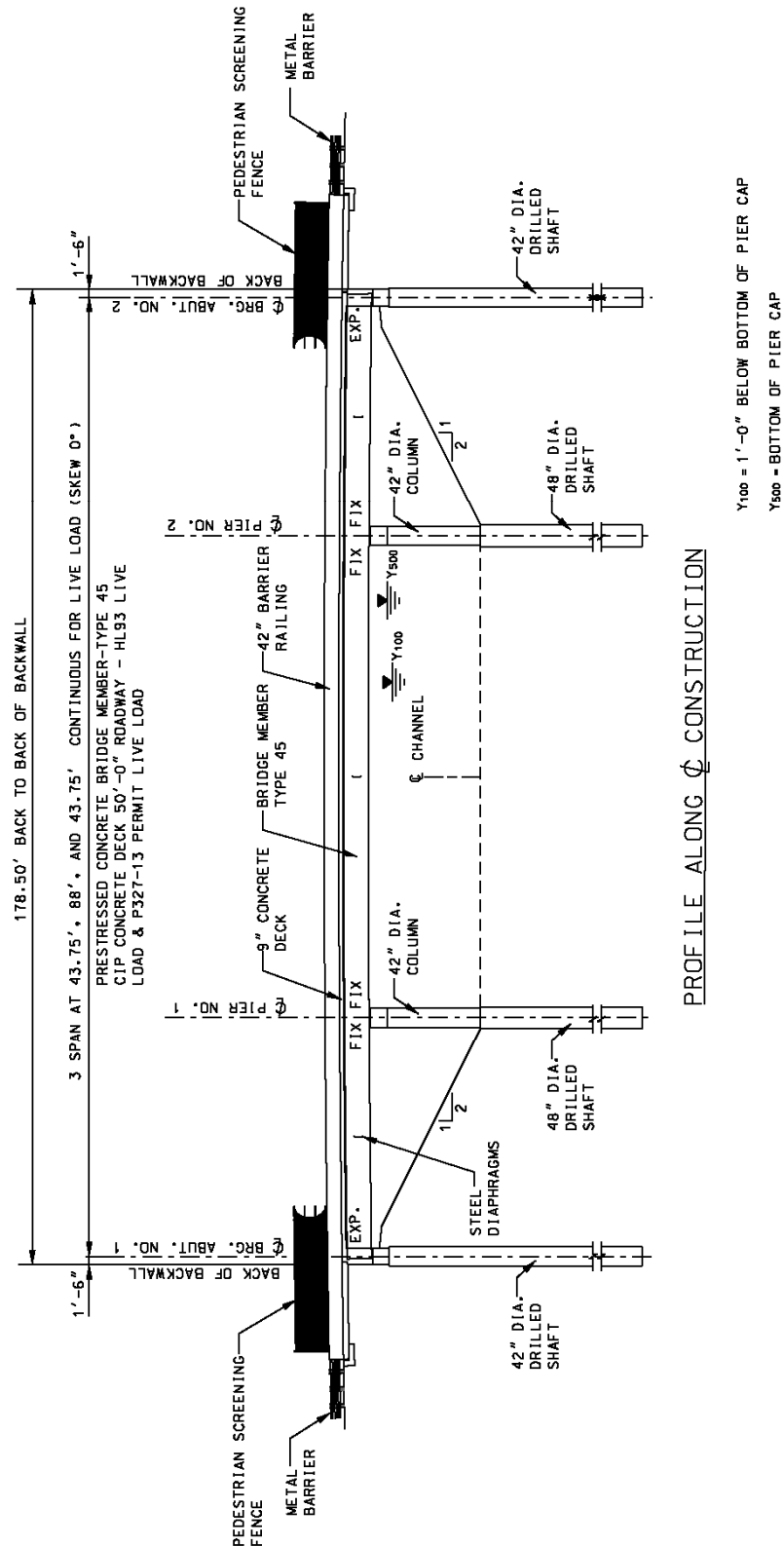
The superstructure design includes the following elements: deck design, prestressed girder design, and bearing pad design. Deck design follows the NMDOT standard deck slab detail in Chapter 4 of the *NMDOT Bridge Procedures and Design Guide*, hereinafter referred to as *Design Guide*. Girder analysis and design is performed using the computer program CONSPAN, Version 09.00.03.01. Input data and design loads needed for the computer analysis are developed and listed. From the resulting output, a final girder design is developed and finally the NMDOT standard beam sheet is completed. Reinforced elastomeric bearing pad design is also illustrated.

The LRFD design vehicular live load as specified in section 3.6.1.2 of the *LRFD Specifications* is designated as a HL-93 and consists of a combination of the design truck or design tandem and design lane load. The NMDOT also requires that new bridges be designed for the NMP327-13 permit load. (Exceptions for the NMP327-13 permit load will be provided for unique bridges.) The design engineer shall design the superstructure with the specified live load, but shall also ensure that the design produces at least the appropriate LFD inventory rating. All new bridges must have a Virtis/Opis inventory rating of HS25 and operating rating of HS42. The designer shall revise the original design if necessary to achieve the required bridge ratings. The Virtis/Opis rating shall be shown on the bridge plans. Additionally, the Virtis/Opis file used for rating the bridge is to be sent to the NMDOT Bridge Design Bureau.

The transverse section and profile views of the sample bridge follow.



## 4.1.2 Span Arrangements



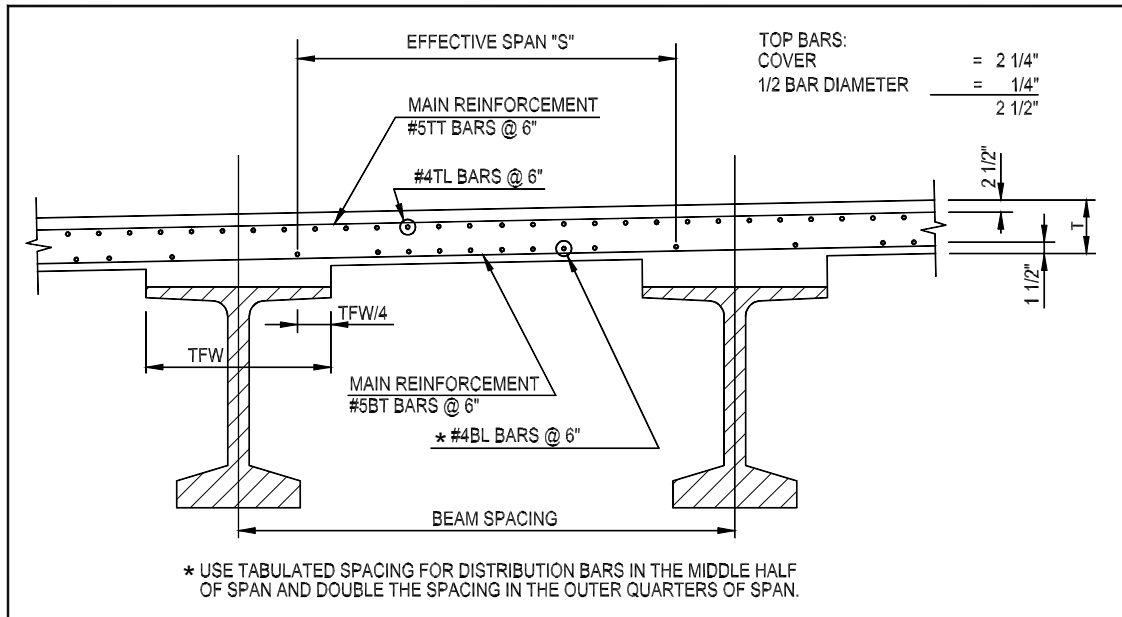


## 4.2 Deck Design

This example will use the standard deck slab design as explained in the *Design Guide*. This design should always be used unless approval to use a thinner deck is obtained from the State Bridge Engineer.

### 4.2.1 Standard Deck Slab Design

The NMDOT standard deck slab detail and slab design tables are shown below.



T (in)	S (ft)
7 1/2"	5'-7"
8"	6'-7"
8 1/2"	7'-7"
9"	8'-6"
9 1/2"	9'-5"
10"	10'-3"
10 1/2"	11'-1"
11	11'-10"

T=Slab Thickness

S=Effective Span

From the figure above, the main top and bottom reinforcement is #5 bars, spaced at 6 in. on center. Top longitudinal bars are #4TL bars, also spaced at 6 in. For each tabulated

deck thickness  $T$ , the design table lists the maximum effective span  $S$  and the distribution reinforcement spacing.

The following calculations illustrate how the slab thickness is derived for the bridge.

Determine Deck Thickness

$S$  = Beam Spacing = 8'-9"

$b_f$  = Top Flange Width = 1'-4"

$S_{eff}$  = Effective Span Length

$$S_{eff} = S - \frac{b_f}{2} = 8'-2"$$

Evaluating and rounding up to the nearest effective span length listed in the table gives:

$$S_{eff} = 8'-6" \rightarrow T = 9 \text{ in.}$$

#### **4.2.2 Deck Cantilever Design**

According to Appendix A13 of the *LRFD Specifications*, a bridge deck overhang shall be designed for the following design cases considered separately:

Design Case 1: the transverse and longitudinal forces specified in Article A13.2-Extreme Event Load Combination II limit state

Design Case 2: the vertical forces specified in Article A13.2 – Extreme Event Load Combination II limit state

Design Case 3: the loads, specified in Article 3.6.1, that occupy the overhang - Load Combination Strength I limit state.

The *Design Guide* indicates that the slab overhang design will not follow the practice of designing the deck slab overhang such that the railing system will fail before the deck does. If this practice is followed, the deck slab overhang would contain an excessive amount of reinforcing steel. As such, additional reinforcement will not be added to the deck for an Extreme Event and the designer may ignore design cases 1 and 2 above. The deck overhang will be designed for the dead load and live load that occupy the overhang.

For design case 3, application of design vehicular live load shall be in accordance with provision 3.6.1.3.4 of the *LRFD Specifications*. However, the NMDOT doesn't use structurally continuous barriers, so 3.6.1.3.4 cannot be used. Instead, the 16 kip live load will be placed 1 ft. from the face of the barrier rail.

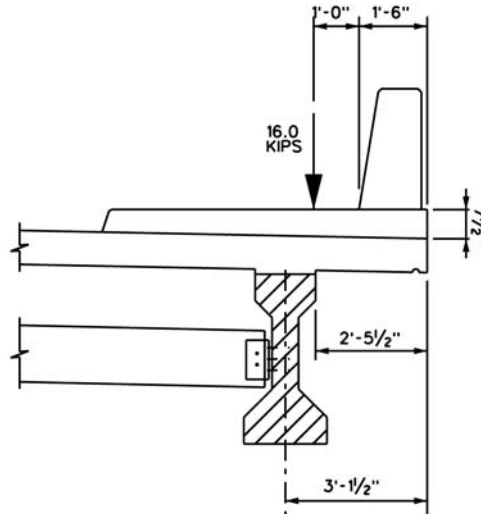
### Effective Strip Width

The effective strip width for the deck slab overhang is calculated using the applicable equation (based on deck type and direction of strip relative to traffic) from Table 4.6.2.1.3-1 of the *LRFD Specifications*:

$$W_{\text{eff}} = 45 + 10 X$$

Where

X = distance from load to point of support (ft.)



Determine the overhang distance:

$$W_{\text{flange}} = 1'-4''$$

$$S_{\text{exterior}} = 3'-1 \frac{1}{2}''$$

$$x_{\text{overhang}} = S_{\text{exterior}} - \frac{W_{\text{flange}}}{2} = 2'-5 \frac{1}{2}''$$

Since the live load is located above the girder flange or off the deck slab overhang, the live load will not be applied to the design and the maximum loading will be on the right side of the bridge with the sidewalk loading.

Cross Section Area of the Overhang Members

$$A_{\text{rail}} = \frac{9.75 + 16.5}{2} \times 42 \times \frac{1}{12^2} = 3.828 \text{ ft.}^3/\text{ft.}$$

$$A_{\text{sidewalk}} = x_{\text{overhang}} \times 7.5 \text{ in.} = 1.536 \text{ ft.}^3/\text{ft.}$$

$$A_{\text{slab}} = x_{\text{overhang}} \times t_{\text{slab}} = 1.844 \text{ ft.}^3/\text{ft.}$$

### Dead Load

$$\begin{aligned}w_{\text{rail}} &= A_{\text{rail}} \times 0.150 \text{ kip/ft.}^3 = 0.574 \text{ kip/ft.} \\w_{\text{sidewalk}} &= A_{\text{sidewalk}} \times 0.150 \text{ kip/ft.}^3 = 0.230 \text{ kip/ft.} \\w_{\text{slab}} &= A_{\text{slab}} \times 0.150 \text{ kip/ft.}^3 = 0.277 \text{ kip/ft.}\end{aligned}$$

### Moment Forces

$$\begin{aligned}M_{\text{rail}} &= w_{\text{rail}} \times (29.5 \text{ in.} - 8.5 \text{ in.}) = 1.00 \text{ kip-ft. / ft.} \\M_{\text{sidewalk}} &= w_{\text{sidewalk}} \times 29.5 \text{ in.} / 2 = 0.283 \text{ kip-ft. / ft.} \\M_{\text{slab}} &= w_{\text{slab}} \times 29.5 \text{ in.} / 2 = 0.340 \text{ kip-ft. / ft.}\end{aligned}$$

### Factored Moment Force

$$M_{\text{negU}} = 1.25(M_{\text{rail}} + M_{\text{sidewalk}} + M_{\text{slab}}) = 2.03 \text{ kip-ft. / ft.}$$

The negative moment resistance capacity of the deck slab overhang with the given amount of steel is:

$$\begin{aligned}A_s &= \#5\text{TT bars at } 6 \text{ in. spacing} = 0.62 \text{ in.}^2 / \text{ft.} \\b &= 12 \text{ in.} \\d &= 9 \text{ in.} - 2.5 \text{ in.} = 6.5 \text{ in.}\end{aligned}$$

$$a = \frac{A_s \times F_y}{0.85 \times f_c \times b} = 0.912 \text{ in.}$$

$$\phi M_n = 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) = 202.4 \text{ kip-in. / ft.} = 16.86 \text{ kip-ft. / ft.}$$

$$\phi M_n = 16.86 \text{ kip-ft. / ft.} > M_{\text{negU}} = 2.03 \text{ kip-ft. / ft.} \quad \text{OK}$$

### 4.3 Girder Design

It is expected that the interior girders will experience a larger share of the total live load and dead load forces. Typically, this assumption needs to be verified. For this example, only the interior design will be shown. In accordance with the *LRFD Specifications* Section 2.5.2.7.1, the resulting design is used for the exterior girders as well, if the loading assumption is correct.

The preliminary design uses six rows of 45 in. prestressed concrete girders, spaced at 8'-9" (see Transverse Section). This configuration will be analyzed, and a prestressing strand pattern designed using the CONSPAN computer program.

For program input, dead loads must be calculated and design data assembled. Once the computer analysis is run, a final girder design is developed. In addition to the

prestressing strand pattern determined by the program, final design also involves designing the transverse steel layout to satisfy vertical and horizontal shear and end anchorage requirements, and determining negative moment reinforcing at interior supports. The final step is to complete the NMDOT standard beam sheet, showing final design configuration and design values necessary for fabrication and erection.

This section will show development of input data including dead load calculation and live load selection, the computer analysis output file, final design using the computer output, and finally the filled out NMDOT standard beam sheet based on the final design.

#### **4.3.1 Loads**

The CONSPAN program calculates the girder, deck and haunch loads internally, along with live load distribution and impact factors, and all live load plus impact forces. Additional non-composite dead loads (stay-in-place forms, diaphragms) and composite dead loads must be calculated and input into the program.

##### Non-Composite Dead Loads

Diaphragms:

Use NMDOT standard drawing for intermediate steel diaphragm details.

Prestressed Concrete Beam Type = Type 45

S = Girder Spacing = 8'-9"

$\gamma_{diap}$  = Diaphragm Unit Weight = 28 lbs./ft.

$P_{clip}$  = Clip Angle Weight = 16 lbs.

$$P_{diap} = (S - A) \times \gamma_{diap} + 2 \times P_{clip}$$

$$P_{diap} = \left(8.75 - \frac{10}{12}\right) \times 28 + 2 \times 16 = 253.67 \text{ lbs.}$$

Non-Composite DC Load

Haunch:

During construction, the actual haunch dimensions will vary from the assumed 2 in. dimension. To ensure the design of the prestressed beam is adequate for the possible haunch dimensions, the prestressed beam will be designed for both a 0 in. and a 2 in. haunch. If during construction, the actual haunch dimensions are outside of the 0 in. to 2 in. range, the designer would need to verify the design based on the actual haunch dimensions before approving the haunch dimensions submitted by the contractor.

For this example, the two separate designs will be combined into one design. The design will be based on a 0 in. haunch dimension, but the weight of the 2 in. haunch will be manually added with the stay-in-place forms as a non-composite dead load. The approach will be slightly conservative, but it will only require one design.

$b_f$  = Top Flange Width = 1'-4"

$h_{haunch}$  = Height of the Haunch = 2 in.

$$\gamma_{\text{conc}} = 150 \text{ lbs. / ft.}^3$$

$$W_{\text{Haunch}} = b_f \times h_{\text{haunch}} \times \gamma_{\text{conc}} = 33.33 \text{ lbs./ ft.}$$

Stay-in-place Forms (and 2 in. haunch):

$$S = \text{Beam Spacing} = 8' - 9''$$

$$b_f = \text{Top Flange Width} = 1' - 4''$$

$$W_{\text{SIP}} = (S - b_f) \times 15 \text{ lbs./ ft.}^2 + 33.33 \text{ lbs./ ft.}$$

$$W_{\text{SIP}} = (8.75' - \frac{16''}{12}) \times 15 + 33.33 = 144.58 \text{ lbs./ ft.} \quad \text{Non - Composite DC Load}$$

#### Composite Dead Loads

Composite dead loads (Superimposed Dead Load in CONSPAN) are input as a load per length or area. The barrier wall, pedestrian fence, sidewalk and future wearing surface weights are the only composite loads for this bridge. Other projects also may include such items as utilities, pipe supports, and light pole pedestals.

Barrier & Pedestrian Screen Fence:

$$V_{\text{barrier}} = \text{Volume of the concrete barrier per unit length} = 3.83 \text{ ft.}^3 / \text{ft.}$$

$$\gamma_{\text{conc}} = 150 \text{ lbs. / ft.}^3$$

$$w_{\text{barrier}} = V_{\text{barrier}} \times \gamma_{\text{conc}} = 3.83 \times 150 = 575 \text{ lbs./ft.}$$

Increase barrier loading by 5% to account for Pedestrian Fence.

$$w_{\text{barrier \& Fence}} = w_{\text{barrier}} \times 1.05 = 603.75 \text{ lbs./ft.} \quad \text{Composite DC Load}$$

Future Wearing Surface:

$$\text{FWS} = 30 \text{ lbs./ft.}^2 \quad \text{Composite DW Load}$$

Sidewalk:

$$V_{\text{sidewalk}} = \frac{(6 \text{ in.} + 7.5 \text{ in.})}{2 \times 12} * \frac{86 \text{ in.}}{12} = 4.03 \text{ ft.}^3 / \text{ft.}$$

$$w_{\text{sidewalk}} = V_{\text{sidewalk}} \times \gamma_{\text{conc}} = 605 \text{ lbs./ft.} \quad \text{Composite DC Load}$$

### 4.3.2 Preparation of Computer Input Data

Required input data are listed in this section. The data are taken from the Design Data section and Loads section of this document. Refer to Section 5.5 of the *Design Guide* for guidance in calculating girder haunches.

Although this example shows data required for the CONSPAN program, bear in mind that other prestressed girder programs will require similar data.

LEAP@ CONSPAN® VBi (SELECTseries 1) - G:\Bridge On-Call\LRFD Design Examples\Report\W...

File Show Libraries LEAP Bridge Help

New Open Save Save As Print Image Model Results Diagram DF Contours Beam Section Tendon Rebar Live Load

Project Geometry Materials Loads Analysis Beam Deck

Project Information

Project Name: LRFD Design Example, 3 Span PCI Girder

User Job Number: CN1234

State: NMDOT State Job Number: CN1234

Date: 1/5/2011 By: PB

Date Checked: Checked By:

Description: LRFD Design Example, 3 Span PCI Girder

Design Code: ☒ LRFD ☐ Standard (LFD)

Units: ☒ U.S. Units ☐ SI Units (Metric)

☐ Flared Girder

State Specification: None

Span Type: ☐ Simple Span ☒ Multi-Span (Continuous) ☐ Multi-Span (Non-Continuous)

Updates IBS:Not Connected IBS:Manual Code: LF

The main menu allows you to toggle between design procedure 1 (Multi-Span Non-Continuous) and design procedure 2 (Multi-Span Continuous).

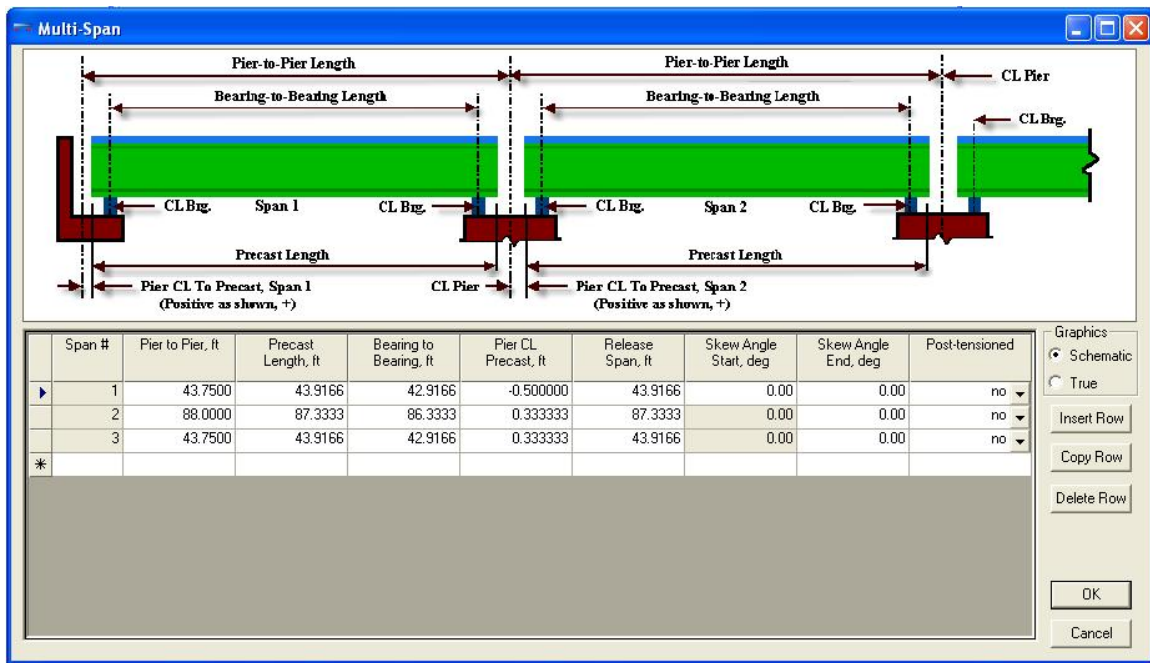
#### Bridge Layout

Overall width = 50 ft.  
Number of lanes = 3  
Lane width = 12 ft.  
Left and Right Curbs = 1.5 ft.  
Supplemental Layer = 0 in.  
Deck Thickness = 9 in.  
Haunch Thickness = 0 in.  
Haunch Width = 16 in.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

Span Data

Span #	Pier to Pier, ft.	Precast Length, ft.	Bearing to Bearing, ft.	Pier CL Precast, ft.	Release Span, ft.	Skew Angle, deg
1	43.75	43.92	42.92	-0.50	43.92	0
2	88.00	87.33	86.33	0.33	87.33	0
3	43.75	43.92	42.92	0.33	43.92	0



Beam Location Data

Beam No.	Beam Type	Beam ID	Dist. From Last Beam, ft.
1	I-Girder	AASHTO-III	3.125
2	I-Girder	AASHTO-III	8.75
3	I-Girder	AASHTO-III	8.75
4	I-Girder	AASHTO-III	8.75
5	I-Girder	AASHTO-III	8.75
6	I-Girder	AASHTO-III	8.75



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

The screenshot shows the LEAP CONSPAN V8i (SELECTseries 1) software interface. The title bar indicates the file path: G:\Bridge On-Call\LRFD Design Examples\Report\Design Example Rpt January 2011- Superstr U... The menu bar includes File, Show, Libraries, LEAP Bridge, and Help. The toolbar contains icons for New, Open, Save, Save As, Print, Image, Model, Results, Diagram, DF Contours, Beam Section, Tendon, Rebar, Live Load, Help, Bentley Site, About, E-mail, and User Manual. The main window is divided into several panels:

- Bridge Layout:** Overall Width: 50.0 ft
- Span Data:** Add/Edit Multi-Span...
- Curb Data:** Left: 1.5 ft, Right: 1.5 ft
- Lane Data:** Number: 3, Width: 12.0 ft
- Topping Data:** Deck Thick (effective): 9.0 in, Deck Thick (sacrificial): 0.0 in, Haunch Thickness: 0.0 in, Haunch Width: 16.0 in, Auto Set Haunch width: ☐
- Beam Type / Location:** Span: 01, Beam Type: I-Girder
- Beam Table:**

Beam No	Beam ID	Dist. From Last Beam, ft
1	AASHTO-III	3.1250
2	AASHTO-III	8.7500
3	AASHTO-III	8.7500
4	AASHTO-III	8.7500
5	AASHTO-III	8.7500
6	AASHTO-III	8.7500
*		
- Buttons:** Add Row, Delete Row, Copy Row, Copy To All, Generate...
- Diagram:** A schematic diagram of a 50 ft bridge with 6 girders. The spacing between girders is 8.75 ft, with 3.125 ft from the ends to the first and last girders.

The status bar at the bottom shows: Ready, Updates, IBS:Not Connected, IBS:Manual, Code: LRFD, NUM.

### Concrete Data

	Girder Release	Girder Final	Deck
Unit Weight (pcf)	150	150	150
Strength (ksi)	7.0	9.5	4.0

The approved mix design that is typically used by prestressed girder fabricators in New Mexico typically produces a final concrete strength of 9.5 ksi. The designer shall use 9.5 ksi strength concrete in the design even if a lower strength concrete could be used so that the beam deflection and cambers are more accurately predicted.

### Strand Data

Strand ID: ½ in. – 270K-LL  
 Depress: Draped, 0.40 Pt., 2.0 in. Increment  
 Elasticity of Prestressed Steel: 29000 ksi

### Rebar Data

Grade 60 for tension rebar  
 Elasticity: 29000 ksi

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

**Main**

Project Geometry Materials Loads Analysis Beam

Concrete

	Girder Release	Girder Final	Deck	
Unit weight	150.	150.	150.	pcf
Strength	7.	9.5	4.	ksi
K1	1.	1.	1.	
Elasticity	5072.24	5908.98	3834.25	ksi
Poisson's Ratio				0.2

Tension Rebar

Elasticity 29000. ksi fy 60. ksi

Transformation of Steel

☐ Transform All Prestressing Tendons

☐ Transform Rebars

Rebar Area 0. in<sup>2</sup> Ycg 0. in

Prestressing Tendon

Tendon ID

1/2" (7W-270) LR  
1/2-250K-1  
1/2-250K-LL-1  
1/2-270K  
1/2-270K-1  
1/2-270K-LL  
1/2-270K-LL-1  
1/2-270K-LL-M  
1/2-270K-SP  
1/2-270K-SP-1  
3/8-250K-1  
3/8-270K

Pattern

☐ Straight

☒ Draped/Straight

Depress Pt. Fraction 0.4

Midspan Increment 2 in

Debonding Length Increment, in 24.

Maximum Auto-Debonding Percentage

Per Row, % 40.

Total, % 25.

### Truck Data

Design Truck (HL93) and Lane Loading  
Design Tandem and Lane Loading  
P327-13 Permit Vehicle

LEAP® CONSPAN® V8i (SELECTseries 1) - G:\Bridge On-Cal\LRFD Design Examples\Report\Design Example Rpt March 2011- Substr Updat...

File Show Libraries LEAP Bridge Help

New Open Save Save As Print Image Model Results Diagram DF Contours Beam Section Tendon Rebar Live Load Help Bentley Site About E-mail User Manual

Project Geometry Materials Loads Analysis Beam Deck

Permanent - using Wizard

Span #	Beam #	Components	Types	Mag1	oc1/W/width	Mag2	Loc2, ft	Units	Description
1	1	Precast DC	Line Load	0.1450	0.0000	0.1450	42.9166	k/ft	Stay-in-Place Deck Forms
1	1	Composite	Composite DC	0.6050	0.0000	0.6050	42.9166	k/ft	Right Barrier Weight
1	1	Composite	Composite DC	0.6050	0.0000	0.6050	42.9166	k/ft	Right Sidewalk
1	1	Composite	Composite DW	0.0300	47.0000	0.0000	0.0000	ksf	Future Wearing Surface
1	2	Precast DC	Line Load	0.1450	0.0000	0.1450	42.9166	k/ft	Stay-in-Place Deck Forms
1	3	Precast DC	Line Load	0.1450	0.0000	0.1450	42.9166	k/ft	Stay-in-Place Deck Forms
1	4	Precast DC	Line Load	0.1450	0.0000	0.1450	42.9166	k/ft	Stay-in-Place Deck Forms
1	5	Precast DC	Line Load	0.1450	0.0000	0.1450	42.9166	k/ft	Stay-in-Place Deck Forms
1	6	Precast DC	Line Load	0.1450	0.0000	0.1450	42.9166	k/ft	Stay-in-Place Deck Forms
1	1	Composite	Composite DC	0.6050	0.0000	0.6050	42.9166	k/ft	Left Barrier Weight

Diaphragm - using Wizard

Span #	Beam #	Location, ft	Load, kips
1	1	21.9583	0.1307
1	2	21.9583	0.2613
1	3	21.9583	0.2613
1	4	21.9583	0.2613
1	5	21.9583	0.2613
1	6	21.9583	0.1307

Design Live Load

☒ Include LL Deflection

☐ Include Pedestrian Load 0. plf

Available

Design Lane  
Design Tandem  
Design Truck  
Double Tandem  
Double Truck  
PV-1(PV)  
Metric Design Lane  
Metric Design Tandem

Selected

Design Lane  
Design Tandem  
Design Truck  
Double Tandem - not incl.  
Double Truck  
P327-13

Ready

Updates IBS Connected IBS Manual Code LRFD CAP NUM

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

Note: NMDOT does not use live load deflection criteria; however, the live load deflection output from CONSPAN will be used to calculate cyclic rotation for the bearing design.

**Permanent Load Wizard**

Left Barrier Weight: 0.605 klf

Left Curb Weight: 0. klf

Future Wearing Surface: 0.03 ksf

Right Curb Weight: 0. klf

Right Barrier Weight: 0.605 klf

Left Sidewalk: 0. klf

Stay-in-Place Deck Forms: 0.145 klf

Right Sidewalk: 0.605 klf

☒ Keep Values

OK Cancel

**Diaphragm Load Wizard**

View Options...

Span #	Location, ft	Magnitude, plf	Skew Angle, deg
1	21.9583	32.0000	0.00
2	43.6667	32.0000	0.00
3	21.9583	32.0000	0.00
*			

Insert Row Copy Row Delete Row Delete All

OK Cancel

### Analysis Factors

Distribute Dead Loads: Equally to all beams

Dead Load: Computed

Dynamic Load Factor: 0.33 (Truck), 0 (Lane), 0.33 (Strength II), 0.15 (Fatigue)

Live Load: Use Code Equations

Load Factors: per Design Specifications.

Modifier: Ductility = 1.0, Redundancy = 1.0, Importance = 1.05

**Analysis Factors**

**Distribution** | Load Factors | Modifier

**Distribute Dead Load**

- ☒ Equally to all beams
- ☐ Based on Tributary Fraction
- ☐ Use Composite DC Distribution as per FDOT

**Dead Load**

- ☒ Computed ☐ Manual
- Span: 01 Beam: 1
- Apply to All Beams
- Comp. DC: 0.166667
- Comp. DW: 0.166667

**Dynamic Load Factor**

Truck: 0.33  
Lane: 0  
Strength II: 0.33  
Fatigue: 0.15

**Live Load - Girder**

Span: 01 Beam: 1

- ☐ Manual
- ☒ Use Code Equations
- ☐ Use Refined Method of Analysis
- Apply to All Beams

Moment -ve  
2+ Loaded Lane: 0.784/0.741  
1 Loaded Lane: 0.737/0.737

Moment +ve  
2+ Loaded Lane: 0.783499  
1 Loaded Lane: 0.737143

Shear  
2+ Loaded Lane: 0.741497  
1 Loaded Lane: 0.737143

Pedestrian: 0.166667

**Live Load LRFD params**

- ☒ Use Permit Vehicle side by side with design loads for Strength II
- ADTT: 5000 Apply ADTT: 1
- ☒ Include Rigid Cross-section Assumption (Art. 4.6.2.2.2d)

OK Cancel

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

**Analysis Factors**

Distribution | Load Factors | Modifier

Strength I Limit State

	Max.	Min.
DC	1.25	0.9
DW	1.5	0.65
Live	1.75	

☒ Included

Strength II Limit State

	Max.	Min.
DC	1.25	0.9
DW	1.5	0.65
Live	1.35	

☒ Included

Service I Limit State

DC / DW	Live
1.	1.

☒ Included

Service III Limit State

DC / DW	Live
1.	0.8

☒ Included

Fatigue I Limit State

Live
1.5

☒ Included

[Load Factors](#)

OK Cancel

**Analysis Factors**

Distribution | Load Factors | Modifier

Ductility: 1.

Importance: 1.05

Redundancy: 1.

OK Cancel

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

**Project Parameters**

Limiting Stress: Use Factor to Calculate

Restraining Moments: Full Continuity, Disregard Restraining Moments

Multipliers: per Design Specifications

Resistance Factor/Losses: 0.9 Flexure, 1.0 Flexure Prestressed, 0.9 Shear

AASHTO Method to Compute Losses with 25% humidity.

Moment and Shear Provisions:

Moment Method: AASHTO equations

Negative M Reinforced Design: Exclude Non-Composite Moments in  $M_u$

Horizontal Shear Method: Exclude Beam and Slab Contribution in  $V_u$

Vertical Shear Method: Simplified (for consistency with Virtis/Opis LFR)

**Project Parameters**

Limiting Stress | Restraining Moments | Multipliers

Resistance Factor/Losses | Moment and Shear Provisions

Strength Reduction Factors,  $\Phi$

Reinforced:		Prestressed:		Shear:
Tens:	Comp:	Tens:	Comp:	
0.9	0.75	1.	0.75	0.9

c/dt ratio  
Compression--> 0.6 <--Transition--> 0.375 <--Tension

Prestress Losses

☒ AASHTO ☐ Manual

Relative Humidity: 25. %  
Release: 10. %  
Days

Release Time (ti): 0.75  
Age of deck placement (td): 56.  
Final Age (tf): 20000.

Steel Relaxation

☒ AASHTO ☐ by Tadros

KL 30. K'L 45.

Compute Losses using

☒ Approximate Method ☐ Refined Method

☐ Pre '2005 Interims' LRFD spec ☐ Neglect Elastic Gains

☐ Compute ES using Eq. C5.9.5.2.3a

OK Cancel



**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

**Project Parameters**

Limiting Stress    Restraining Moments    Multipliers  
Resistance Factor/Losses    Moment and Shear Provisions

Moment Method

- ☒ AASHTO equations
- ☐ Strain Compatibility  
Ultimate Concrete Strain:

Negative Moment Reinforced Design

- ☐ Include Non-Composite Moments in Mu
- ☒ Exclude Non-Composite Moments from Mu

Horizontal Shear Method

- ☐ Include Beam and Slab Contribution in Vu
- ☒ Exclude Beam and Slab Contribution from Vu

Vertical Shear Method

- ☐ General (LRFD 5.8.3.4.2) Beta Theta Tables
- ☐ General (LRFD 5.8.3.4.2) Beta Theta Equations
- ☒ Simplified (LRFD 5.8.3.4.3)

Modulus of Rupture

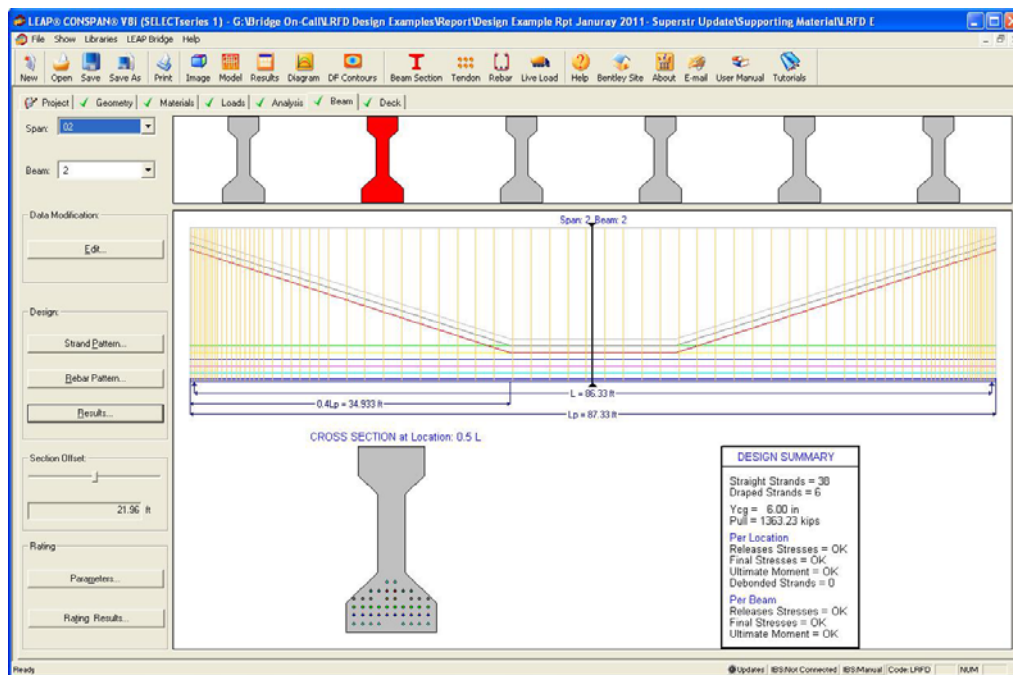
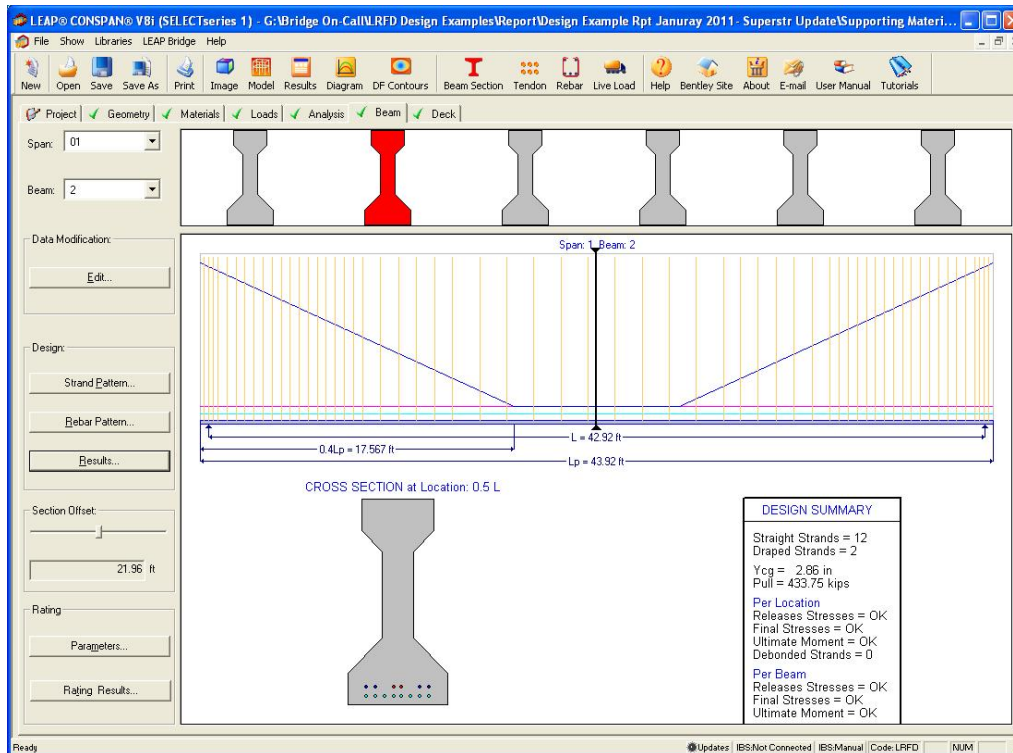
- ☒ AASHTO equation
- ☐ User Defined  x sqrt (f'c)

OK Cancel

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

### 4.3.3 Computer Analysis results

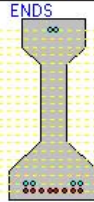
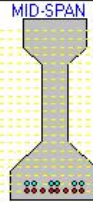
The shear reinforcement and strand pattern are shown below for Spans 1 & 3 and Span 2.





# **EXAMPLE NO.1: Concrete Bridge LRFD Specifications**

**Strand Pattern: Span 1, Beam 2**

Type	End Template	End Height	Middle Height	# of Strands
► Straight	10	2.00	----	8
Straight	10	4.00	----	4
Draped	2	42.00	4.0000	2
*				

Strand Pattern Library  
☐ Save/Load Strand Patterns

Pattern

SYMMETRICAL DRAPED

☐ Initial Pull/CG Method

Kern Points (in)

Lower: 11.22  
Upper: 31.32

Total Strands: 14

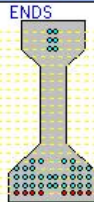
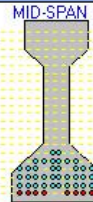
Ycg (in)

End: 8.29  
Mid: 2.86

Draping

Design

**Strand Pattern: Span 2, Beam 2**

Type	End Template	End Height	Middle Height	# of Strands
► Straight	10	2.00	----	8
Straight	10	4.00	----	10
Straight	10	6.00	----	10
Straight	8	8.00	----	6
Straight	6	10.00	----	4
Draped	2	38.00	8.0000	2
Draped	2	40.00	10.0000	2
Draped	2	42.00	12.0000	2
*				

Strand Pattern Library  
☐ Save/Load Strand Patterns

Pattern

SYMMETRICAL DRAPED

☐ Initial Pull/CG Method

Kern Points (in)

Lower: 11.22  
Upper: 31.32

Total Strands: 44

Ycg (in)

End: 10.09  
Mid: 6.00

Draping

Design

#### **4.3.4 Computer Output**

The CONSPAN output file is shown in Appendix A. The output lists the following items:

- Input parameters
- Composite dead and live load forces
- Section properties
- Design strand pattern and end and midspan sections
- Service load forces
- Flexural stresses
- Vertical and horizontal shear required reinforcement
- Ultimate capacity and minimum steel data
- Camber and deflections
- Detensioning data
- Negative moment flexural stresses and forces

#### **4.3.5 Final Girder Design**

The computer output gives an optimum strand pattern developed to satisfy the input loads. In addition, the computer program also develops a transverse steel arrangement and negative moment reinforcement.

##### Hold Down Force

As noted in the input data, draped strands are used in this design. This is typical although some design or fabrication circumstances may dictate use of debonded strand instead of, or in conjunction with, draped strands.

As detailed in the program output, a total of 14 strands (spans 1 & 3) and 44 strands (span 2) are used, with 2 strands (spans 1 & 3) and 6 strands (span 2) draped beginning at the 0.4L point.

Based on experience with local fabricators, the maximum hold down force at the harped point of the draped strands is limited to 40 kips. The hold down force reported in the output is 10.993 kips (span 1 & 3) and 13.270 kips (span 2), which are both less than the 40 kips maximum. If the hold down force is greater than the allowed amount by the fabricator, first try lowering the draped strands at the end; if this doesn't work use multiple hold down points as required.

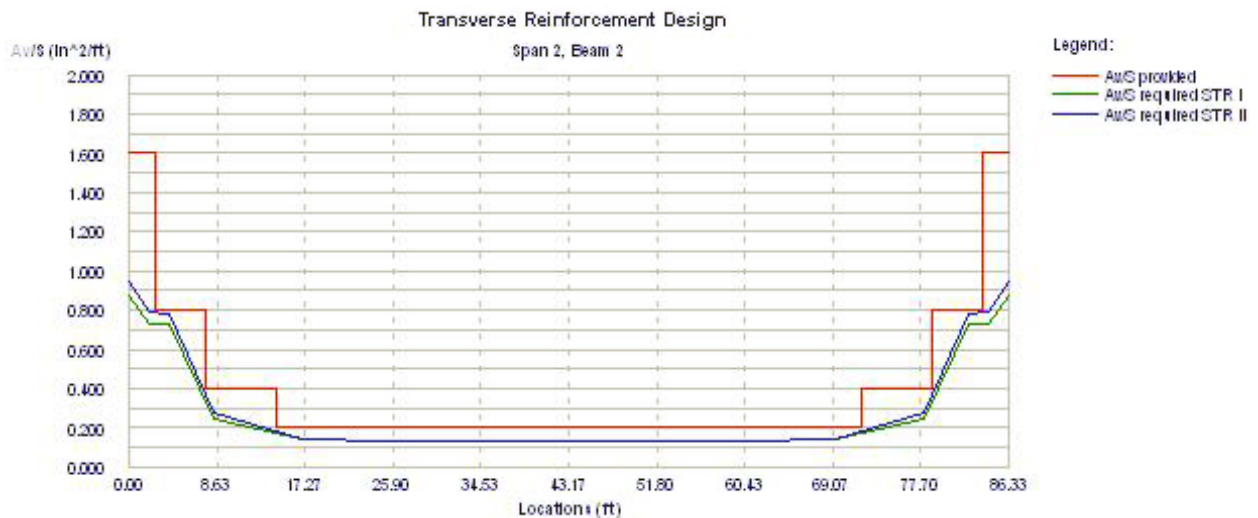
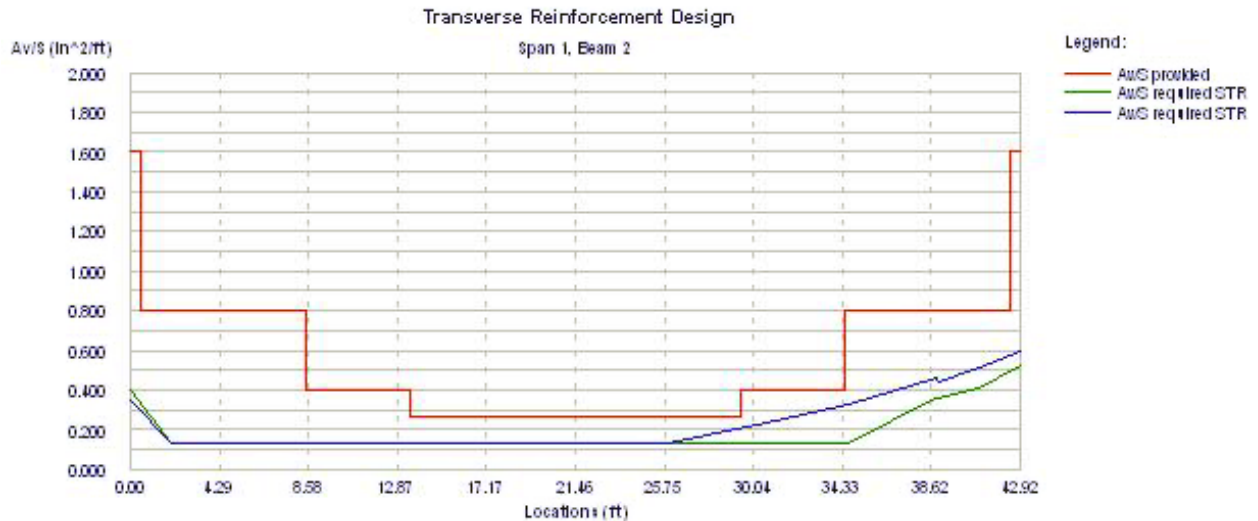
##### Transverse Steel Layout

The placement of transverse reinforcing steel must satisfy requirements for both vertical and horizontal shear capacity. In addition, at the girder ends additional reinforcement is generally required to resist tensile stresses in the vicinity of the end anchorage.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

### Vertical Shear Reinforcement

The computer output calculates the required reinforcing area per foot length and is listed as  $A_v/s$ . The computer program also allows the engineer to develop the vertical shear layout. The following shear envelopes have been developed with a pair of #4 bars for stirrups.



### Horizontal Shear Reinforcement

The calculated reinforcement requirement (per foot of length) for horizontal shear is listed in the program output. Since the same reinforcement (pairs of #4 bars) will be used to satisfy both vertical and horizontal shear, the area provided is again  $A_v = 0.4 \text{ in.}^2$ .

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

Note that two steel areas are listed in the output,  $A_{vh-sm}$  and  $A_{vh-rg}$ . These two values correspond to the top flange surface conditions listed in the *LRFD Specification* Section 5.8.4.2. The NMDOT requires that the top surface of girder flanges be intentionally roughened to an amplitude of 0.25 in. Therefore, the values for  $A_{vh-rg}$  are used. The required spacing (in.) of horizontal reinforcing is then:

$$A_v = 2 \times 0.2 \text{ in}^2 = 0.4 \text{ in}^2$$

$$S_{reg'd} = \frac{A_v}{A_{vh-rg}} \times 12$$

The table below lists the required reinforcing area and maximum spacing (based on minimum steel requirements) for vertical and horizontal shear from the program output. Also shown are the spacing requirements for pairs of #4 bars based on the above two equations. Controlling spacing at each section is bolded.

SPAN 1 & 3								
	Bearing	Transfer	Critical	0.1L / 0.9L	0.2L / 0.8L	0.3L / 0.7L	0.4L / 0.6L	0.5L
Location (ft.)	0.5	2.5	2.75/4.75	4.39	8.78	13.17	17.57	21.96
Vertical Shear								
$A_v$ (sq.in./ft.)	0.803	0.547	0.438	0.426	0.264	0.142	0.136	0.136
$S_{max}$ (in.)	24	24	24	24	24	24	24	24
$S_{req'd}$ (in.)	<b>6</b>	<b>9</b>	<b>11</b>	<b>11</b>	<b>18</b>	<b>24</b>	<b>24</b>	<b>24</b>
Horizontal Shear								
$A_{vh-rg}$ (sq.in./ft.)	0	0	0	0	0	0	0	0
$S_{max}$ (in.)	24	24	24	24	24	24	24	24
$S_{req'd}$ (in.)	24	24	24	24	24	24	24	24
$S$ provided (in.)	3	6	6	6	12	24	24	24

SPAN 2								
	Bearing	Transfer	Critical	0.1L / 0.9L	0.2L / 0.8L	0.3L / 0.7L	0.4L / 0.6L	0.5L
Location (ft.)	0.5	2.5	2.75/4.53	8.73	17.47	26.2	34.93	43.67
Vertical Shear								
$A_v$ (sq.in./ft.)	1.109	0.499	0.505	0.281	0.136	0.136	0.136	0.136
$S_{max}$ (in.)	24	24	24	24	24	24	24	24
$S_{req'd}$ (in.)	<b>4</b>	<b>10</b>	<b>10</b>	<b>17</b>	<b>24</b>	<b>24</b>	<b>24</b>	<b>24</b>
Horizontal Shear								
$A_{vh-rg}$ (sq.in./ft.)	0.019	0.015	0.011	0.007	0	0	0	0
$S_{max}$ (in.)	24	24	24	24	24	24	24	24
$S_{req'd}$ (in.)	24	24	24	24	24	24	24	24
$S$ provided (in.)	3	3	6	12	24	24	24	24

#### End Anchorage Reinforcement

According to the *LRFD Specification* Section 5.10.10, vertical stirrups must be provided within a distance  $h/4$  from the girder end to resist a minimum force equal to 4% of the total prestressing force. The reinforcement is assumed to act at a stress of 20 ksi.

The NMDOT standard bridge member detail sheet calls for the use of a single #7 vertical bar at the end of the girder. This bar will contribute to the overall resistance and will be accounted for in the calculations below. Additional resistance will be provided by closely spaced pairs of #4 bars.

Using the initial pull force just before release provided in the program output, the prestressing force  $P_i$ , just before transfer is:

$$P_i = (0.75 \times f_{pu}) \times A_s \times n$$

$f_{pu}$  = Prestressing Strand Tensile Strength (270 ksi)  
 $A_s$  = Prestressing Strand Area  
 $n$  = Number of Prestressing Strands

Span 1 & 3

$$P_i = (0.75 \times 270) \times 0.153 \text{ in.}^2 \times 14 = 433.8 \text{ kips}$$
$$F = 4\% \times P_i = 17.35 \text{ kips}$$

Span 2

$$P_i = (0.75 \times 270) \times 0.153 \text{ in.}^2 \times 44 = 1363.2 \text{ kips}$$
$$F = 4\% \times P_i = 54.53 \text{ kips}$$

Using a working stress of 20 ksi, the capacity of one #7 bar ( $A_s = 0.60 \text{ in.}^2$ ) is:

$$F_n = 20 \text{ ksi} \times 0.60 \text{ in.}^2 = 12.0 \text{ kips}$$

Subtracting this from the force  $F$  gives the force that must be resisted by additional stirrups:

Span 1

$$\Delta F = F - F_n = 17.35 \text{ kips} - 12.0 \text{ kips} = 5.35 \text{ kips}$$

The capacity of a pair of #4 stirrups ( $A_s = 0.40 \text{ in.}^2$ ) is:

$$F_n = 20 \text{ ksi} \times 0.40 \text{ in.}^2 = 8.0 \text{ kips}$$

(One pair of #4 stirrups added to the #7 bar is sufficient for End Anchorage Reinforcement)

Span 2

$$\Delta F = F - F_n = 54.53 \text{ kips} - 12.0 \text{ kips} = 42.53 \text{ kips}$$

The capacity of a pair of #5 stirrups ( $A_s = 0.62 \text{ in.}^2$ ) is:

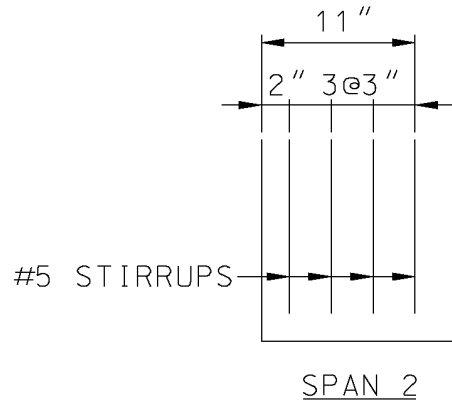
$$F_n = 20 \text{ ksi} \times 0.62 \text{ in.}^2 = 12.4 \text{ kips}$$

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

The number of bar pairs required is then:

$$\frac{\Delta F}{F_n} = \frac{42.53}{12.4} = 3.43$$

(Use 4 pairs of #5 bars. See figure to the right.)



Using the effective height of the girder alone, the distance  $h/4$  is 11.25 in.

*LRFD Specification* Section 5.10.10.2 also requires that prestressing strands in the lower flange be enclosed by steel reinforcing for the distance of  $1.5d$  from the end of the beams at a maximum spacing of 6 in. on center to confine the prestressing steel in the bottom flange. On the NMDOT Type 45 standard beam sheets, transverse hoops (H bars) are carried out a distance  $1.5d$  or 5.625 ft. from the end of the beam. The transverse hoop bars will be conservatively carried out a distance of 6.0 ft. from the ends of the beam for this example.

#### Release Stress (Tension)

The prestressing forces at release produce temporary tensile stresses in the top of the concrete member which need to be checked to make sure that they do not exceed the tensile capacity of the concrete. *LRFD Specification* Section 5.9.4.1 specifies that the temporary tensile stress shall not exceed 0.2 ksi without bonded reinforcement and  $0.24\sqrt{f'_c} = 0.63$  ksi with bonded reinforcement. The CONSPAN output reports that the tensile release stress for Span 1 and 3 beams is 0.37 ksi and for Span 2 beams is 0.20 ksi. A minimum two #4 bars are typically placed in the top flange of the prestressed bridge member. If the tensile stresses had been larger, additional bonded reinforcement could have been added in the top flange.

#### Negative Moment Reinforcement

Additional longitudinal reinforcement in the deck is generally required near piers to provide adequate moment capacity for negative bending. Away from the piers, however, the temperature and distribution longitudinal reinforcement is usually sufficient by itself.

The steel area provided by the temperature and distribution steel within the deck effective width is:

$$\begin{aligned} \text{Top mat (\#4 bars at 6 in. spacing): } A_{st} &= 17 \times 0.20 = 3.4 \text{ in.}^2 \\ \text{depth from the top of deck} &= 2.25 \text{ in.} + 0.625 \text{ in.} + 0.25 \text{ in.} = 3.125 \text{ in.} \end{aligned}$$

$$\begin{aligned} \text{Bottom mat (12 \#4 Bars): } A_{sb} &= 12 \times 0.20 \text{ in.}^2 = 2.4 \text{ in.}^2 \\ \text{depth from the top of deck} &= 9.0 \text{ in.} - 1.25 \text{ in.} - 0.625 \text{ in.} - 0.25 \text{ in.} = 6.875 \text{ in.} \end{aligned}$$

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

The following table, from the program output, lists the negative moment reinforcing needed at tenth points along each span. The temperature and distribution steel by itself is adequate for the majority of the length. Additional steel will be needed in the area of the piers. The #4 bars could be replaced with a larger bar or additional bars could be placed between the #4 bars.

x/L	X	Asb	Ast	Ast-p	Ast-r	Required Area
0	0.0	2.4	3.4	5.8	0	0
0.1	3.9	2.4	3.4	5.8	0.648	0
0.2	8.3	2.4	3.4	5.8	1.427	0
0.3	12.7	2.4	3.4	5.8	2.256	0
0.4	17.1	2.4	3.4	5.8	3.136	0
0.5	21.5	2.4	3.4	5.8	4.032	0
0.6	25.9	2.4	3.4	5.8	4.576	0
0.7	30.2	2.4	3.4	5.8	5.631	0
0.8	34.6	2.4	3.4	5.8	6.67	0.87
0.9	39.0	2.4	3.4	5.8	7.981	2.181
1	43.75	2.4	3.4	5.8	10.613	4.813
1.1	52.8	2.4	3.4	5.8	4.137	0
1.2	62.6	2.4	3.4	5.8	3.529	0
1.3	70.3	2.4	3.4	5.8	3.529	0
1.4	79.0	2.4	3.4	5.8	3.529	0
1.5	87.8	2.4	3.4	5.8		0

#### Positive Restraint Moments

The NMDOT follows the recommendations of *NCHRP Report 322, Design of Precast Prestressed Bridge Girders Made Continuous*, whereby positive restraint moments at piers are ignored. However, the NMDOT still requires that bent prestressing strands be used for connecting girders together at piers.

#### **4.3.6 Standard Beam Plan Sheet**

The NMDOT uses a standardized plan sheet to show prestressed girder details. Information from the preceding analysis is used to fill out the prestressed beam plan sheet, as shown in this section. Refer to the attached Type 45 standard beam sheet template and the attached Type 45 beam sheet that has been completed for this example.

Required items for completing the sheet are listed below. Sheet references in parentheses refer to the program output where the subject information is located. The details for spans 1 and 3 are completed. For an actual design, details for span 2 would be completed in a similar manner.

End View

- Revise the strand pattern to reflect the actual design,
- Add number of strand row spaces for top and bottom strands



Section near CL of Beam

- Revise the strand pattern to reflect the actual design,
- Add top projection dimension for #4S1 bars. For a 9 in. deck thickness, subtract 2 1/4 in. for clearance to the top of deck and 5/8 in. to avoid interference with the transverse deck bars. This yields 6 1/8 in., but to be conservative, set the projection at 6 in. This will provide minimum clearance to the top of deck for a 0 haunch height.

Beam Data

- Fill in girder weight, camber at release, camber at erection, and dead load deflection.

$$\begin{aligned}\text{Girder Weight} &= 583.3 \text{ lbs./ft.} \times 43.92 \text{ ft.} = 25.62 \text{ kips (Span 1 \& 3)} \\ &= 583.3 \text{ lbs./ft.} \times 87.33 \text{ ft.} = 50.94 \text{ kips (Span 2)}\end{aligned}$$

The camber at release is calculated using the prestress + self weight cambers (deflections) at release.

$$\begin{aligned}\text{Camber at Release} &= 0.367 \text{ in.} - 0.077 \text{ in.} = 0.290 \text{ in. (Span 1 \& 3)} \\ &3.557 \text{ in.} - 1.200 \text{ in.} = 2.357 \text{ in. (Span 2)}\end{aligned}$$

In accordance with General Note 4 on the beam sheet, the camber at erection is calculated using the prestress + self weight cambers (deflections), with an allowance for camber to 90 days. Therefore, use the camber/deflection value listed in the erection column of the beam output, which includes multipliers to account for the above time period:

$$\begin{aligned}\text{Camber at Erection} &= 0.660 \text{ in.} - 0.142 \text{ in.} = 0.518 \text{ in. (Span 1 \& 3)} \\ &6.403 \text{ in.} - 2.221 \text{ in.} = 4.182 \text{ in. (Span 2)}\end{aligned}$$

In accordance with General Note 5, the dead load deflection is calculated using the weight of deck (plus haunch), diaphragms, and superimposed dead load (SIP forms, composite dead load, etc.). Again using the values in the erection column,

$$\begin{aligned}\text{Dead Load Deflection} &= 0.124 \text{ in. (Span 1 \& 3)} \\ &= 2.03 \text{ in. (Span 2)}\end{aligned}$$



#### Half Elevation Type 45 Beams

- Add prestressing strand row profiles
- Add S bar (web reinforcement) spacing
- Add values for number of beams required,  $f_{se}$ , and losses
- Add girder bearing-to-bearing length, distance from centerline to hold down
- Fill in blank for total number of strands, and number of draped and straight strands
- Add length from girder CL to location for steel diaphragms

#### Reinforcing Bars Required for One Beam

- Add dimension X and Length for S1 bars, and length for T1 bars:

S1 Bars:

$$X = 24.75 \text{ in.}$$

$$\text{Length} = 2X + \pi R = 56.57 \text{ in. Use } 4'-9''$$

T1 Bars: For Spans 1 and 3, one bar can span the girder length. Deducting for a 2 in. clearance to the girder end, the length of one bar is:

$$\text{Length} = 43'-7''$$

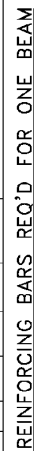
T1 Bars: For Span 2, use 2 bars. Deducting for a 2 in. clearance to the girder end, and including provisions for a 1'-6" splice at midspan the length of one bar is:

$$\text{Length} = 44'-3''$$

- Add number of bars required for S1, S2, H, T1, and T2 bars.

Delete both *Note to Designers*: notes in lower right corner of sheet.





#### 4.4 Bearing Pad Design

Elastomeric bearing pads, plain or reinforced, are typically used on bridges in New Mexico. The *LRFD Specifications* give two design procedures for reinforced elastomeric pads in Sections 14.7.5 (Method B) and 14.7.6 (Method A). The stress limits associated with Method A usually result in a bearing with a lower capacity than a bearing designed using Method B. This increased capacity resulting from the use of Method B requires additional testing and quality control. The Department prefers to use Method A as it is a conservative design and requires less testing (See Chapter 7 of the *Design Guide*).

Assume that transverse movement at all bearings will be prevented by concrete keeper blocks or by other methods. In addition, the pier bearings are considered fixed in the longitudinal direction. Longitudinal movements are unrestrained at the expansion bearings. As discussed below, the *Design Guide* specifies temperature ranges to be used in calculating structure movements in New Mexico.

The design method presented in the *LRFD Specifications* differentiates between locations where shear deformations are permitted and where they are not. The fixed bearings are prevented from deforming in shear by the dowels or by other methods. At the expansion bearings, shear deformations will occur.

##### 4.4.1 Abutment (Exp.) Bearing Pad Design

###### Loads

The (unfactored) forces can be obtained from the Service I shear and moment envelope in the program output. These same forces can be obtained using the method shown for live load on the following page. Bearing pad design is based on live load forces without the addition of an impact allowance.

$$R_{DL \text{ Self Wt.}} = 12.5 \text{ kips}$$

$$R_{DL \text{ Deck.}} = 21.1 \text{ kips}$$

$$R_{DL \text{ Diaphragm.}} = 0.1 \text{ kips}$$

$$R_{DL \text{ Prec. DC}} = 3.1 \text{ kips}$$

$$R_{DL \text{ Comp DC}} = 2.9 \text{ kips}$$

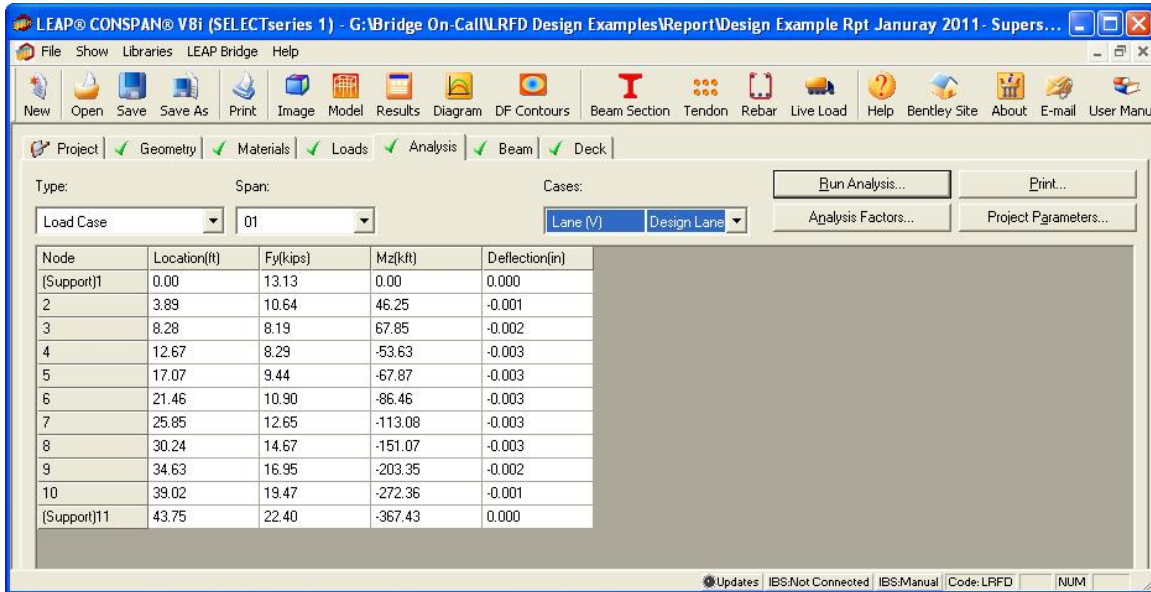
$$R_{DL \text{ Comp DW}} = 2.5 \text{ kips}$$

$$R_{DL \text{ Abut. Diaphragm \& Wingwalls}} = 33.0 \text{ kips (Abutment diaphragm, approach slab, backwall, and wingwall dead load value is not computed by the design program. Engineer will need to compute.)}$$

$$R_{DL \text{ Total}} = 75.2 \text{ kips}$$

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

Live loads are given in the program output per lane with no distribution factor and no impact. These loads can be found from the Analysis screen as shown below. To find live loads at the abutment, select Load Case from the Type pull down. Next, in the Span pull down, select span 01. Finally, from the Cases pull down, select the maximum shear live load (lane, truck, double truck, or permit) shear.



As shown in the above figure, Fy at Support 1 is 13.13 kips for the lane load. Adding this to the truck load of 54.36 kips at the same support, we get a total of 67.5 kips. This is the load per lane. The shear distribution factor for the beam we are designing is 0.879, giving us the design live load for bearings shown below.

$$R_{LL \text{ Total}} = 67.5 \text{ kips/Lane} \times DF_{\text{Shear}} = 67.5 \text{ kips/Lane} \times 0.879 = 59.33 \text{ kips/Beam}$$

### Structure Movement

In the longitudinal direction, superstructure movement occurs due to the combined effects of temperature changes plus creep and shrinkage of the prestressed girders.

#### Temperature:

Table 3.1B of the *Design Guide* specifies temperature ranges based on the values given in the *LRFD Specification* Section 3.12.2. From Table 3.1B for “South of I-40”, structure movement is to be based on a temperature range from 10° F to 90° F. The *Design Guide* states, “The full temperature range is used in design of the superstructure because the structure is anticipated to have these full movements during its life.” However, the *Design Guide* also says, “The thermal movement used in the design of elastomeric bearing pads shall be not less than 75% of the total anticipated movement due to temperature.”

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

$$\Delta L = L \times 0.000072 \times \Delta T$$

where:

L = Length of the bridge that will move due to temperature changes (ft.)

Each abutment will receive half of the total bridge movement.

$$L = 44 \text{ ft.} + 43.75 \text{ ft.} = 87.75 \text{ ft.}$$

$$\Delta T = 80^\circ \text{ F}$$

$$\Delta L = 87.75 \text{ ft.} \times 0.000072 \times 80^\circ \text{ F} \times 75\% = 0.38 \text{ in.}$$

Shrinkage and Creep:

Movement due to shrinkage and creep can be found from time dependent losses in the program output. The values from the program output are consolidated in the following table.

Symbol	Description	Span 1	Span 2
$\Delta_{ES}$	Beam shortening (PL/AE)	0.077 in	0.455 in
$\Delta f_{pSH}$	Concrete shrinkage loss, final	10.88 ksi	10.87 ksi
$\Delta f_{pCR}$	Concrete creep loss, final	7.02 ksi	22.06 ksi
$\Delta f_{pES}$	Initial total prestress loss	8.55 ksi	19.44 ksi

Because bearings at the piers are fixed, all of the movement from Span 1 and half of the movement of Span 2 is assumed to be taken up by the bearings at Abutment 1. Also, 50% of the total creep and shrinkage is assumed to occur before beam erection. Steel relaxation is neglected. Calculations for both spans are shown below. This approach is derived from the NYSDOT Bridge Manual, 1<sup>st</sup> edition with Addendum, 2010.

Alternatively, *LRFD Specification* Section 5.4.2.3 could be used.

$$\Delta_{SPAN\_1} = \frac{(\Delta f_{pSH} + \Delta f_{pCR})(\%_{SH+CR})}{\Delta f_{pES}} (\Delta_{ES}) = \frac{(10.88 \text{ ksi} + 7.02 \text{ ksi})(0.5)}{8.55 \text{ ksi}} (0.077 \text{ in.}) = 0.081 \text{ in.}$$

$$\Delta_{SPAN\_2} = \frac{(\Delta f_{pSH} + \Delta f_{pCR})(\%_{SH+CR})}{\Delta f_{pES}} (\Delta_{ES}) = \frac{(10.87 \text{ ksi} + 22.06 \text{ ksi})(0.5)}{19.44 \text{ ksi}} (0.455 \text{ in.}) = 0.385 \text{ in.}$$

$$\Delta_{CR+SH} = 0.081 \text{ in.} + 0.385 \text{ in.} / 2 = 0.27 \text{ in.}$$

The total superstructure movement due to temperature, shrinkage, and creep is then:

$$\Delta_s = 0.38 + 0.27 = 0.65 \text{ in.}$$

Preliminary Bearing Configuration

Try a 10 in. x 22 in. bearing pad.

$$R_T = R_{DL \text{ Total}} + R_{LL \text{ Total}} = 75.2 + 59.33 = 134.5 \text{ kips}$$

Assume a preliminary bearing configuration consisting of three ½ in. internal elastomer layers, with 5/16 in. exterior layers and four 1/8 in. metal shims. (For steel-reinforced elastomeric bearings the internal layers shall be the same thickness, and the cover layers shall be no more than 70 percent of the thickness of internal layers. (14.7.6.1))

Check Stresses and Deformations

Compressive Stress (14.7.6.3.2):

The following two equations limit compressive stress:

$$\sigma_s \leq 1.25 \text{ ksi}$$

$$\sigma_s \leq 1.25GS_i$$

Where:

$\sigma_s$  = service average compressive stress due to total load (ksi)

G = shear modulus of the elastomer = 0.170 ksi

$S_i$  = shape factor of internal layer of an elastomeric bearing (14.7.5.1)

$$S_i = \frac{LW}{2h(L+W)} = \frac{10 \times 22}{2 \times 0.5 \times (10 + 22)} = 6.88$$

$$\sigma_s = \frac{R_T}{A} = \frac{134.5}{10 \times 22} = 0.61 \text{ ksi}$$

$$\sigma_s \leq 1.25 \text{ ksi} \quad \text{OK}$$

$$\sigma_s \leq 1.25 \times 0.170 \times 6.88 = 1.46 \text{ ksi} \quad \text{OK}$$

Compressive Deflection (14.7.6.3.3):

Where bridge joints and seals are used above a bearing area, limiting instantaneous (live load) deflections is necessary to prevent damage. For this example there are no joints on the bridge. Therefore, this effect will not be checked.

Shear Deformations (14.7.6.3.4):

To prevent rollover and fatigue problems, bearing pad deformation due to calculated movement ( $\Delta_s$  above) is limited to half of the total elastomer thickness, or

$$h_{rt} \geq 2\Delta_s$$

$$h_{rt} = 2 \times 0.3125 + 3 \times 0.50 = 2.125 \quad \geq \quad 2 \times 0.65 = 1.30 \text{ in.} \quad \text{OK}$$

Rotation (14.7.6.3.5d)

Rotation about transverse axis:

$$\sigma_s \geq 0.5GS \left( \frac{L}{h_{ri}} \right)^2 \frac{\theta_{s,x}}{n}$$

where:

$L$  = length of the rectangular elastomeric bearing (parallel to longitudinal bridge axis)(in.) = 10 in.

$h_{ri}$  = thickness of the  $i^{\text{th}}$  elastomer layer (in.) = 0.5

$\theta_{s,x}$  = Maximum service rotation about transverse axis.

Rotations are calculated assuming the beam deflects in a parabolic shape. As shown below, the final beam camber is 0.00472 rad.

Subtracting the rotation due to dead plus live load and adding an allowance of 0.005 rad to account for uncertainties, the total design rotation is 0.00725 rad. Since this rotation is less than 0.01 rad, a tapered sole plate is not required. (See *LRFD Specification* Section 14.8.2)

$$\theta_{\text{camber}} = \frac{2\Delta_{\text{camber}}}{0.5L_{\text{span}}} = \frac{2(0.622'')(1/12)}{0.5(43.917')} = 0.00472 \text{ rad}$$

$$\theta_{\text{LL}} = \frac{2\Delta_{\text{LL}}}{0.5L_{\text{span}}} = \frac{2(0.065'')(1/12)}{0.5(43.917')} = 0.00049 \text{ rad}$$

$$\theta_{\text{DL}} = \frac{2\Delta_{\text{DL}}}{0.5L_{\text{span}}} = \frac{2(0.259'')(1/12)}{0.5(43.917')} = 0.00198 \text{ rad}$$

where :  $\Delta_{\text{camber}}$  = final beam camber at center span from CONSPAN

$\Delta_{\text{LL}}$  = live load deflection at center span from CONSPAN

$\Delta_{\text{DL}}$  = dead load deflection at center span from CONSPAN

$N$  = number of interior layers of elastomer = 3

$H_{rt}$  = total elastomer thickness =  $3 \times 0.5 + 2 \times 0.25 = 2.125$  in.

$$\sigma_s = 0.61 \text{ ksi} \geq 0.5 \times 0.170 \times 6.88 \left( \frac{10}{0.5} \right)^2 \times \frac{0.00725}{3} = 0.57 \text{ ksi} \quad \text{OK}$$



Stability (14.7.6.3.6)

The only requirements for stability are that the total pad thickness does not exceed 1/3 of the pad length or width. By inspection, this requirement is satisfied.

Bearing Reinforcement (14.7.6.3.7)

Reinforcement thickness is subject to the following two conditions based on service stresses and fatigue:

$$1) \quad h_s > \frac{3.0h_{ri}\sigma_s}{F_y} = \frac{3.0 \times 0.5 \times 0.61}{36} = 0.025 \quad h_s = 0.125 \text{ in. OK}$$

$$2) \quad h_s > \frac{2.0h_{ri}\sigma_L}{\Delta F_{TH}} = \frac{2.0 \times 0.5 \times 0.27}{24} = 0.011 \quad h_s = 0.125 \text{ in. OK}$$

where:

$$\sigma_L = \frac{R_L}{A} = \frac{59.33}{10 \times 22} = 0.27 \text{ ksi}$$

$\Delta F_{TH}$  = Allowable fatigue stress range for over 2,000,000 cycles and  
Category A details = 24 ksi (Table 6.6.1.2.5-3)

The *Design Guide* states that the Department requires that the thickness of the laminate steel reinforcement layers (sheet metal shims) be specified as 1/8 in. and conform to ASTM A1008 or A1011.

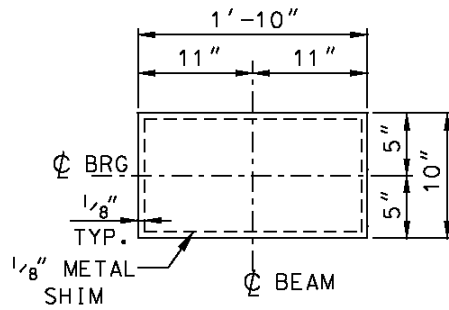
Bearing pad design at the pier is similar to the design above. However, longitudinal movement does not occur at the fixed pier. This prevents shear deformations in the pads, and, as a result, AASHTO allows a 10% increase in allowable stresses.

In certain situations, plain (unreinforced) elastomeric pads can be designed for the fixed bearings. Plain pads are considerably cheaper than reinforced pads. However, the thickness of plain pads is limited to 3/4 in. For this example, 3/4 in. lacks sufficient rotation capacity. Therefore, a reinforced design is used.

Based on the same design steps shown above for the abutment, the same bearing pad is satisfactory for span 1 at the fixed pier bearing. For span 2, the fixed pier bearings will need to be designed with the methods shown in this example using the span 2 reactions and rotations.

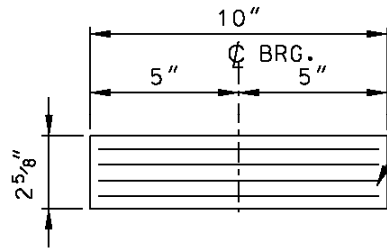
Final details for the bearing pads are shown.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**



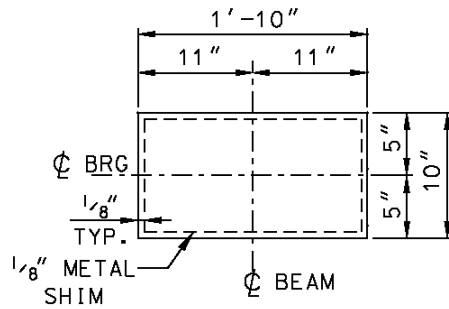
**DESIGN LOAD**

DL	75.2 k
LL	59.33 k



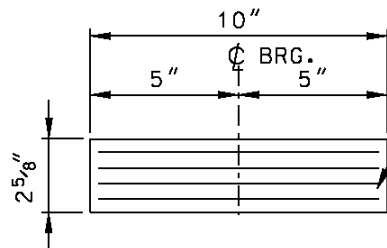
ELASTOMERIC BEARING PAD  
0.170 KSI SHEAR MODULUS BEARING  
1'-10" X 10" X 2 3/8"  
(5) LAMINATIONS  
(5/16" EXT., 1/2" INT.)  
(4) 1/8" METAL SHIM

**EXPANSION BEARING**



**DESIGN LOAD**

DL	63.4 k
LL	71.48 k



ELASTOMERIC BEARING PAD  
0.170 KSI SHEAR MODULUS BEARING  
1'-10" X 10" X 2 3/8"  
(5) LAMINATIONS  
(5/16" EXT., 1/2" INT.)  
(4) 1/8" METAL SHIM

**FIXED BEARING**

**ELASTOMERIC BEARING PAD DETAIL  
(SPANS 1 AND 3)**

## **5. SUBSTRUCTURE AND FOUNDATION DESIGN**

Substructure and foundation design includes design of the piers and abutments. For this example the program RC-PIER, Version 09.00.03.01, will be used as the primary design aide. Refer to the preliminary design sections of this document for details of the preliminary pier and abutment configurations.

The pier is a three-column bent with circular columns that will frame directly into supporting drilled shafts. Piers 1 and 2 bearings are fixed against longitudinal movement.

The abutment is a semi-integral, floating type that is also supported by three drilled shafts. The superstructure is free to move longitudinally on the abutment cap.

Since all longitudinal loads will be resisted by Piers 1 and 2, Pier 1 will serve to illustrate the design process for this example. Since the process of abutment design is quite similar to that of a pier, an abutment design will not be completed. The design of an abutment, however, will be discussed with an emphasis on the differences between it and a pier design.

Substructure design is an iterative process that requires the designer to calculate initial loads based on an assumed point of moment fixity for the column/shaft system. Design moments found using fixed end moment can be excessive. Therefore, the loads resulting from an assumed point of fixity are used by the foundation engineer to determine shaft length and run a lateral load analysis. The structural engineer can then compare the location of the point of maximum moment in the shaft obtained from the lateral load analysis to the point of fixity he has assumed and, if they are substantially different, adjust the design model accordingly. Steps in the process are as follows:

- 1) Discuss site conditions and requirements with the Foundation engineer, agree upon a workable foundation type, and estimate foundation depth.
- 2) Calculate total factored loadings for each pier and abutment location based upon an assumed point of shaft fixity. Submit the loads to the foundation engineer for the final foundation report.
- 3) Obtain a final foundation report and recommendations including a capacity chart for the foundation system and a lateral load analysis.
- 4) If necessary, adjust the structural model to more accurately determine forces based on the point of maximum moment obtained from the lateral load analysis.

In addition to the preliminary and final foundation reports, stream flow data and the foundation drill logs will be needed. The stream flow information is obtained from the drainage report. For this bridge, stream flow data are as follows:

V100= 10 ft./sec.  
D100 = 18 ft.  
Scour (100) = 10 ft.

V500 = 12 ft./sec.  
D500 = 15 ft.  
Scour (500) = 15 ft.

A foundation investigation has not been completed for this bridge. Values will be assumed for this design.

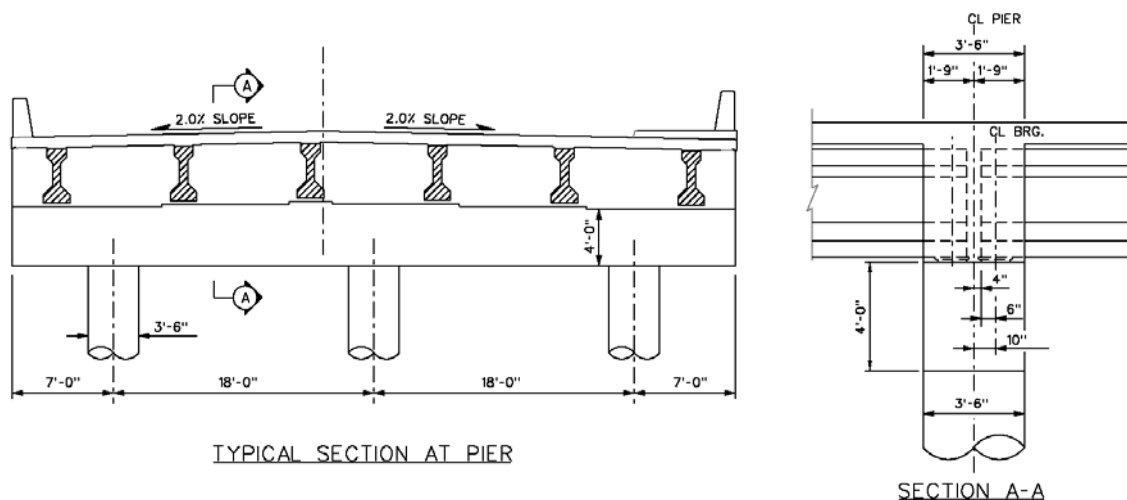
## 5.1 Preliminary Pier Design

This example will follow RC-PIER step-by-step through the design process. For each step in the process the appropriate screen from RC-PIER will be presented followed by any explanation necessary. In most cases, loads and designs are calculated automatically by the program upon command. In those instances where outside calculation is needed those calculations are presented.

To complete the design three separate runs of RC-PIER will be needed: one run to evaluate all loads exclusive of extreme event load cases, a separate run to assist in the seismic evaluation, and a final run to evaluate the 500 year flood extreme event case.

### 5.1.1 Design Run Excluding Extreme Events

The preliminary pier configuration is shown below.



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

To start RC-PIER, open the previously run LEAP Bridge file containing the design for the superstructure. Opening RC-PIER this way instead of creating a completely new file will be advantageous as the autogenerate function will be enabled.

After opening LEAP Bridge, the following screen will appear.

LEAP Bridge V8i (SELECTseries 1) - LRFD Example 3 Span PC Girder Final with LL Defl.xml

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

Project Name: LRFD Design Example, 3 Span PCI Girder

Project Number: CN1234

State: NMDOT Job Number: CN1234

Designed by: PB Date: 1/ 5/2011

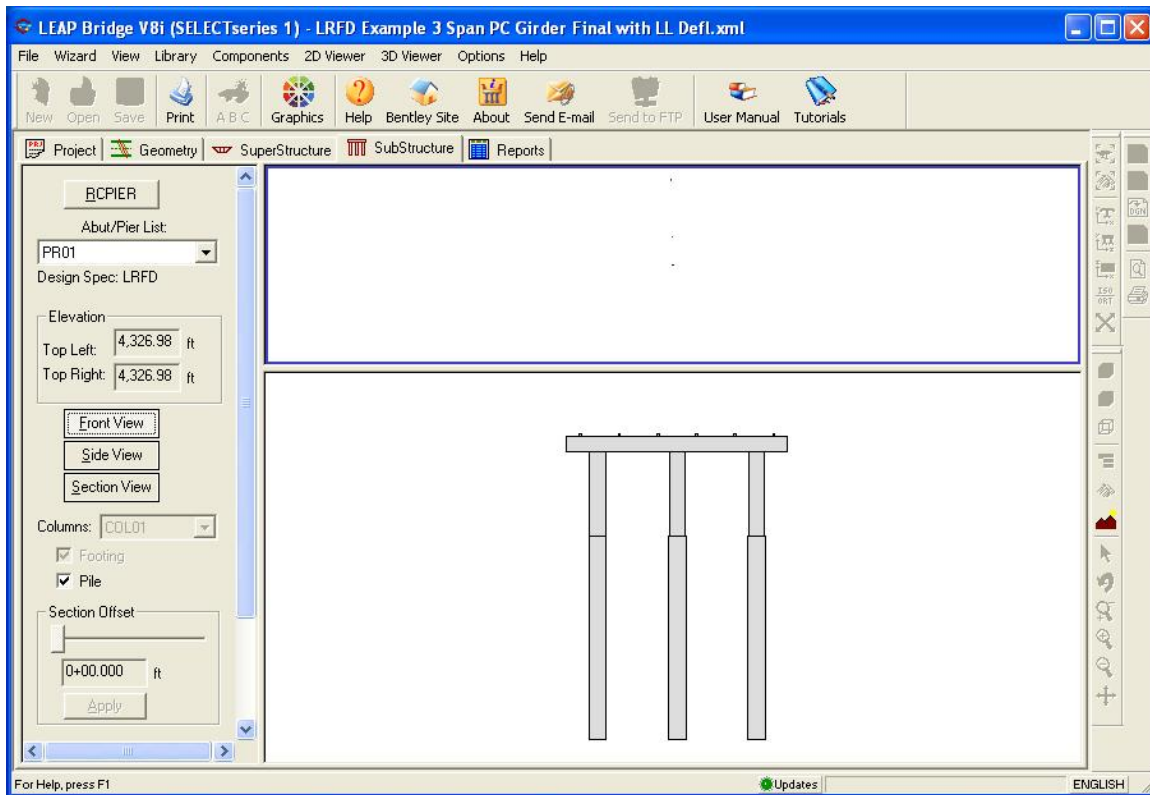
Checked by: Date:

Description: LRFD Design Example, 3 Span PCI Girder

RCPIer component started... Updates ENGLISH

Ensure that the screen is filled out as shown above, and click the substructure tab. The following screen will appear. When the screen is first opened, it will be blank until some essential geometry data are entered in later screens.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications



Click RC-PIER and the following screen will appear.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

The screenshot shows the RCPier V8i software interface. The title bar indicates the file path: G:\Bridge On-Call\LRFD Design Examples\Report\Design Example Rpt March 2011- Substr Update\LRFD Example ... The menu bar includes File, Show, Libraries, LEAP Bridge, and Help. The toolbar contains icons for 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, and Tutorials. The main window has tabs for Project, Geometry, Loads, Analysis, Cap, Column, and Footing. The Project tab is active, displaying the Project Information section. The Project Name is "LRFD Design Example, 3 Span PCI Girder". The User Job Number is "CN1234". The State is "NMDOT". The Date is "Jan/5/2011". The State Job Number is "CN1234". The By is "PB". The Date Checked is empty. The Checked By is empty. The Description is "LRFD Design Example, 3 Span PCI Girder". At the bottom, there are three sections: Structure type with radio buttons for Pier (selected) and Abutment; Design Specifications with radio buttons for AASHTO Standard and AASHTO LRFD (selected), and a State Specifications dropdown menu set to None; and Units with radio buttons for U.S. Units (selected) and SI Units (Metric). The status bar at the bottom shows Updates, IBS: Connected, IBS: Manual, Pier View: Upstation, and NUM.

Project Information

Project Name: LRFD Design Example, 3 Span PCI Girder

User Job Number: CN1234

State: NMDOT

Date: Jan/5/2011

Date Checked:

State Job Number: CN1234

By: PB

Checked By:

Description: LRFD Design Example, 3 Span PCI Girder

Structure type

☒ Pier

☐ Abutment

Design Specifications

☐ AASHTO Standard

☒ AASHTO LRFD

State Specifications

None

Units

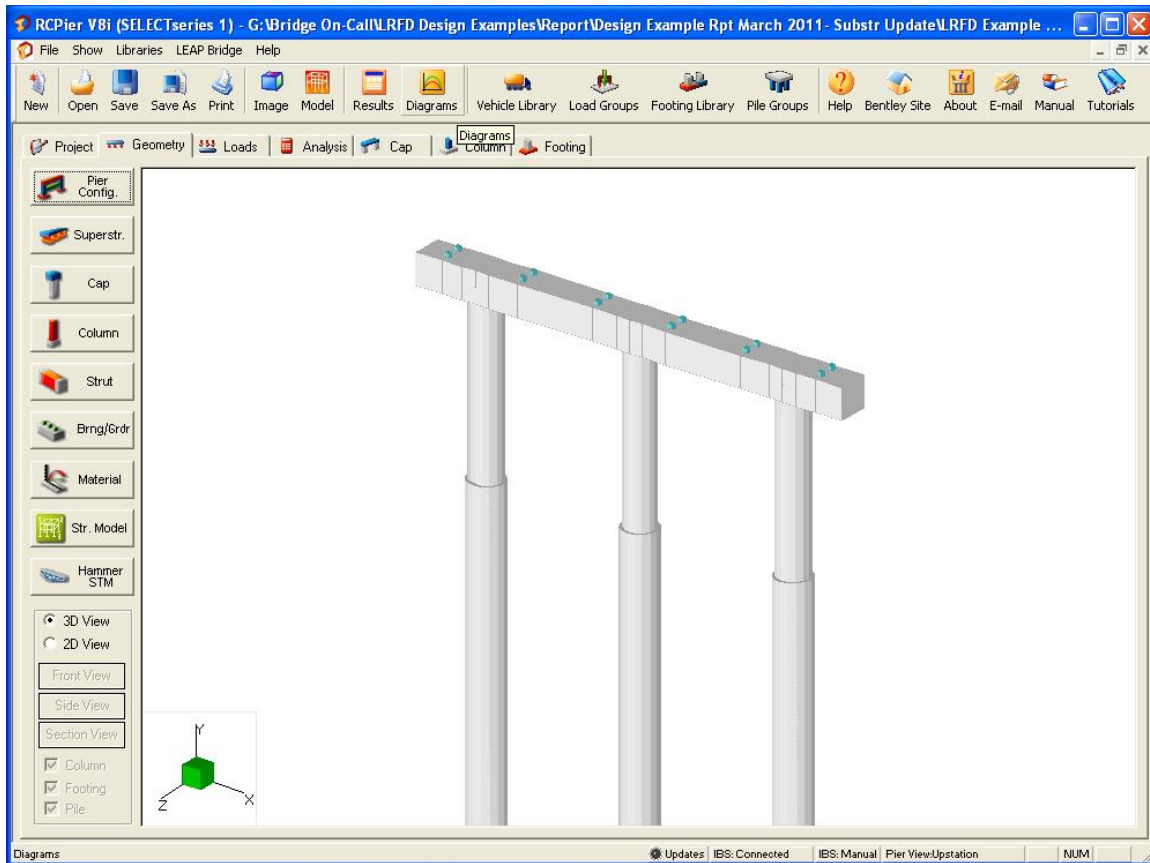
☒ U.S. Units

☐ SI Units (Metric)

Fill in the appropriate information if it is not already present, and click the Geometry tab.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

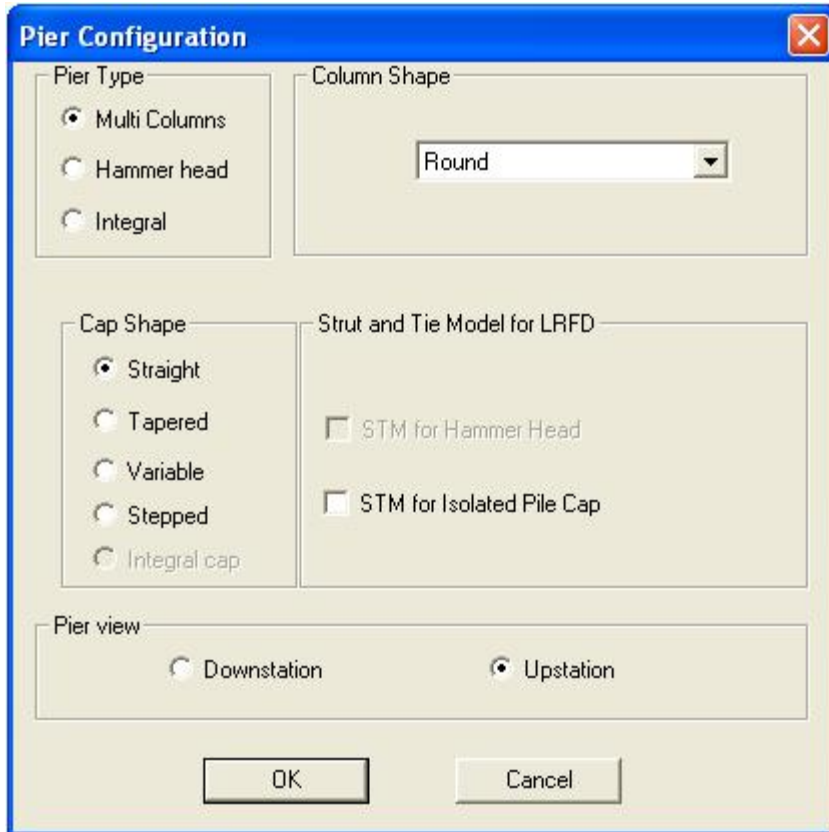
The Geometry screen is shown below. When it is first opened, the screen will not show the columns. The program will build the pier details after the geometry information is input.



Click the Pier Config icon and the following screen will appear.



**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**



The image shows a 'Pier Configuration' dialog box with a blue title bar and a close button. It contains several sections: 'Pier Type' with radio buttons for 'Multi Columns' (selected), 'Hammer head', and 'Integral'; 'Column Shape' with a dropdown menu set to 'Round'; 'Cap Shape' with radio buttons for 'Straight' (selected), 'Tapered', 'Variable', 'Stepped', and 'Integral cap'; 'Strut and Tie Model for LRFD' with checkboxes for 'STM for Hammer Head' and 'STM for Isolated Pile Cap' (both unchecked); and 'Pier view' with radio buttons for 'Downstation' and 'Upstation' (selected). At the bottom are 'OK' and 'Cancel' buttons.

Complete the screen as shown above. For our example we will have multi-column round piers. The cap is straight and we want to look at the pier upstation. After the information is input click OK.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

This will bring you back to the Geometry screen. Click on the Superstr. icon to bring up the following screen.

**Superstructure Parameters**

Number of Lanes: 3

Beam Height: 45. in

Beam Section Area: 560. in<sup>2</sup>

Beam Inertia (I<sub>xx</sub>): 125390 in<sup>4</sup>

Beam Inertia (I<sub>yy</sub>): 12216.6 in<sup>4</sup>

Beam C.G (Y<sub>cg</sub>): 20.27 in

Barrier/Railing Height: 42. in

Depth of Slab: 9. in

Span Number Rear to Current Pier: 1

Curb to Curb Distance: 47. ft

☒ Auto compute geometry by girders

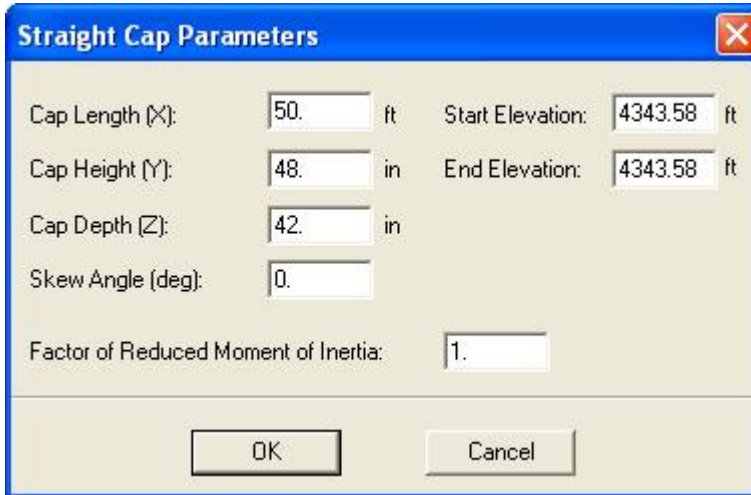
Span #	Span Length ft	Bridge Width ft
1	43.750	50.000
2	88.000	50.000
3	43.750	50.000
End Bridge	-	50.000

Buttons: Add, Modify, Delete

If you started RC-PIER from the LEAP Bridge file, this should already be filled out. If not, complete the screen as shown. Click OK.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

This will bring you back to the Geometry screen. Click on the Cap icon, which brings up the following screen.



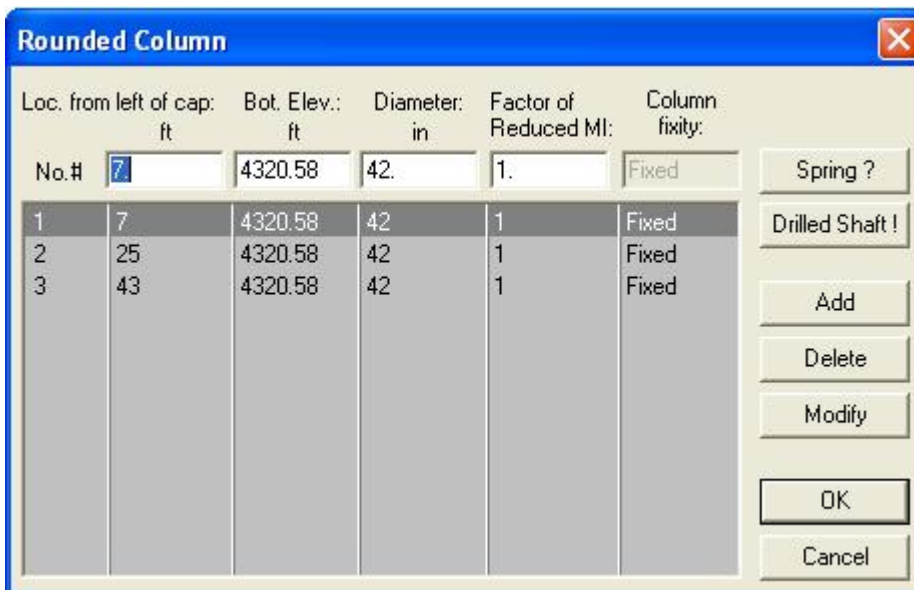
The "Straight Cap Parameters" dialog box contains the following fields:

- Cap Length (X): 50. ft
- Start Elevation: 4343.58 ft
- Cap Height (Y): 48. in
- End Elevation: 4343.58 ft
- Cap Depth (Z): 42. in
- Skew Angle (deg): 0.
- Factor of Reduced Moment of Inertia: 1.

Buttons: OK, Cancel

Information appropriate to our example has been entered. "Start" and "End" elevations are at the left and right side at the top of the cap, respectively. In the CONSPAN model for this bridge, the skew angle was input as zero. We will match that skew for the pier design.

Click OK and return to the Geometry screen. Click on the Column icon to bring up the Rounded Column screen.



The "Rounded Column" dialog box contains a table with the following data:

No. #	Loc. from left of cap: ft	Bot. Elev.: ft	Diameter: in	Factor of Reduced MI:	Column fixity:
1	7	4320.58	42	1	Fixed
2	25	4320.58	42	1	Fixed
3	43	4320.58	42	1	Fixed

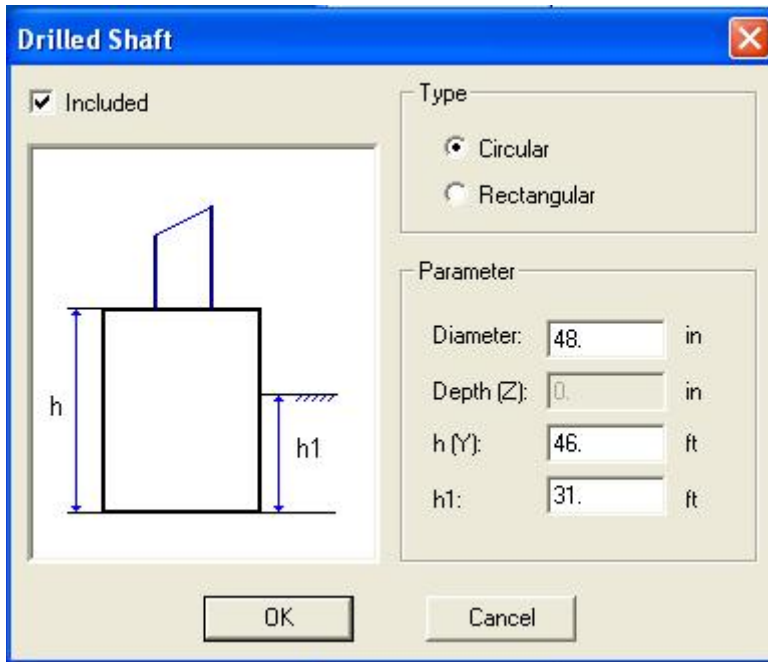
Buttons: Spring ?, Drilled Shaft !, Add, Delete, Modify, OK, Cancel

Fill in the appropriate information for each column in the bent. The first column is located 7 ft. from the left end of the cap. The cap is 4 ft. deep, and the column length is 19.0 ft. resulting in a bottom elevation of 4320.58 ft. The bottom of the column is rigidly

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

fixed to the drilled shaft. After completing all the information for column 1, click the Add button and fill in the appropriate information for each additional column.

After inputting all column information, click the Drilled Shaft button to bring up the following screen.



The image shows a software dialog box titled "Drilled Shaft". It has a blue title bar with a close button (X) in the top right corner. Inside the dialog, there is a checkbox labeled "Included" which is checked. To the right of the checkbox is a "Type" section with two radio buttons: "Circular" (selected) and "Rectangular". Below the "Type" section is a "Parameter" section with four input fields: "Diameter:" with the value "48." and unit "in"; "Depth (Z):" with the value "0." and unit "in"; "h (Y):" with the value "46." and unit "ft"; and "h1:" with the value "31." and unit "ft". On the left side of the dialog, there is a diagram of a rectangular column on a drilled shaft. The column is labeled with height "h" and the shaft is labeled with depth "h1". The shaft is shown as a rectangle extending downwards from the base of the column. At the bottom of the dialog are two buttons: "OK" and "Cancel".

The drilled shaft diameter is 48 in. For preliminary design we have assumed a depth of 46 ft. The 100-year scour is 10 ft., and we have assumed a depth of fixity (location of maximum moment) at 5 ft. below scour. This puts the point of fixity,  $h_1$ , at 31 ft. After input is complete, click OK. This will return you to the Column screen. Complete the drilled shaft input for each column and click OK on the Column screen to return to the Geometry screen.

Click the Brng/Grdr icon to bring up the screen shown below.

**Bearing / Girders**

Configuration  
Bearing Line: ☐ Single ☒ Double

Eccentricity from CL of Cap  
First Line: 0.83333 ft Second Line: -0.83333 ft

Line  
☒ First  
☐ Second

Distance From  
☒ Cap Left End  
☐ Last Point 0. ft

Line	Point	From	Dist./Abs. Dist.
1	1	Left	3.12/3.12
	2	Left	11.87/11.87
	3	Left	20.62/20.62
	4	Left	29.37/29.37
	5	Left	38.12/38.12
	6	Left	46.87/46.87
2	1	Left	3.12/3.12
	2	Left	11.87/11.87
	3	Left	20.62/20.62

Add  
Delete  
Modify  
OK  
Cancel

Since this RC-PIER run is linked to a CONSPAN run, the information on bearing location is input automatically. Click OK to return to the Geometry screen.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

In the Geometry screen, click on the Material icon to bring up the Materials screen.

The Materials dialog box is a window with a blue title bar and a close button (X) in the top right corner. It contains several input fields and buttons. The fields are organized into three main sections: Concrete Strength, Concrete Density, and Concrete Modulus of Elasticity, each with three rows for Cap, Column, and Footing. Below these are Steel Yield Strength and Concrete Type sections. The Steel Yield Strength section has four rows for Cap (flex), Cap (shear), Column, and Footing. The Concrete Type section has three rows for Cap, Column, and Footing, each with a dropdown menu. The OK and Cancel buttons are located at the bottom right of the dialog box.

Concrete Strength	Concrete Density	Concrete Modulus of Elasticity
psi	pcf	ksi
Cap: 3000.	Cap: 150.	Cap: 3320.56
Column: 3000.	Column: 150.	Column: 3320.56
Footing: 3000.	Footing: 150.	Footing: 3320.56

Steel Yield Strength	Concrete Type
ksi	
Cap ( flex): 60.	Cap: Normal
Cap ( shear): 60	Column: Normal
Column: 60.	Footing: Normal
Footing: 60.	

OK  
Cancel

Input the appropriate concrete ultimate and steel yield strengths. Standard NMDOT strengths are 3000 psi for concrete and 60 ksi for steel. Concrete modulus of elasticity is calculated by the program. Click OK and return to the Geometry screen.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

In the Geometry screen, click on the Str. Model icon to bring up the Structure Model screen. Ensure that the radio buttons under Cap design are selected as shown below and click OK to return to the Geometry screen.

The Structure Model dialog box contains the following sections:

- Objects:** Cap
- Components:** 01
- Table:**

Member	Node	Hinge	Check Point	Distance (ft)	Elem Length (ft)
7	10	-		0.00	
	11	-		3.12	3.12
8	11	-		3.12	
	12	-	f	5.45	2.33
9	12	-	f	5.45	
	3	-		7.00	1.55
10	3	-		7.00	

**Additional Check Points**

☐ Add default check points

0. ft From Left:

Buttons: Add, Delete, Modify, (De)Activate

**Reset to Base Structure**

Reset All

**Hinge**

Local Direction: ☐ Z

**Cap design**

**Flexure**

- ☒ Centerline of column
- ☐ Face of support
- ☐ Offset from CL of the column [ ] ft

**Shear**

- ☐ Centerline of column
- ☒ Face of support
- ☐ Offset from CL of the column [ ] ft

**Plastic Hinge locations**

**Near Column Top**

- ☒ Cap Column joint
- ☐ At Cap Soffit
- ☐ Below Cap Soffit [0.] ft

**Near Column Bottom**

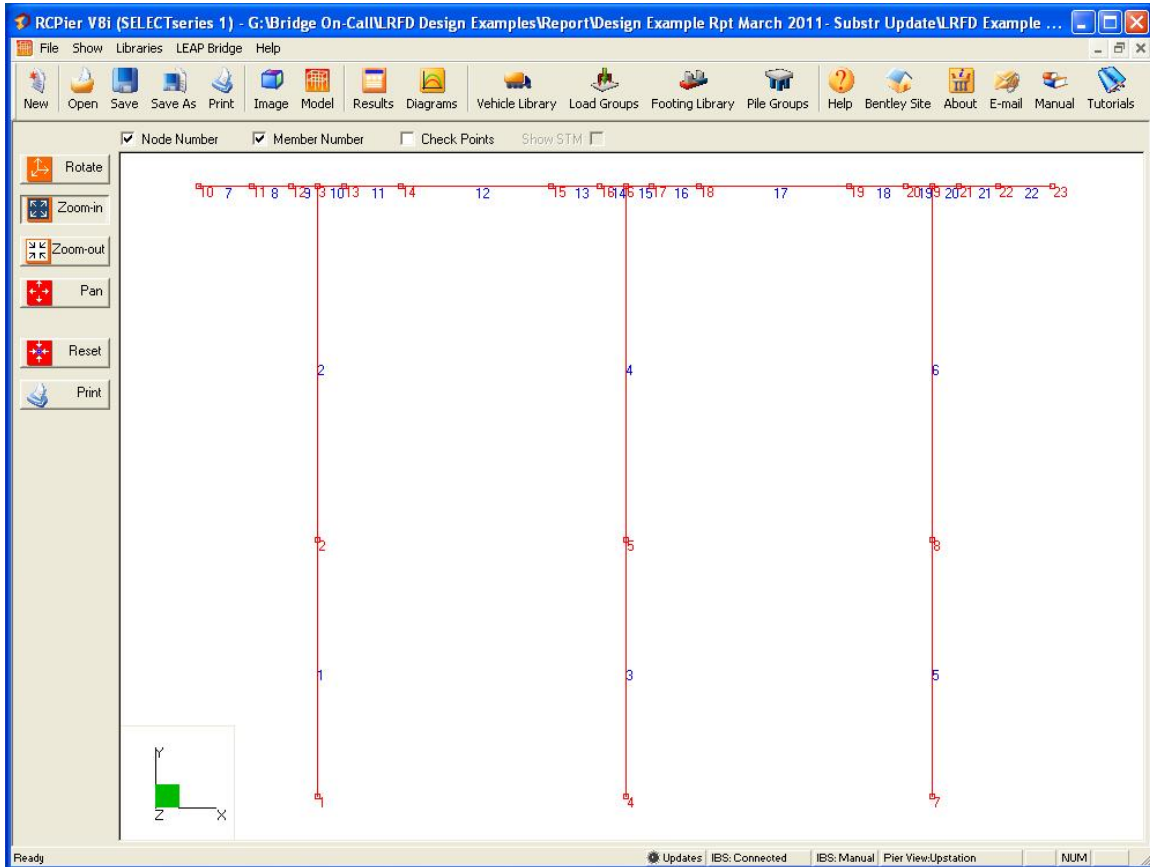
- ☒ At Column Base
- ☐ Above Column Base [0.] ft

Buttons: OK, Cancel

This completes the geometry input.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

To view node and member numbers, click on the Model icon in the top toolbar, and the following screen will appear.

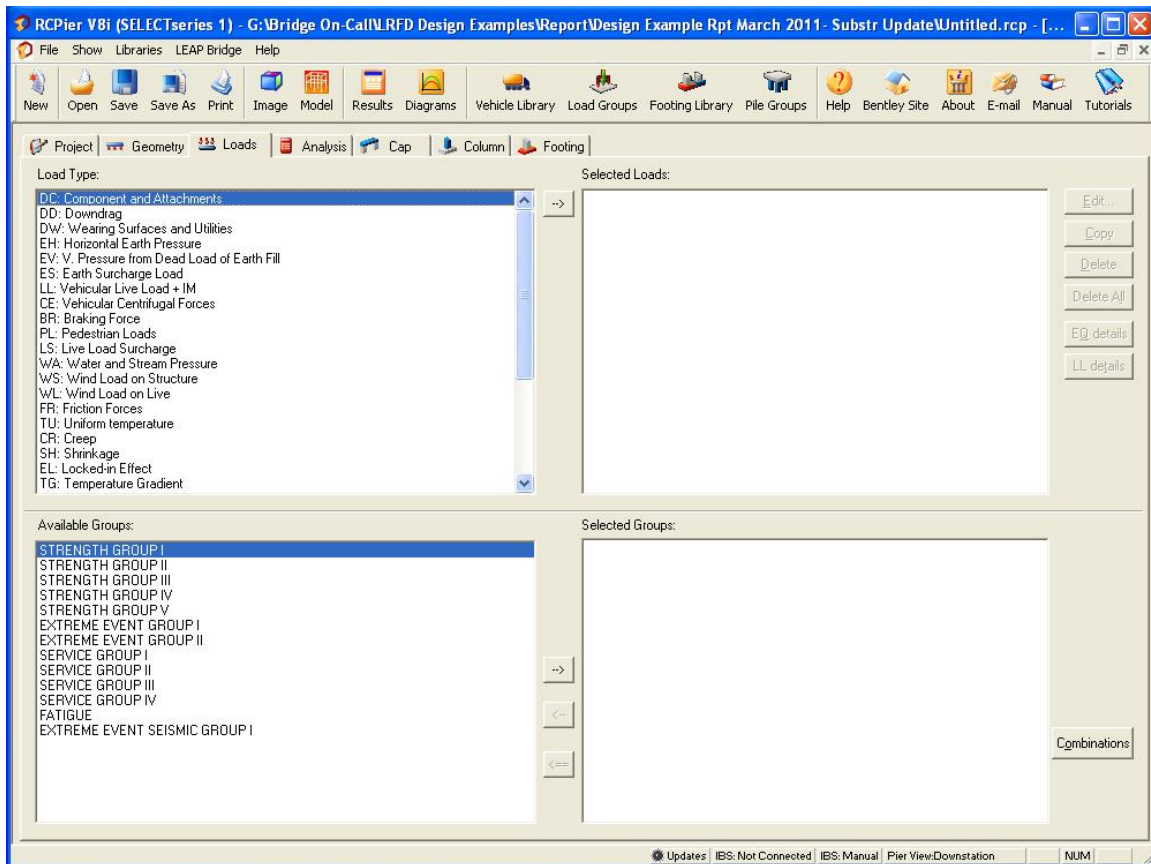


Once in the screen, click the “Node Number” and “Member Number” check boxes to display the numbers.

Return to the Geometry screen by closing the Model window. Once back in the Geometry screen, click the Loads tab to bring up the Loads screen. The Loads screen is shown below.



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

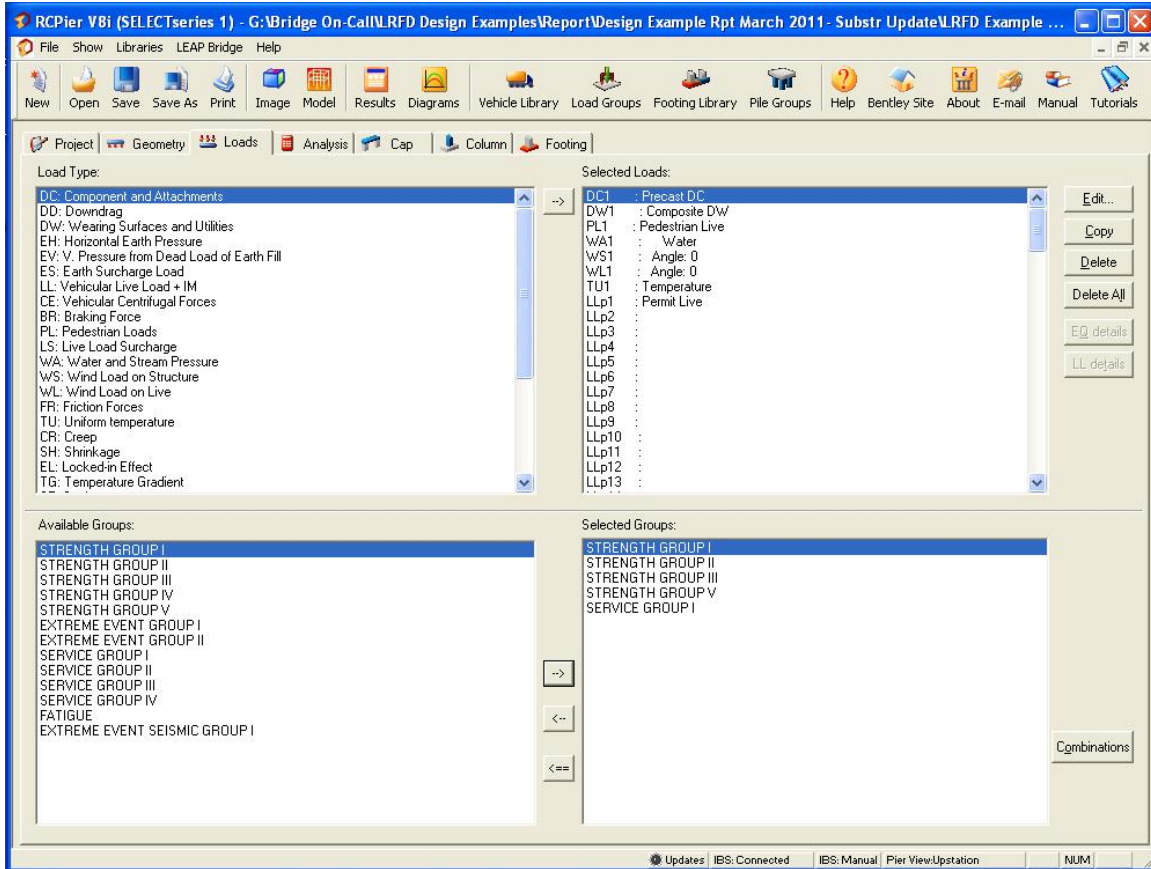


The upper left-hand box in this screen contains a list of all AASHTO LRFD loads. Highlight the load and click the horizontal arrow to move it to the right into the Selected Loads box. For Pier 1 in this example, applicable loads are DC, DW, LL, LLp, BR, PL, WA, WS, WL, and TU. EQ will not be applied in this first run. Highlight and move these loads into to the Selected Loads box.

All AASHTO load cases are shown in the lower left-hand box. Referring to Article 3.4.1 of the *LRFD Specifications*, it is determined that load cases applicable to this pier are Strength I, Strength II, Strength III, Strength V, and Service I. Extreme Event I for the 500-year flood and Extreme Event Seismic Group I will not be used for this first run. Those cases will be checked in the next two runs. Highlight the applicable cases and move them to the Selected Groups box. The completed screen is shown below.

Note that it is not necessary to combine the 100-year local and contraction scour with the Strength II (permit) load case, since the 100-year storm is a rare, short-duration event. For convenience in this example, the 100-year scour is assumed to be concurrent with the Strength II loading, which is conservative.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications



To enter loads or to have RC-PIER calculate loads, highlight the desired load in the Selected Loads box, and click the Edit button on the right side of the screen. For example, highlight DC1 and click on Edit and bring up the following screen.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

**Loads: Load data**

Bearing / Girder loads			
Bearing Line	Bearing Point#	Dir	Load (kips)
*			

Insert Copy Delete Delete All

Column Loads / Settlement							
Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units

Insert Copy Delete Delete All

Cap Loads							
Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units

Insert Copy Delete Delete All

Strain Load  
Unit: 0.  
+ Expansion - Contraction

Name: DC1  
Description:

Factors  
Multiplier for Loads: 1.

Auto Generation  
Generate

Import Loads  
Import

Note: Vertically downward loads be added as negative loads in Y direction.

OK Cancel

To have RC-PIER automatically generate the DC1 loads, click the Generate button located in the lower right hand corner of the screen to bring up the following screen.

**Auto Load Generation: Structure DC**

**Superstructure**

☐ Include Slab Unit Weight: 150 pcf

☐ Include Girders Unit Weight: 150 pcf

☒ Use simple span load distribution for barrier and wearing surface

☒ Include Barriers Total Load per foot: 418 plf

☐ Include Wearing Surface Load per foot: 0 plf  
(in longitudinal dir)

☐ Use Continuous Bridge Model to compute dead load reactions

☐ Input composite dead load reaction

Composite dead load reaction: 0 kips

☒ Input composite dead load reaction from Conspan

Import Selected Pier: 0

Reaction distribution among bearing lines:

Bearing Line 1: 0.5 Bearing Line 2: 0.5

Generate Cancel

Since this model is connected to our CONSPAN run through LEAP Bridge, we can import our superstructure dead loads. Select Input composite dead load reaction from CONSPAN and click the Import button. (If your model is not connected to your CONSPAN run, you can export your superstructure loads to a .txt file from the File menu in CONSPAN and then navigate to that file from the screen that appears when you click the Import button.)

Clicking the Generate button yields these loads from CONSPAN. However, these loads do not include pier or midspan diaphragms. These can be calculated by hand and then added to the automatically generated loads shown on the Loads screen.

**Loads:**

---

Bearing Line	Bearing Point#	Dir	Load (kips)
1	1	Y	-51.4940
1	2	Y	-55.9440
1	3	Y	-55.9440
1	4	Y	-55.9440
1	5	Y	-55.9440
1	6	Y	-51.4940
2	1	Y	-88.5511
2	2	Y	-96.0538
2	3	Y	-96.0538
2	4	Y	-96.0538
2	5	Y	-96.0538

Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units
--------	-----------	-----	------	------	------	------	-------

Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units
-----------	-----	----------	------	------	------	------	-------

Name: DC1 Precast DC  
Description: Composite and Non-composite DC

Note: Vertically downward loads be added as negative loads in Y direction.

**Parsons Brinckerhoff**

In a like manner, input DW1 loads. The Auto Load Generation screen for DW1 is shown below.

**Auto Load Generation: Structure DW**

**Superstructure**

☐ Include Slab Unit Weight: 150 pcf

☐ Include Girders Unit Weight: 150 pcf

☐ Use simple span load distribution for barrier and wearing surface

☐ Include Barriers Total Load per foot: 418 plf

☐ Include Wearing Surface Load per foot: 418 plf (in longitudinal dir)

☐ Use Continuous Bridge Model to compute dead load reactions

☐ Input composite dead load reaction

Composite dead load reaction: 0 kips

☒ Input composite dead load reaction from Conspan

Import Selected Pier: 0

Reaction distribution among bearing lines:

Bearing Line 1: 0.5 Bearing Line 2: 0.5

Generate Cancel

As with the DC loads, import the wearing surface load from CONSPAN.

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

Moving on to Live Load, the Auto Load Generation screen for Live Load is shown below.

**Auto Load generation: Live Load**

**Longitudinal Reaction**

☐ Compute Simple Span Reaction

Available:

- Design Truck
- Design Truck + Lane Load
- Design Tandem + Lane Load
- Two Design Trucks + Lane Load
- Two Design Tandem + Lane Load
- Fatigue Truck
- [PV]P-5 Truck

Selected:

- Design Truck + Lane Load
- Two Design Trucks + Lane Load

☐ Compute Continuous Beam Reaction

Normal pier

Max Truck Load: 0 kips

Max Lane Load: 0 kips

☐ Input Already Computed Reaction

☐ Import Conspan Reaction

Import:

**Integral pier**

	Max Load	Moment, k-ft		Load, kips	Max. Moment
Truck:			Truck:		
Lane:			Lane:		

**Reaction distribution among bearing lines**

	Bearing Line 1	Bearing Line 2
Truck Case A:	0.5	0.5
Lane Case A:	0.5	0.5
Truck Case B:	1	0
Lane Case B:	0.5	0

☐ Generate Reverse Cases also

**Transverse Positioning**

Loaded Lanes: All combinations

**Live Load Positions**

☒ Variable spacing

Minimum spacing between positions: 1 ft

☐ Constant spacing

Minimum distance from curb: 2 ft

Center to center spacing: 10 ft

**Longitudinal Force**

☐ Generate Longitudinal Load Cases also

☒ Auto Compute ☐ Manual input

Truck Load: 0 kips

Lane Load: 0 kips

**Centrifugal Force**

☐ Generate Centrifugal Load Cases also

☒ Auto Compute ☐ Manual input

Truck Load: 0 kips

Radius of curve: 0 ft

Design speed: 0 ft/s

Direction of centrifugal force: ☒ + (X) ☐ - (X)

**Generate** **Cancel**

In accordance with the *LRFD Specification* Article 3.6.1.3.1, applicable live loads are Design Truck + Lane, Design Tandem + Lane, and Design Two Trucks + Lane. Experience has shown that the design tandem will not govern for spans over about 30 ft., so that load was not included. Again, clicking Generate on the Auto Load Generation screen and OK on the Load screen applies the loads.



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

Permit Live Load must be entered separately. The Auto Load Generation screen for Permit Live Load is shown below.

**Auto Load generation: Live Load**

**Longitudinal Reaction**

☒ Compute Simple Span Reaction  
Available: [PV]P-5 Truck, [PV]P-7 Truck, [PV]P-9 Truck, [PV]P-11 Truck, [PV]P-13 Truck, [PV]P-15 Truck, [PV]P327-13  
Selected: [PV]P327-13  
Buttons: Add -, <- Remove, <- Remove All, View

☐ Compute Continuous Beam Reaction  
Normal pier  
Max Truck Load: 0 kips  
Max Lane Load: 0 kips

☐ Input Already Computed Reaction

☐ Import Conspan Reaction  
Import

**Integral pier**

	Max Load	Moment, k-ft		Load, kips	Max. Moment
Truck:			Truck:		
Lane:			Lane:		

**Reaction distribution among bearing lines**

	Bearing Line 1	Bearing Line 2
Truck Case A:	0.5	0.5
Lane Case A:	0.5	0.5
Truck Case B:	1	0
Lane Case B:	0.5	0

☐ Generate Reverse Cases also

**Transverse Positioning**  
Loaded Lanes: All combinations

**Live Load Positions**  
☒ Variable spacing  
Minimum spacing between positions: 1. ft  
☐ Constant spacing  
Minimum distance from curb: 2. ft  
Center to center spacing: 10. ft

**Longitudinal Force**  
☐ Generate Longitudinal Load Cases also  
☒ Auto Compute ☐ Manual input  
Truck Load: 0 kips  
Lane Load: 0 kips

**Centrifugal Force**  
☐ Generate Centrifugal Load Cases also  
☒ Auto Compute ☐ Manual input  
Truck Load: 0 kips  
Radius of curve: 0 ft  
Design speed: 0 ft/s  
Direction of centrifugal force: ☒ +(\) ☐ -(\)

Generate Cancel

If you do not already have the P327-13 permit load defined in RC-PIER, you will have to add it to the library by clicking the Vehicle Library icon in the top toolbar. A diagram of the P327-13 permit load is provided in the *Design Guide*.



Braking loads can also be automatically generated. The Auto Load Generation screen for braking loads is shown below.

**Auto Load Generation: Braking Force (BR)**

Live Load

☒ Select Live Load

Available:

- Design Truck + Lane Load
- Design Tandem + Lane Load
- Two Design Trucks + Lane Load
- Two Design Tandem + Lane Load

Selected:

- Design Truck + Lane Load

☐ Manual input: Total Live Load

Truck load: 0 kips

Lane Load: 0 kips

Contributing Length: 87.75 ft

Number of Lanes Loaded: 2

Generate Cancel

The braking load should be calculated in accordance with *LRFD Specification* Section 3.6.4 as a percentage of the truck or truck + lane load and shall be applied in all loaded design lanes carrying traffic in the same direction. The bridge currently carries one lane in each direction, but it was assumed that it could be restriped in the future to carry an additional lane in one direction. Thus, the option of Truck + Lane Load was selected and applied to two lanes. Contributing length was taken as the length of Span 1 and the portion of Span 2 tributary to Pier 1:

$$\text{Contributing Length} = 43.75 \text{ ft.} + (88 \text{ ft.}/2) = 87.75 \text{ ft.}$$

It should also be noted here that, when generating braking loads, RC-PIER considers all axles of a truck in computing the load even if the span is shorter than the truck.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

The generated braking loads are shown below.

**Loads: Load data**

Bearing Line	Bearing Point#	Dir	Load (kips)
1	1	Z	-3.0000
1	2	Z	-3.0000
1	3	Z	-3.0000
1	4	Z	-3.0000
1	5	Z	-3.0000
1	6	Z	-3.0000
2	1	Z	-3.0000
2	2	Z	-3.0000
2	3	Z	-3.0000
2	4	Z	-3.0000
2	5	Z	-3.0000

Insert Copy Delete Delete All

Strain Load  
Unit: 0.  
+ Expansion - Contraction

Name: BR1 Braking  
Description: Braking Load

Factors  
Multiplier for Loads: 1.

Auto Generation  
Generate

Import Loads  
Import

Note: Vertically downward loads be added as negative loads in Y direction.

OK Cancel

Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units

Insert Copy Delete Delete All

Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units
Moment	X	0.0000	-378.0000	0.5000	0.0000	0.0000	k-ft
*							

Insert Copy Delete Delete All

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

RC-PIER does not auto generate pedestrian loads. The reaction from this load was calculated in accordance with *LRFD Specification* Section 3.6.1.6 as 0.075 ksf acting over the sidewalk width of 5.67 ft. and a tributary length of 65.875 ft. The calculated value is equal to 27.9 kips. This was equally divided among all 12 bearing points as 2.5 kips per bearing, rounding up to the next 0.5 kips. The completed load screen is shown below.

**Loads: Load data**

**Bearing / Girder loads**

Bearing Line	Bearing Point#	Dir	Load (kips)
1	1	Y	-2.5000
1	2	Y	-2.5000
1	3	Y	-2.5000
1	4	Y	-2.5000
1	5	Y	-2.5000
1	6	Y	-2.5000
2	1	Y	-2.5000
2	2	Y	-2.5000
2	3	Y	-2.5000
2	4	Y	-2.5000
2	5	Y	-2.5000

Insert Copy Delete Delete All

**Strain Load**

Unit: 0

+ Expansion - Contraction

**Column Loads / Settlement**

Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units
--------	-----------	-----	------	------	------	------	-------

Insert Copy Delete Delete All

**Cap Loads**

Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units
-----------	-----	----------	------	------	------	------	-------

Insert Copy Delete Delete All

Name: PL1 Pedestrian Live

Description: Pedestrian loading

Factors: Multiplier for Loads: 1

Auto Generation: Generate

Import Loads: Import

Note: Vertically downward loads be added as negative loads in Y direction.

OK Cancel

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

Stream flow pressure is calculated in accordance with *LRFD Specification* Section 3.7.3.1. For the 100-year flood,  $V=10$  ft./sec. Using this velocity and a  $C_d$  of 0.7, the stream flow pressure turns out to be 0.07 ksf. This is input as a column load acting from the top of the column to the scour depth.  $Y_1$  and  $Y_2$  are measured from the point of fixity on the drilled shaft. Looking back to our column and shaft design, the shaft extends 15 ft. above the point of fixity, the scour depth is 5 ft. above the point of fixity, and the column is 19 ft. long. Therefore,

$$L = (15 \text{ ft.} + 19 \text{ ft.}) = 34 \text{ ft.}$$

$$Y_1 = (\text{scour depth} - \text{point of fixity}) = 5 \text{ ft.}$$

$$Y_2 = (\text{top of column} - \text{point of fixity}) = 34 \text{ ft.}$$

The input screen for stream flow loads is shown below.

**Loads: Load data**

**Bearing / Girder loads**

Bearing Line	Bearing Point#	Dir	Load (kips)

Insert Copy Delete Delete All

**Column Loads / Settlement**

Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units
1+Shaft	Pressure	X	-0.0700	0.1500	0.0000	0.9700	ksf
*							

Insert Copy Delete Delete All

**Cap Loads**

Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units

Insert Copy Delete Delete All

**Strain Load**

Unit: 0.

+ Expansion - Contraction

Name: WA1 Water

Description: 100-yr flood

Factors: Multiplier for Loads: 1.

Auto Generation: Generate

Import Loads: Import

Note: Vertically downward loads be added as negative loads in Y direction.

OK Cancel

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

The next load on the Load screen is wind. Bringing up the Auto Load Generation screen for Wind on Struc brings up the screen shown below. Once the screen comes up, input a wind angle of zero, toggle on “Open Country” under Bridge Location, enter a value of zero for “Elevation above which wind load acting,” and click Generate.

**Auto Load Generation: Wind on Struc**

Wind Angle, deg: 0

Generate for multiple angles ☐

Generate Wind on Live at the same time ☐ Length of LL: 65.875 ft

Elevation above which Wind Load acting: 0. ft (for column)

Bridge location :  
☒ Open country  
☐ Suburban  
☐ City

Default Wind Pressure ☒

Wind Pressure for superstructure

Trans: 58.9866 psf  
Longit: 0. psf  
Consider Overturning ☒  
Overturning: 20. psf

Wind Pressure for substructure in wind direction

Cap: 43.8474 psf  
Column: 43.8474 psf

Generate Cancel

The Auto Load Generation screen for Wind Load on Live Load is shown below. In that screen, input zero for the wind angle and 65.875 for the tributary length. Again, click Generate to have RC-PIER calculate the loads.

**Auto Load Generation: Wind Load on LL**

Wind Angle: 0

Generate for multiple angles ☐

Length of Live Load: 65.875 ft

Generate Cancel

**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

Temperature, creep, and shrinkage are the final loads that need to be entered. RC-PIER contains individual load screens for each of these. However, for this example, we have chosen to include the creep and shrinkage movement with the temperature by calculating a contributing length that results in the same structural movement as the sum of the three.

Pier 1 & 2 bearings are fixed and each takes half the thermal, creep, and shrinkage movement from Span 2. (Span 1 movement is taken up by the expansion bearing at the abutment.) Change in temperature is specified as 80°F for concrete bridges in the *Design Guide*.

$$\Delta L_{TEMP} = L(0.000072)\Delta T = 44 \text{ ft.}(0.000072)(80^{\circ}\text{F}) = 0.2534 \text{ in.}$$

$$\Delta L_{CR\&SH} = \frac{0.385 \text{ in.}}{2} = 0.1925 \text{ in.}$$

$$L = \frac{\Delta L}{(0.000072)\Delta T} = \frac{0.2534 \text{ in.} + 0.1925 \text{ in.}}{(0.000072)80^{\circ}\text{F}} = 77.41 \text{ ft.}$$

The Auto Load Generation screen for Temperature Load is shown below. Once all these values are entered, click Generate.

**Auto Load generation: Temperature load**

**Bearing data**

☒ Fixed bearings  
☐ Expansion bearings  
Elastomeric bearings

Stiffness elastomer:  
☐ Manual input ☒ Compute

Area of bearing, A:  in<sup>2</sup>  
Shear modulus of Elastomer:  ksi  
Total elastomer thickness:  in

**Superstructure data**

Contributing length:  ft  
Change in temperature:  °F  
Coeff Thermal Expansion, alpha:  /°F

**Pier data**

☒ Auto compute  
☐ User input:  
Kcols:  kips/in

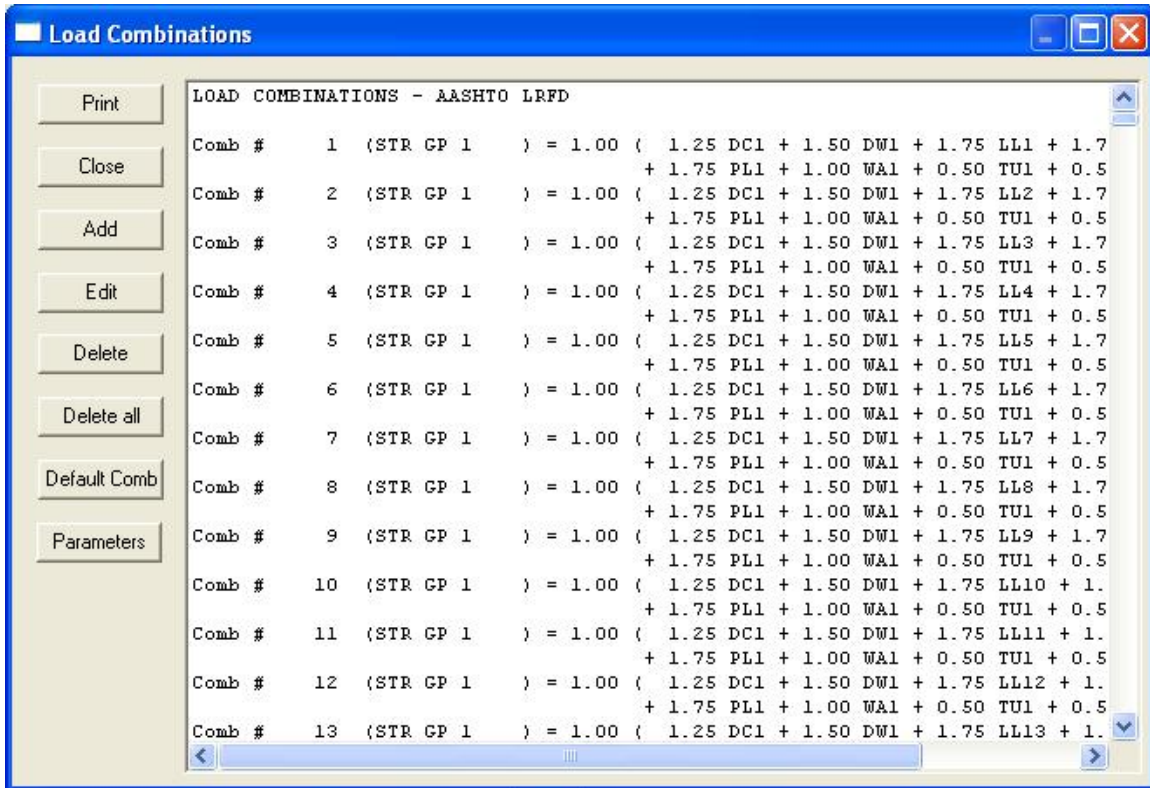
Generate Cancel

Direction of thermal force (Z): ☒ +[Z] ☐ -[Z]

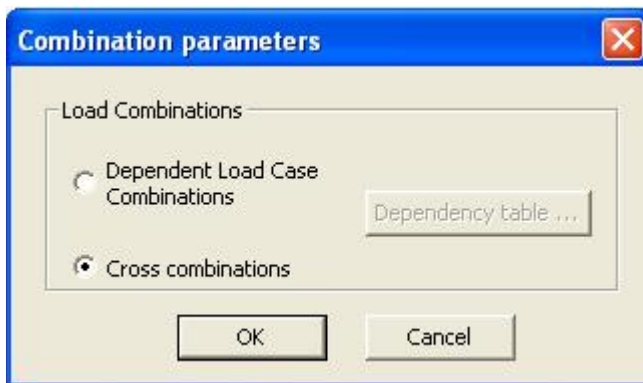


## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

This completes the load data for the first preliminary run. The next step is to generate the load combinations. To do this, click Combinations in the lower right corner of the Loads screen. This will bring up the Load Combinations Screen. This screen may be blank if you have not yet generated the combinations.



Once in the Load Combinations screen, click Parameters.



Select Cross Combinations. If this is not done, the analysis will not run due to a difference in the number of live load and braking load cases. Click OK to return to the Load Combinations Screen. At this point, if your load combinations screen is blank, click Default Comb to generate the load combinations. Click Close to return to the Loads Screen.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

The next step is to run the analysis. Bring up the Analysis screen by clicking on the Analysis tab. Once in the Analysis screen, click the A/D Parameters button to bring up the screen shown below.

**Analysis/Design Parameters (LRFD)**

Resistance Factor,  $\phi$   
☒ Phi as per 2006 classification  
☐ Phi as per classic approach

Tension Controlled: 0.9

Shear and torsion: (normal weight) 0.9

Shear and torsion: (lightweight) 0.7

Compression Controlled: (ties) 0.75

Compression Controlled: (spiral) 0.75

Compression in STM: 0.7

Dynamic Load Allowance, IM

	Truck	Lane	Fatigue
Cap:	0.33	0.	0.15
Column:	0.33	0.	0.15
Footing:	0.	0.	0.

Crack Control Criteria  
☐ LRFD 2004  
☒ LRFD 2005 Interims

Fatigue  
 ff term: 24.

Clear Concrete Cover, in

Cap top/bottom: 2.

Cap side: 2.

Column: 2.

Footing top/bottom: 3.

Footing side: 3.

Multiple Presence Factors

Lane# 1: 1.2

Lane# 2: 1.

Lane# 3: 0.85

Lane# 4: 0.65

Crack Control Factor,  $z$ , kips/in

Cap: 170.

Column: 170.

Footing: 130.

Exposure Factors

Cap: 1.

Column: 1.

Footing: 1.

Seismic Design  
 Seismic Design Parameters ...

Column Slenderness Consideration

☐ P-delta Method

Number of iterations: auto 2.1

Degree of Fixity in Foundations for Moment Magnification 0.4

☐ Compute K for braced columns as per Interim 2006

Modulus of rupture

Normal: 0.37 x  $\sqrt{f_c}$

Sand-lightweight: 0.2 x  $\sqrt{f_c}$

All-lightweight: 0.17 x  $\sqrt{f_c}$

Design cap/footing for magnified moments

☐ Design cap for magnified moments

☐ Design footing for magnified moments

c/dt ratio

Comp -> 0.6 <- Transition -> 0.375 <- Tension

OK Cancel

For Shear and Torsion Calculations, select the Simplified radio button for both the Cap method and the Footing method to keep our LEAP Bridge run consistent with load ratings done in Virtis/Opis. Under Column Slenderness Consideration, set the Degree of Fixity in Foundations for Moment Magnification to 0.4, since the continuous shaft provides a stiff end condition for the column. The remainder of the data on the screen can stay on the default setting. Click OK to return to the Analysis screen. Once in that screen, click the Run Analysis button. The completed Analysis screen is shown below.



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

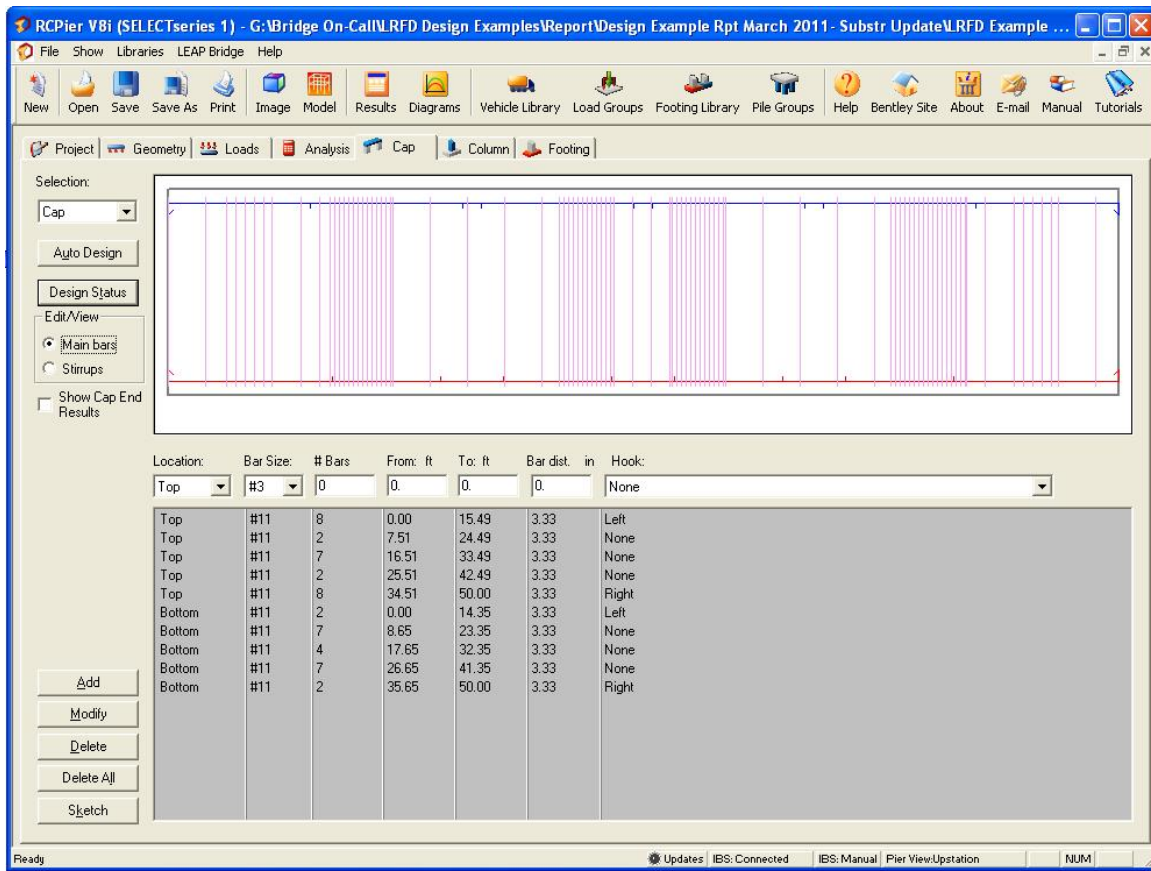
Memb	Node	Fx(Max/Min)	Fy	Fz	Mx	My	Mz
1	1	1109	-7.561	4.82	0.1437	-107.9	-107.1
		274.3	15.72	4.751	1.029	-136.4	328.6
1	2	-248.9	-15.22	-4.751	-1.029	65.18	-107
		-1074	4.873	-4.82	-0.1437	35.6	6.617
2	2	1074	-4.873	4.82	0.1437	-35.6	-6.617
		248.9	15.22	4.751	1.029	-65.18	107
2	3	-224.2	-16.02	-4.751	-1.029	-34.6	219.6
		-1039	-0.007698	-4.82	-0.1437	-65.61	-41.83
3	4	1205	-1.493	6.988	0.3212	-361	-29.83
		356	23.28	4.869	0.9605	-136.7	413.6
3	5	-330.5	-20.08	-4.869	-0.9605	63.69	-91.63
		-1170	1.493	-6.988	-0.3212	319.2	7.438
4	5	1170	-1.493	6.988	0.3212	-319.2	-7.438
		330.5	20.08	4.869	0.9605	-63.69	91.63
4	6	-305.8	-16	-4.869	-0.9605	-38.56	283.2
		-1136	1.493	-6.988	-0.3212	-395.1	-23.91
5	7	1106	1.319	4.817	-0.1286	-107.8	22.8
		333.5	-22.09	4.746	-1.018	-119.1	-414.6
5	8	-308.1	18.9	-4.746	1.018	47.95	110.4
		-1071	-1.319	-4.817	0.1286	35.59	-3.019
6	8	1071	1.319	4.817	-0.1286	-35.59	3.019
		308.1	-18.9	4.746	-1.018	-47.95	-110.4
6	9	-283.4	14.81	-4.746	1.018	-51.7	-239.4
		-1037	-1.319	-4.817	0.1286	-65.57	24.68
7	10	0	0	0	0	0	0
		0	0	0	0	0	0
7	11	0	8.19	0	0	0	-12.78
		0	8.19	0	0	0	-12.78

The analysis information shown in the above screen is for forces and moments for the Envelope Strength case. To show forces and moments for other load cases simply select the case for which the forces and moments are desired in the Type drop down menu. To display rotations and displacements, select Displ. & Rotation in the Effect drop down menu.

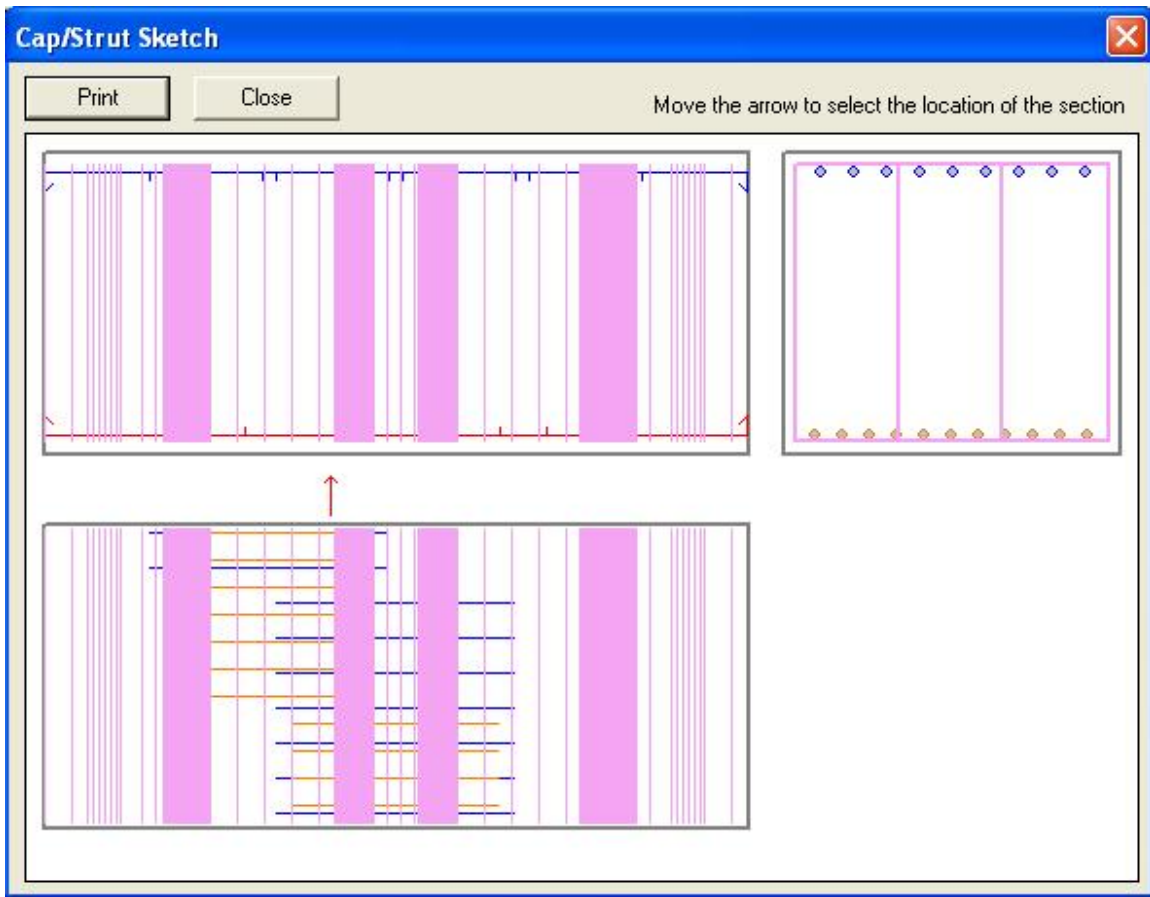
At this point, input and analysis are complete for all load cases except the extreme event load cases.

To complete the cap design for these loading combinations, click the Cap tab at the top of the screen. Once the Cap Design screen appears, click the Auto Design button. Select a preliminary main bar and stirrup size and click OK. Next, in the Edit/View box, toggle "Main bars" on. The resulting screen is shown below.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications



The above screen is a bit difficult to decipher but, to make things a bit clearer, click the sketch box in the lower left-hand corner of the Cap screen, and a sectional view of the cap appears. A copy of the cap section is shown on the next page.

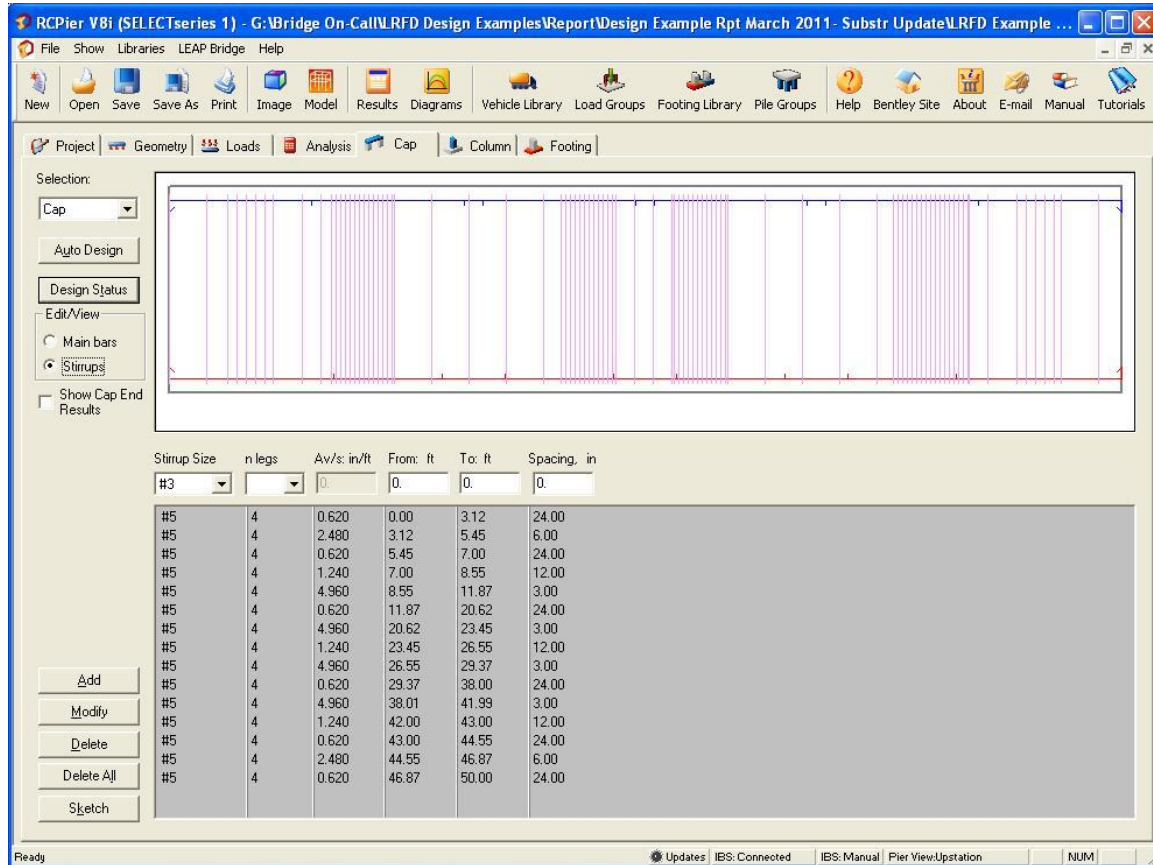


In the Cap/Strut Sketch screen, you can set your cursor at any location along the length of the cap and left click. The sectional view will change to show the reinforcing requirements at the selected location. The section shown (see the location of the arrow in the screen view above) is the critical section.

Top of cap reinforcing will be 9 #11 bars and bottom reinforcing will be 11 #11 bars. Again, this is a workable design, and the preliminary cap section that was selected is acceptable for further design.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

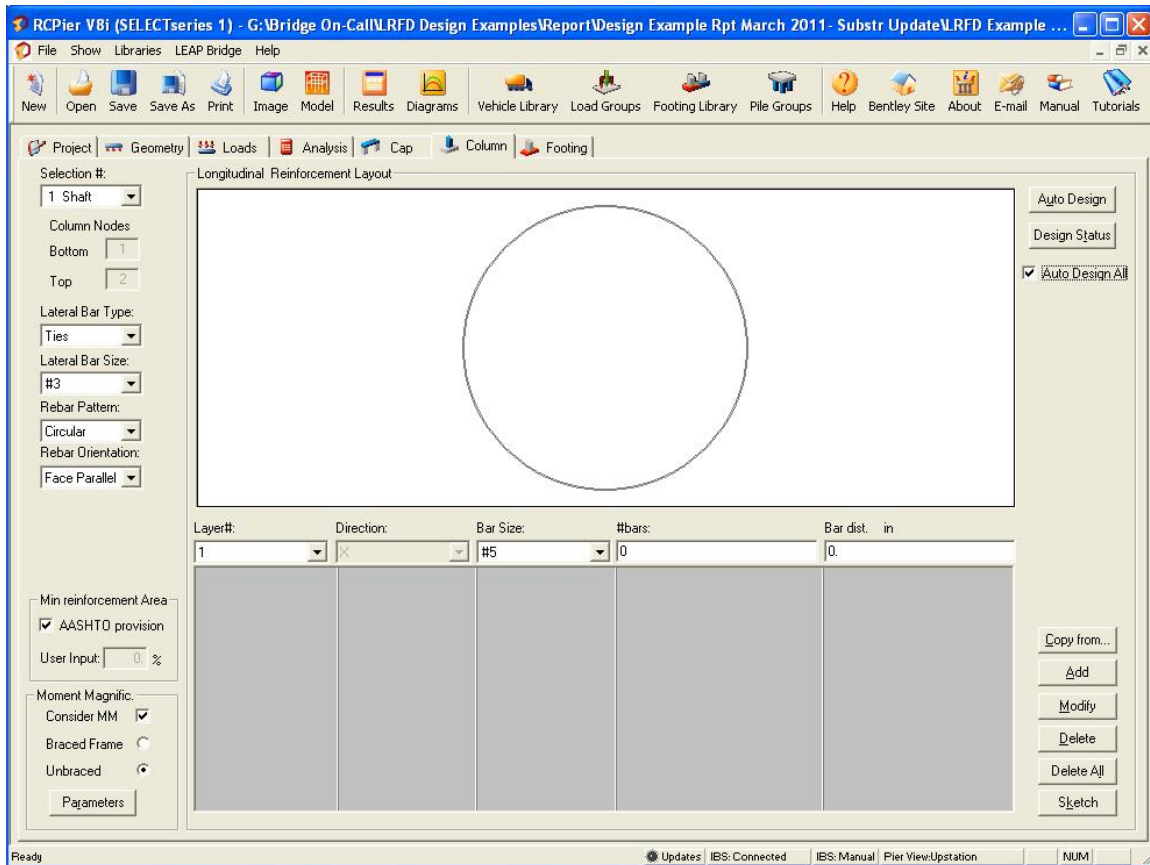
For stirrups, (Toggle on Stirrups in the Edit / View box) the Cap Design screen is shown below.



The minimum spacing for the selected bar size (double # 5 bars) is 3 in. This is a bit tight, but the spacing can be increased by selecting a larger bar. Again then, we will say that the preliminary cap proportions are acceptable.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

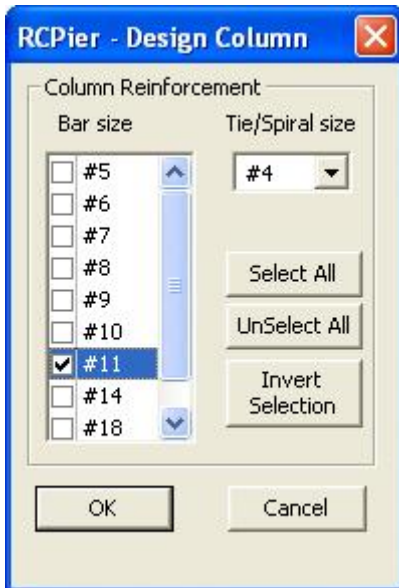
To design the columns and shafts, click on the Column tab. This will bring up the Column Design screen. At first the columns will appear without any reinforcement, as shown below.



Once in the screen, under Moment Magnification, select the check box for Consider MM, and click the Unbraced button. Note that this needs to be done for each column and shaft separately.

Click on the Auto Design All checkbox on the right side of the screen. In the Column screen, click Auto Design and the following screen comes up.

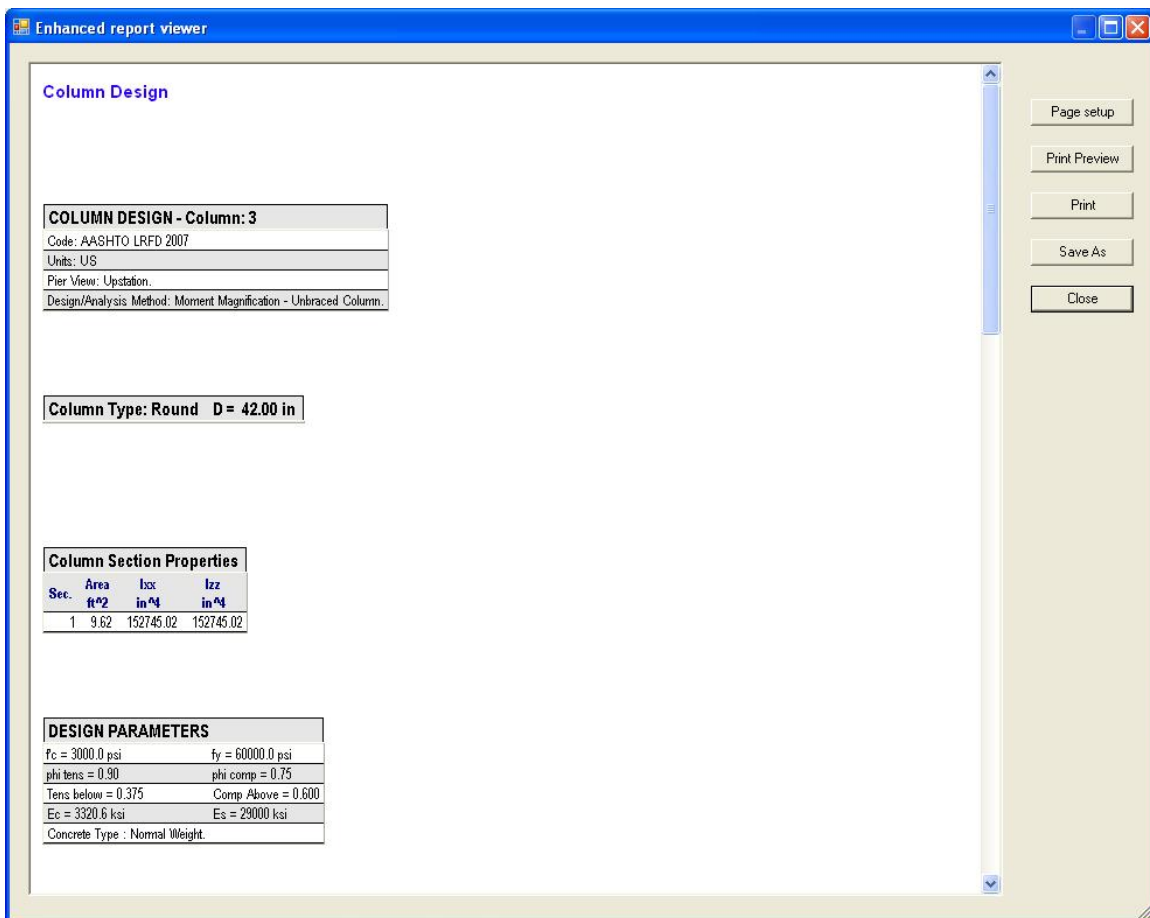
**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**



The dialog box is titled "RCPier - Design Column". It contains a section for "Column Reinforcement" with two columns: "Bar size" and "Tie/Spiral size". The "Bar size" column has a list of bar sizes from #5 to #18, with #11 selected. The "Tie/Spiral size" column has a dropdown menu set to #4. There are buttons for "Select All", "UnSelect All", and "Invert Selection". At the bottom are "OK" and "Cancel" buttons.

Bar size	Tie/Spiral size
<input type="checkbox"/> #5	#4
<input type="checkbox"/> #6	
<input type="checkbox"/> #7	
<input type="checkbox"/> #8	
<input type="checkbox"/> #9	
<input type="checkbox"/> #10	
<input checked="" type="checkbox"/> #11	
<input type="checkbox"/> #14	
<input type="checkbox"/> #18	

Select rebar sizes and click OK. RC-PIER completes the design shown below.



The window is titled "Enhanced report viewer". It displays the "Column Design" report for "Column: 3". The report includes the following sections:

**COLUMN DESIGN - Column: 3**  
Code: AASHTO LRFD 2007  
Units: US  
Pier View: Upstation.  
Design/Analysis Method: Moment Magnification - Unbraced Column.

**Column Type: Round D = 42.00 in**

**Column Section Properties**

Sec.	Area ft <sup>2</sup>	Ixx in <sup>4</sup>	Izz in <sup>4</sup>
1	9.62	152745.02	152745.02

**DESIGN PARAMETERS**

$f_c = 3000.0$ psi	$f_y = 60000.0$ psi
$\phi_{\text{tens}} = 0.90$	$\phi_{\text{comp}} = 0.75$
Tens below = 0.375	Comp Above = 0.600
$E_c = 3320.6$ ksi	$E_s = 29000$ ksi
Concrete Type : Normal Weight.	

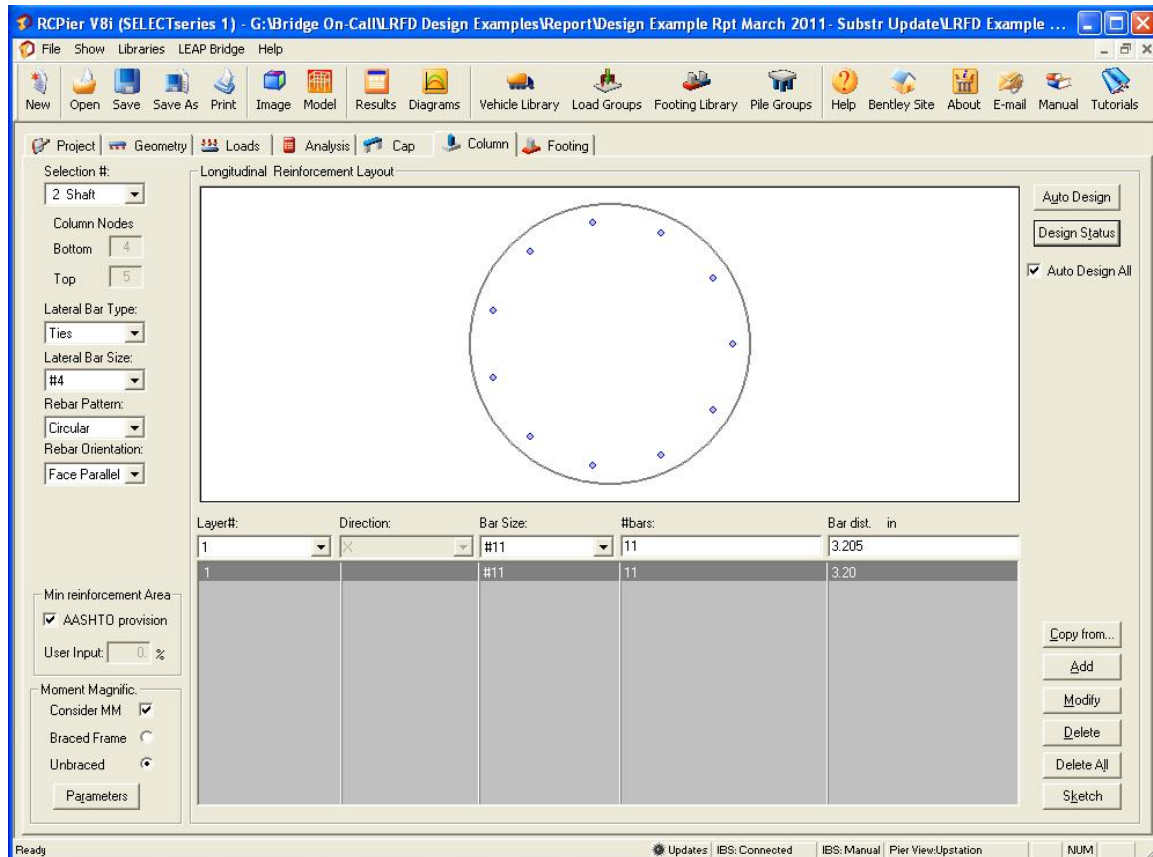
On the right side of the window, there are buttons for "Page setup", "Print Preview", "Print", "Save As", and "Close".



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

For the loading input so far, the main reinforcing for the column will consist of 10 #11 bars. This is a workable design and the determination at this point is made that the preliminary column diameter is acceptable.

Shaft design was already completed using the Auto Design All checkbox. In the Column tab, select “2 Shaft” from the drop down menu in the upper left corner. (This shaft was determined to be the most heavily loaded from an inspection of the analysis screen.) The resulting screen is shown below.



As shown above, main shaft reinforcing will consist of 11 #11 bars. Again this is an acceptable design, and it appears that the selected shaft diameter will work.

This completes the preliminary design except for the Extreme Event load cases. Next, we will proceed with the seismic evaluation.

### 5.1.2. Seismic Evaluation

The NMDOT has adopted the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* for use in seismic evaluations. In this example the provisions of that Specification will be followed step by step using the flowcharts in Section 1 as a guide. In this section wherever a specification, article, or figure is referred to, the reference is the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, hereinafter referred to as *Seismic*

*Specifications*, unless noted otherwise. NMDOT will also accept the *AASHTO LRFD Bridge Design Specifications*, Fifth Edition, 2010 for seismic design.

Refer then to the first flow chart in the *Seismic Specifications*, Figure 1.3-1a on page 1-6. The first box encountered deals with the applicability of the specification. A review of the referenced Article, 3.1, reveals that the specification is applicable to this bridge.

The second box pertains to temporary bridges and is not applicable to this example.

The third box refers to Article 3.2 for Performance Criteria. This article is informational and explains the specification's philosophy. No action is required.

The fourth box references Article 6.2. That article discusses foundation investigation requirements. For this example, we will assume values that would normally be provided from a foundation investigation.

The fifth box deals with liquefaction. Since ground water is not present at this site, liquefaction will not be an issue.

The next box refers to Article 3.3 for the selection of an Earthquake Resisting System (ERS). Since this is only applicable to seismic design categories (SDC) C and D, we will wait until the SDC is determined before making a determination of ERS.

The next box refers to Article 3.4 for a determination of the Design Response Spectrum.

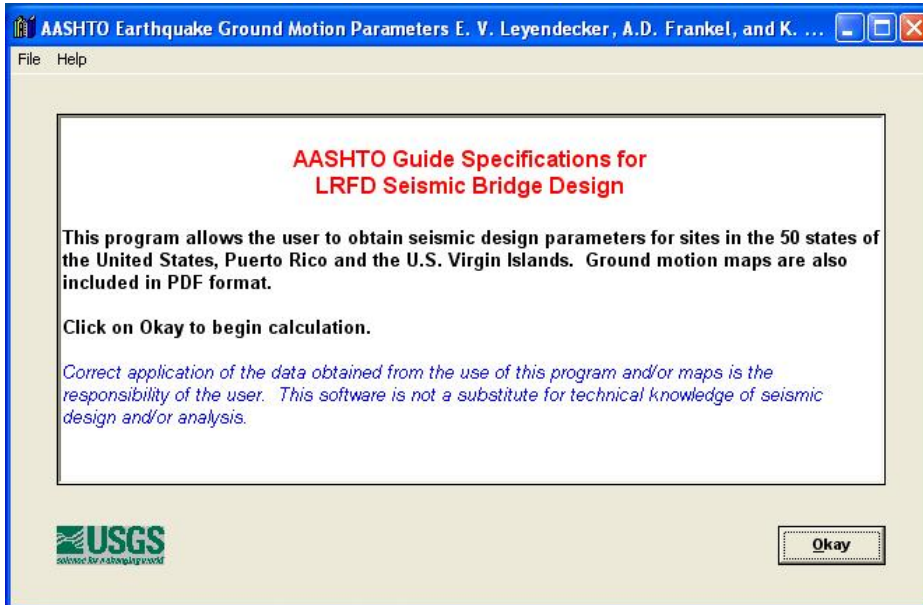
#### **5.1.2.1 Design Response Spectrum:**

The design response spectrum for this bridge will be determined using the AASHTO Seismic Design Parameters software, Version 2.10, available on the USGS website or from the Seismic Design Parameters CD that accompanies the *LRFD Specifications*. The design response spectrum given by the software can be checked with the hand calculation procedure shown in Article 3.4.1 of the *Seismic Specifications*.

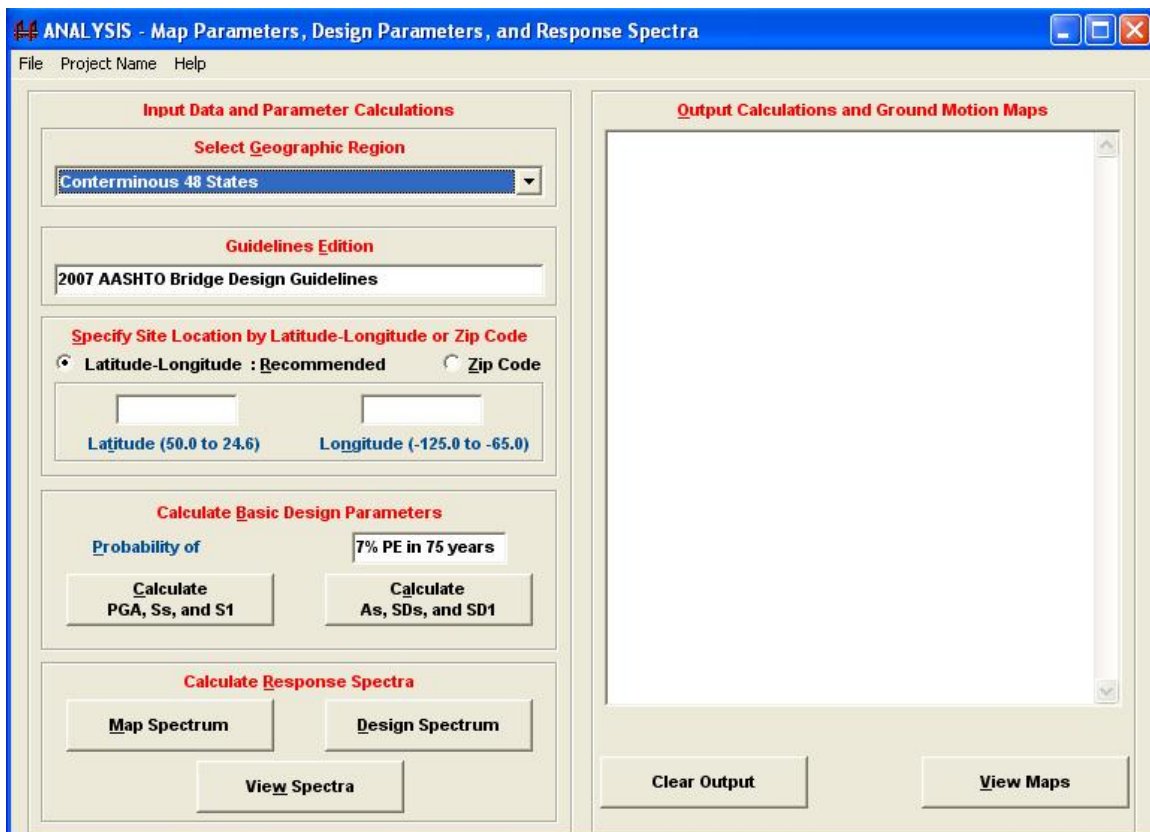


## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

After you open the AASHTO Seismic Design Parameters software, the following screen will appear.



Click OK to bring up the Analysis screen.



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

To calculate the design response spectrum for our bridge, we will have to input a location by latitude-longitude or by zip code. On page one of this example, we stated that our bridge was near Socorro, NM. In the Specify Site Location by Latitude-Longitude or Zip Code, make sure that the Latitude-Longitude radio button is selected and then type 34.0595 in the Latitude box and -106.8990 in the Longitude box.

Click the Calculate As, SDs, and SD1 button to generate the Site Coefficients screen.

**Calculate Site Coefficient**

Accelerations

PGA: 0.132

Ss: 0.313

S1: 0.086

**Site Class**

Site Class A  
Site Class B  
Site Class C  
**Site Class D**  
Site Class E  
Site Class F

**Site Coefficients**

Fpga: 1.54

Fa: 1.55

Fv: 2.40

OK

Site Class	Peak Ground Acceleration Coefficient (PGA)				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (Ss)				
	Ss ≤ 0.25	Ss = 0.50	Ss = 0.75	Ss = 1.00	Ss ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

Site Class	Spectral Acceleration Coefficient at Period 1.0 sec (S1)				
	S1 ≤ 0.10	S1 = 0.20	S1 = 0.30	S1 = 0.40	S1 ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

**SITE FACTOR TABLE NOTES**

1. Use straight-line interpolation to calculate values of site coefficients for intermediate values of PGA, Ss, and S1.
2. Site-specific geotechnical investigation and dynamic site response analyses should be performed for all sites in Site Class F.

Site Class definitions are presented in Table 3.4.2.1-1 on page 3-45 of the specification. To determine site class from this table, site class parameters need to be determined. Article 3.4.2.2 presents the equations needed to determine the site class parameters.  $N_{bar}$  can be determined using equation 3.4.2.2-2 and information given in drill logs obtained through a foundation investigation. We will assume  $N_{bar}$  from the foundation investigation to be 15.2. Entering Table 3.4.2.1-1 with this information, the site class is D.

Ensure that Site Class D is selected in the Site Coefficients screen, and click OK. This will display the output values shown in the Output Calculations and Ground Motion Maps window in the Analysis Screen.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

The resulting Analysis screen is shown below. Note that values for  $PGA$ ,  $S_s$ , and  $S_1$  are shown in the top half of the Output Calculations and Ground Motion Maps window, and the values for  $F_{pga}$ ,  $F_a$ ,  $F_v$ ,  $A_s$ ,  $S_{DS}$ , and  $S_{D1}$  are shown in the bottom half.

**ANALYSIS - Map Parameters, Design Parameters, and Response Spectra**

File Project Name Help

**Input Data and Parameter Calculations**

**Select Geographic Region**

Conterminous 48 States

**Guidelines Edition**

2007 AASHTO Bridge Design Guidelines

**Specify Site Location by Latitude-Longitude or Zip Code**

☒ Latitude-Longitude : Recommended ☐ Zip Code

34.0595 -106.8990

Latitude (50.0 to 24.6) Longitude (-125.0 to -65.0)

**Calculate Basic Design Parameters**

Probability of 7% PE in 75 years

Calculate PGA, Ss, and S1 Calculate As, SDs, and SD1

**Calculate Response Spectra**

Map Spectrum Design Spectrum

View Spectra

**Output Calculations and Ground Motion Maps**

Conterminous 48 States  
2007 AASHTO Bridge Design Guidelines  
AASHTO Spectrum for 7% PE in 75 years  
Latitude = 34.059500  
Longitude = -106.899000  
Site Class B  
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.132	PGA - Site Class B
0.2	0.313	Ss - Site Class B
1.0	0.086	S1 - Site Class B

Conterminous 48 States  
2007 AASHTO Bridge Design Guidelines  
Spectral Response Accelerations SDs and SD1  
Latitude = 34.059500  
Longitude = -106.899000  
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1  
Site Class D - Fpga = 1.54, Fa = 1.55, Fv = 2.40  
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.204	As - Site Class D
0.2	0.485	SDs - Site Class D
1.0	0.208	SD1 - Site Class D

Clear Output View Maps

To calculate the design response spectrum, click the Design Spectrum button. You can view the spectrum data in the Output Calculations and Ground Motion Maps window of the Analysis screen.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

**ANALYSIS - Map Parameters, Design Parameters, and Response Spectra**

File Project Name Help

**Input Data and Parameter Calculations**

**Select Geographic Region**

Conterminous 48 States

**Guidelines Edition**

2007 AASHTO Bridge Design Guidelines

**Specify Site Location by Latitude-Longitude or Zip Code**

☒ Latitude-Longitude : Recommended ☐ Zip Code

34.0595 -106.8990

Latitude (50.0 to 24.6) Longitude (-125.0 to -65.0)

**Calculate Basic Design Parameters**

Probability of 7% PE in 75 years

Calculate PGA, Ss, and S1 Calculate As, SDs, and SD1

**Calculate Response Spectra**

Map Spectrum Design Spectrum

View Spectra

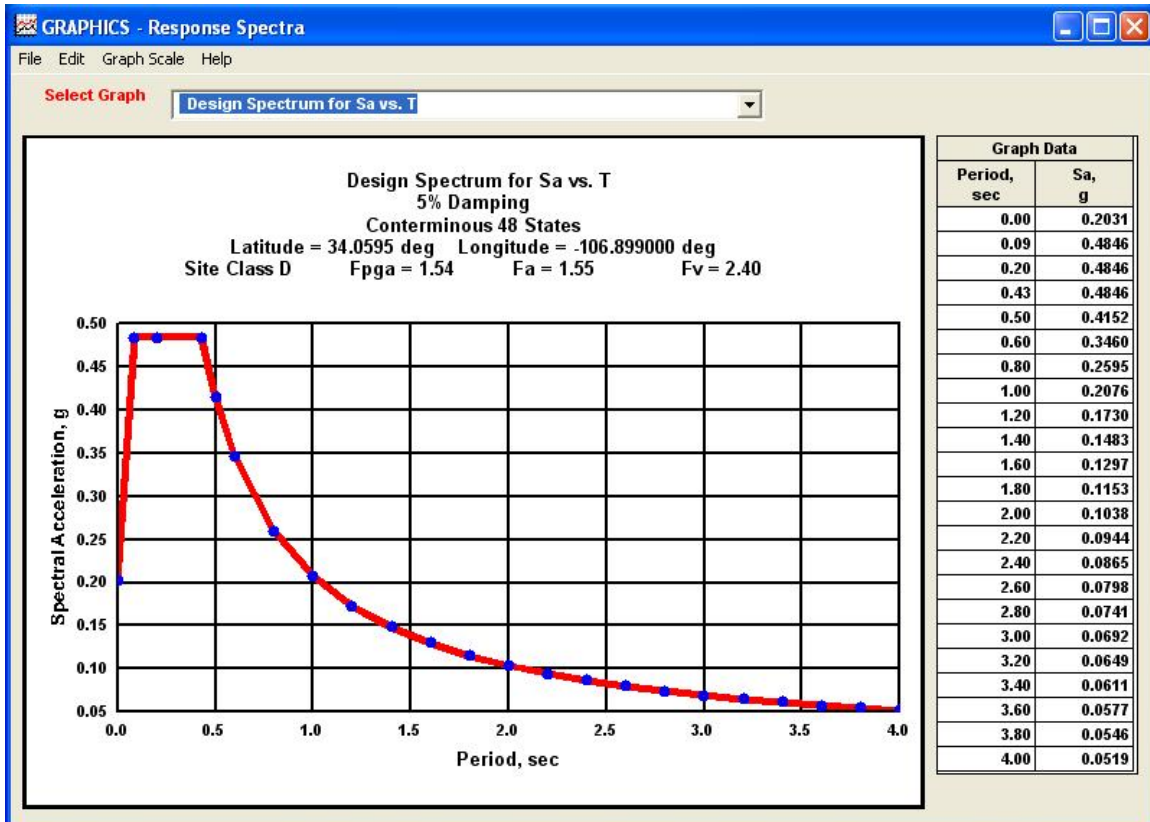
**Output Calculations and Ground Motion Maps**

Conterminous 48 States  
2007 AASHTO Bridge Design Guidelines  
Design Response Spectra for Site Class D  
Latitude = 34.059500  
Longitude = -106.899000  
As = FpgaPGA, SDs = FaSs, SD1 = FvS1  
Site Class D - Fpga = 1.54, Fa = 1.55, Fv = 2.40  
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd (in.)	
0.000	0.203	0.000	T = 0.0, Sa = As
0.086	0.485	0.035	
0.200	0.485	0.189	T = 0.2, Sa = SDs
0.428	0.485	0.869	T = Ts, Sa = SDs
0.500	0.415	1.014	
0.600	0.346	1.217	
0.800	0.259	1.622	
1.000	0.208	2.028	T = 1.0, Sa = SD1
1.200	0.173	2.434	
1.400	0.148	2.839	
1.600	0.130	3.245	
1.800	0.115	3.651	
2.000	0.104	4.056	
2.200	0.094	4.462	
2.400	0.086	4.867	
2.600	0.080	5.273	
2.800	0.074	5.679	
3.000	0.069	6.084	
3.200	0.065	6.490	
3.400	0.061	6.896	
3.600	0.058	7.301	
3.800	0.055	7.707	
4.000	0.052	8.112	

Clear Output View Maps

Once you have calculated the design response spectrum, you can view a chart of the data by clicking the View Spectra button. The following chart shows the design response spectrum for our bridge.



This completes the determination of the design response spectrum and we return to the flow chart: Figure 1.3-1a.

The next box in the flow chart is the determination of the seismic design category (SDC) per Article 3.5 of the *Seismic Specifications*.

#### 5.1.2.2 Select Seismic Design Category:

From Article 3.5 and Table 3.5-1, SDC is B since  $S_{D1}=0.208$

Going back to the flow charts, Figure 1.3-1a, with this information leads us to Figure 1.3-1b. The first box in that figure under the SDC B column refers us to Figure 1.3-2 for the displacement demand analysis.

#### 5.1.2.3 Displacement Demand Analysis:

The first action box in Figure 1.3-2 refers to Article 4.1 for design proportioning recommendations. This bridge meets those recommendations.

The next box refers to Article 4.2 for the determination of the analysis procedure. Table 4.2.1 indicates that Procedure 1 would be acceptable for use since this is a regular bridge with more than two but less than six spans. Table 4.2-2 identifies Procedure 1 as an Equivalent Static Analysis described in Article 5.4.2.



Going back to Figure 1.3-2 and skipping the SDC D diamond leads to “Select Horizontal Axes for Ground Motions,” discussed in Article 4.3.1. We will select the longitudinal and transverse directions as our axes.

The next box in Figure 1.3-2 contains “Damping Considerations.” These considerations are discussed in Article 4.3.2. They are discretionary and will not be used in this example.

The next item in the box is “Short Period Structure Considerations.” These are discussed in Article 4.3.3. The equations in that article reveal that we need the period of the structure (T) to determine the displacement magnification factors. That value has not yet been determined. We will return to this provision after the structure period has been calculated.

Return to Figure 1.3-2. The next box refers us to Figure 1.3-4, “Analytical Modeling and Procedures.” In that figure, skip SDC C or D boxes and go to the Select Analytical Procedures box. That box identifies Article 5.4.2 as containing the requirements for Procedure 1. As you will recall, that is the procedure we have selected for use in this example. Skip the next box as not applicable and go to the “Effective Section Properties box. This requirement is discussed in Article 5.6.

#### **5.1.2.3.1 Effective Section Properties:**

We will determine the effective stiffness ratio from Figure 5.6.2-1 in Article 5.6 as follows:

Referring back to the Analysis screen in our RC-PIER run, we find that the maximum unfactored axial load is about 663 kips. Also,  $f_c' = 3$  ksi and  $A_g = 1385 \text{ in.}^2$  for a 42 in. column. (For simplicity, we are assuming here that the 42 in. column extends down to the point of fixity.)

Using these values yields

$$P / (f_c' A_g) = 0.16.$$

Entering Figure 5.6.2-1a with this value at the  $A_{st}/A_g = 0.01$  line gives an elastic stiffness ratio,  $I_{eff}/I_g$ , of 0.38.

The formula for moment of inertia of a circular section is

$$I_g := \pi \cdot \frac{D^4}{64}$$

$$I_g = 152745 \text{ in.}^4 \text{ for a 42 in. column.}$$

$$I_{eff} = 0.38 \times 152745 = 58043 \text{ in.}^4$$

The diameter of a circular section that would have this moment of inertia is 33 in.

The next box in Figure 1.3-4 addresses Abutment Modeling. Abutment contribution to the seismic resistance of this bridge will be calculated using the *Caltrans Seismic Design Criteria*, Version 1.6 (available on the Caltrans web site). For the transverse demand, the transverse abutment stiffness is assumed to be half the stiffness of the adjacent pier. For the longitudinal demand, longitudinal stiffness is based on an effective embankment fill stiffness.

Go back to Figure 1.3-4. Skip the Foundation Modeling box since liquefaction is not of concern in this example. This brings us to the Conduct Demand Analysis box.

#### 5.1.2.3.2 Demand Analysis:

To determine the demand capacity in both the transverse and longitudinal direction we will use the Equivalent Static Analysis (ESA) using the uniform load method outlined in Article 5.4.2.

##### Transverse Direction

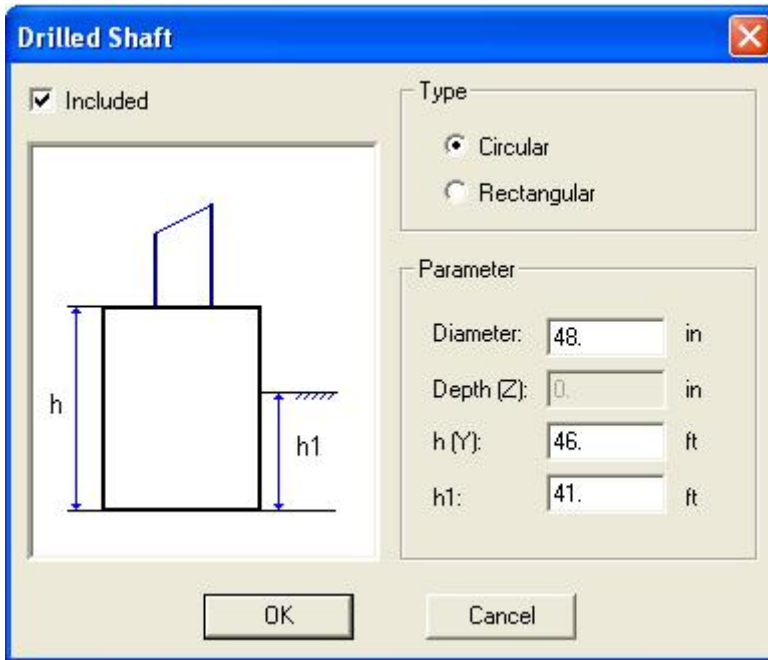
Step 1: Calculate  $v_s(x)$

To calculate  $v_s(x)$ , we will use a  $p_o$  load of 1 kip/ft. acting along the length of the bridge. Since the bridge is 178.5 ft. long, this translates into a load of 89.25 kips acting horizontally over half the bridge length. To determine the transverse stiffness, we will assume this acts at the top of the pier cap, ignoring the abutment stiffness for the time being. We will use RC-PIER to determine  $v_s(x)$ .

Make a copy of the previous RC-PIER run. First open the Geometry screen and then the Column screen. Once in the Column screen, change the column diameters from 42 in., the gross diameter, to 33 in., the effective diameter. The changed screen is shown below.

No. #	Loc. from left of cap. (ft)	Bot. Elev. (ft)	Diameter (in)	Factor of Reduced MI	Column fixity
1	7	4320.58	33	1	Fixed
2	25	4320.58	33	1	Fixed
3	43	4320.58	33	1	Fixed

For each column in the Rounded Column screen, click the Drilled Shaft button to view the following screen.



The image shows a software dialog box titled "Drilled Shaft". It has a blue title bar with a close button (X) in the top right corner. Inside the dialog, there is a checkbox labeled "Included" which is checked. To the right of the checkbox is a "Type" section with two radio buttons: "Circular" (selected) and "Rectangular". Below this is a "Parameter" section with four input fields: "Diameter:" with the value "48." and unit "in"; "Depth (Z):" with the value "0." and unit "in"; "h (Y):" with the value "46." and unit "ft"; and "h1:" with the value "41." and unit "ft". On the left side of the dialog is a diagram of a rectangular column with a trapezoidal pile on top. The total height of the column is labeled "h", and the height from the base to the top of the pile is labeled "h1". At the bottom of the dialog are two buttons: "OK" and "Cancel".

From Article 3.4.1 of the *LRFD Specification*, the local and contraction scour depth and EQ loading do not need to be considered simultaneously. Therefore, we will need to adjust h1 for the drilled shafts. Keeping the point of fixity at 5 ft. below the streambed elevation and neglecting scour, the depth of fixity, h1, is at 41 ft. After input is complete, click OK. This will return you to the Column screen. Complete the drilled shaft input for each column and click OK on the Column screen to return to the Geometry screen.



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

Next open the Loads screen. Highlight the EQ1 load in the Load Type box, and click the horizontal arrow to move it to the right into the Selected Loads box.

Highlight the EQ1 load and click Edit. Input the 89.25 kips nominal load at the top of the cap as shown in the screen below.

**Loads: Load data**

**Bearing / Girder loads**

Bearing Line	Bearing Point#	Dir	Load (kips)
--------------	----------------	-----	-------------

Insert Copy Delete Delete All

**Column Loads / Settlement**

Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units
--------	-----------	-----	------	------	------	------	-------

Insert Copy Delete Delete All

**Cap Loads**

Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units
Force	X	0.0000	89.2500	0.0000	0.0000	0.0000	kips
*							

Insert Copy Delete Delete All

**Strain Load**

Unit: 0.

+ Expansion - Contraction

Name: EQ1

Description:

**Factors**

Multiplier for Loads: 1.

**Auto Generation**

Generate

**Import Loads**

Import

Note: Vertically downward loads be added as negative loads in Y direction.

OK Cancel

Click OK to return to the Loads screen. Next, highlight Extreme Event Seismic Group I in the Available Groups box and click the horizontal arrow to move it to the right into the Selected Groups box.

Next, click on the Analysis tab and then click Run Analysis. After the analysis is run, select Load Case under the Type pull down menu, Displ. & Rotation under the Effect pull down menu, and EQ1 under the Item pull down menu.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

The resulting screen is shown below.

The screenshot shows the RCPIER V8i (SELECTseries 1) software interface. The title bar indicates the file path: G:\Bridge On-Call\LRFD Design Examples\Report\Design Example Rpt April 2011 - Seismic Update\LRFD Example ... The menu bar includes File, Show, Libraries, LEAP Bridge, and Help. The toolbar contains icons for 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, and Tutorials. The main window displays the Analysis Results table for a bridge design example.

Node	Dx	Dy	Dz	Rx	Ry	Rz
1	0	0	0	0	0	0
2	0.00985	0.0002715	0	0	0	-0.0003081
3	0.2709	0.002684	0	0	0	-0.0001662
4	0	0	0	0	0	0
5	0.0103	0	0	0	0	-0.0003216
6	0.269	-2.403e-005	0	0	0	-2.338e-005
7	0	0	0	0	0	0
8	0.009747	-0.000269	0	0	0	-0.0003049
9	0.268	-0.00266	0	0	0	-0.000164
10	0.272	0.01664	0	0	0	-0.0001662
11	0.2715	0.01042	0	0	0	-0.0001662
12	0.2711	0.005113	0	0	0	-0.0001662
13	0.2708	0.0005578	0	0	0	-0.0001253
14	0.2704	-0.002701	0	0	0	-3.015e-005
15	0.2694	-0.0005253	0	0	0	3.272e-005
16	0.2691	0.0001554	0	0	0	-1.909e-006
17	0.2689	-0.0002045	0	0	0	-2.043e-006
18	0.2687	0.0004634	0	0	0	3.237e-005
19	0.2683	0.002649	0	0	0	-2.926e-005
20	0.2681	-0.0005619	0	0	0	-0.0001236
21	0.268	-0.0005058	0	0	0	-0.000164
22	0.268	-0.01028	0	0	0	-0.000164
23	0.268	-0.01644	0	0	0	-0.000164

The node numbers of the cap/column connections are 3,6, and 9. Note that the X-direction displacements at these nodes are 0.27 in.

Step 2: Calculate the bridge stiffness,  $K$ , and total weight,  $W$ .

$$K := \frac{p_o \cdot L}{v_{smax}} \quad \text{Eq. C5.4.2-1}$$

$$K_{\text{pier}} = (1 \text{ kip/ft.})(89.25 \text{ ft.})/0.27 \text{ in.} = 331 \text{ kip/in.}$$

From *Caltrans Seismic Design Criteria*, Section 7.8.2, abutment stiffness is half the stiffness of the adjacent pier. Therefore, total stiffness for our substructure is

$$K_{\text{total}} = (2 \text{ piers})(K_{\text{pier}}) + (2 \text{ abutments})(0.5K_{\text{pier}}) = 3K_{\text{pier}} = 993 \text{ k/in.}$$

By separate calculation, the unfactored dead load is  $W = 3194$  kips. This weight was calculated using the superstructure dead load generated by CONSPAN and adding the weight of the diaphragms, abutment backwalls, and piers down to the point of fixity.

Step 3: Calculate the period of the bridge.

$$T = 2\pi \sqrt{\frac{W}{Kg}}$$

Eq. C5.4.2-3

$$T = 2\pi \sqrt{\frac{3194 \text{ kip}}{(993 \text{ kip/in.})(32.2 \text{ ft./s}^2)(12 \text{ in./ft.})}} = 0.573 \text{ sec.}$$

Step 4: Calculate the equivalent static earthquake loading  $p_e$ .

$$p_e := S_a \cdot \frac{W}{L}$$

Eq. C5.4.2-4

Refer back to the portion of the example where the response spectrum curve was developed (page 82).  $T$  is equal to 0.573, which is greater than  $T_s$  ( $T_s = 0.428$ ). Since this is the case,

$$S_a = \frac{S_{D1}}{T}$$

Eq. 3.4.1-8

$$S_a = \frac{0.208}{T} = 0.363$$

Evaluating the above expression for  $p_e$  yields,

$$p_e = 0.363 \times \frac{3194 \text{ kip}}{178.5 \text{ ft.}} = 6.50 \text{ kip/ft.}$$

The portion of this force distributed to the piers is based on stiffness.

$$P_{\text{piers}} = p_e \times (K_{\text{pier}}/K_{\text{total}}) = 6.50 \text{ kip/ft.} \times (2/3) = 4.33 \text{ kip/ft.}$$

Step 5: Calculate displacements by scaling the displacement result from RC-PIER (Step 1) by the ratio of  $p_e/p_o$ .

$$\Delta_{DT} = \frac{v_{s \max} p_e}{p_o} = \frac{(0.27 \text{ in.})(4.33 \text{ kip/ft.})}{1 \text{ kip/ft.}} = 1.17 \text{ in.}$$

At this point, we will need to return to Article 4.3.3 to determine the magnification factor for short period structures. As you will recall, we skipped the step earlier since the period of the structure had not been determined. From previous calculations,  $T=0.573 \text{ sec.}$  and  $T_s=0.428 \text{ sec.}$

$$T^* = 1.25 T_s = 1.25(0.428 \text{ sec.}) = 0.535 \text{ sec.} \quad \text{Eq. 4.3.3-3}$$

$$\frac{T^*}{T} = \frac{0.535 \text{ sec.}}{0.573 \text{ sec.}} = 0.934 \leq 1.0$$

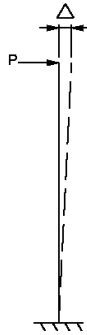
By Equation 4.3.3-2, since  $T^*/T$  is less than one,  $R_d = 1.0$ . Thus,  $\Delta_{DT}$  remains 1.17 in.

This completes the demand analysis in the transverse direction. We will next compute the demand analysis in the longitudinal direction.

#### Longitudinal Direction

In the longitudinal direction we will neglect the drilled shafts supporting the abutments but will include the passive soil pressure behind the backwalls. As an analytical model for the piers, we will assume that the columns are fixed at the point of maximum moment in the shafts (5 ft. below streambed elevation) and pinned but free to translate and rotate at the top. The equation for the displacement at the free end of this type of structure is

$$\Delta := \frac{P \cdot l^3}{3 \cdot E \cdot I}$$



This equation is presented in the Beam Diagrams and Formulas section of the Allowable Stress version of the *AISC Manual of Steel Construction*.

With this information, we can go back to Article 5.4.2 to determine the demand analysis.

Step 1: Calculate  $v_s(x)$ .

From the above discussion of the deflection at the free end of a fixed/free end structure,

$$\Delta := \frac{P \cdot l^3}{3 \cdot E \cdot I}$$

Again, assume  $p_o$  equal to 1 kip/ft. The load  $P$  acting on each column is

$$1 \text{ kip./ft. (178.5 ft.)} / 6 \text{ columns} = 29.75 \text{ kips/col.}$$

$$l = 19 + 5 = 24 \text{ ft.}$$

$E = 3320.56 \text{ ksi}$  for 3 ksi concrete, and

$$I = I_{\text{eff}} = 58043 \text{ in.}^4$$

Using these values and substituting into the above deflection equation yields,

$$\Delta = \frac{29.75 \text{ kip} (24 \text{ ft.} \times 12 \text{ in.} / \text{ft.})^3}{3 \times 3320.56 \text{ kip} / \text{in.}^2 \times 58043 \text{ in.}^4} = 1.23 \text{ in.}$$

This is the value for  $v_s(x)$  due to an arbitrary force applied at the piers only.

Step 2: Calculate the bridge stiffness  $K$ , and the total weight  $W$ .

Stiffness of the piers can be found from  $v_s(x)$  in Step 1 using the following equation.

$$K := \frac{P_o \cdot L}{v_{\text{smax}}} \quad \text{Eq. C5.4.2-1}$$

$$K_{\text{pier}} = (1 \text{ kip/ft.})(178.5 \text{ ft.}) / 1.23 \text{ in.} = 145.12 \text{ kip/in.}$$

From *Caltrans Seismic Design Criteria*, Section 7.8.1, embankment fill stiffness behind the abutment can be calculated using an assumed initial stiffness,  $K_i = 50 \text{ kip/in./ft.}$

Substituting  $K_i$ , abutment width, and fill height into equation 7.43b of the *Caltrans Seismic Design Criteria*, we can calculate stiffness of the embankment fill.

$$K_{\text{fill}} = K_i w \frac{h}{5.5 \text{ ft.}}$$
$$K_{\text{fill}} = \left( \frac{50 \text{ kip/in.}}{\text{ft.}} \right) (50 \text{ ft.}) \left( \frac{4.9167 \text{ ft.}}{5.5 \text{ ft.}} \right) = 2234.86 \text{ kip/in.}$$

Total longitudinal stiffness is the sum of the pier and embankment fill stiffness.

$$K_{\text{total}} = K_{\text{pier}} + K_{\text{fill}} = 2380 \text{ kip/in.}$$

From the demand analysis calculation for the transverse direction,  $W$  is equal to 3194 kips.

Step 3: Calculate the period of the bridge.

$$T = 2\pi \sqrt{\frac{W}{Kg}}$$

Eq. C5.4.2-3

$$T = 2\pi \sqrt{\frac{3194 \text{ kip}}{(2380 \text{ kip/in.})(32.2 \text{ ft./s}^2)(12 \text{ in./ft.})}} = 0.37 \text{ sec.}$$

Step 4: Calculate the equivalent static earthquake loading  $p_e$ .

$$p_e := S_a \frac{W}{L}$$

Refer back to the portion of the example where the response spectrum curve was developed.  $T$  is equal to 0.37 sec., which is less than  $T_s$  ( $T_s = 0.428 \text{ sec.}$ ). Since this is the case,

$$S_a = S_{DS} \quad \text{Eq. 3.4.1-7}$$

Evaluating the above expression yields,

$$S_a = 0.485$$

$$p_e = 0.485 \times \frac{3194 \text{ kip}}{178.5 \text{ ft.}} = 8.68 \text{ kip/ft.}$$

The portion of this force distributed to the piers is based on stiffness.

$$P_{\text{pier}} = p_e \times (K_{\text{pier}}/K_{\text{total}}) = 8.68 \text{ kip/ft.} \times (145.12 \text{ kip/in.} / 2380 \text{ kip/in.}) = 0.53 \text{ kip/ft.}$$

Step 5: Calculate displacements.

$$\Delta_{DL} = \Delta \times \frac{P_{\text{pier}}}{P_0}$$

$$\Delta_{DL} = 1.23 \text{ in.} \times \frac{0.53 \text{ kip/ft.}}{1 \text{ kip/ft.}} = 0.66 \text{ in.}$$

This displacement, multiplied by the embankment fill stiffness, should give us the force that will resist movement at the abutment. However, if our assumed initial stiffness was too large, this force could be larger than the abutment fill can support. Therefore, we will check it against the effective abutment stiffness bilinear model shown in *Caltrans Seismic Design Criteria*, Figure 7.14A.

The passive pressure force resisting movement at the abutment,  $P_{dia}$ , can be calculated using the following equation.

$$P_{dia} = A_e \times 5.0 \text{ ksf} \times \frac{h_{dia}}{5.5} \quad \text{Caltrans Eq. 7.44}$$

$$P_{dia} = (50 \text{ ft.} \times 4.9167 \text{ ft.})(5.0 \text{ ksf}) \left( \frac{4.9167 \text{ ft.}}{5.5 \text{ ft.}} \right) = 1098.82 \text{ kip}$$

The passive pressure force provided by the initial assumed stiffness and resulting displacement value is shown below.

$$P = \Delta_{DL} \times K_{fill} = 0.66 \text{ in.}(2234.86 \text{ kip/in.}) = 1475.01 \text{ kip}$$

$P$  is larger than  $P_{dia}$ . This is inconsistent with the assumed bilinear abutment stiffness relationship, as shown in Fig. 7.14A of the *Caltrans Seismic Design Criteria*.

*Seismic Specifications* Section 5.2.3.3.2 allows us to iteratively soften the embankment fill stiffness until the displacements are consistent with the assumed resisting force. After several iterations, the following values were obtained.

$$K_i = 22 \text{ kip/in/ft.}$$

$$K_{fill} = 983.33 \text{ kip/in.}$$

$$K_{total} = 1129.68 \text{ kip/in.}$$

$$T = 0.537 \text{ sec.}$$

$$S_a = 0.39$$

$$P_e = 6.92 \text{ kip/ft.}$$

$$P_{pier} = 0.89 \text{ kip/ft.}$$

$$\Delta_{DL} = 1.09 \text{ in.}$$

Our final values for  $\Delta_{DL}$  and  $K_{fill}$  give us a  $P$  of 1075.94 kip, which is very close to our  $P_{dia}$  of 1098.80 kip.

At this point, we will need to return to Article 4.3.3 to determine the magnification factor for short period structures. As you will recall, we skipped the step earlier since the period of the structure had not been determined. From previous calculations,  $T=0.537 \text{ sec.}$  and  $T_s=0.428 \text{ sec.}$

$$T^* = 1.25 T_s = 1.25(0.428 \text{ sec.}) = 0.535 \text{ sec.} \quad \text{Eq. 4.3.3-3}$$

$$\frac{T^*}{T} = \frac{0.535 \text{ sec.}}{0.537 \text{ sec.}} = 0.996 \leq 1.0$$

By Equation 4.3.3-2, since  $T^*/T$  is less than one,  $R_d = 1.0$ . Thus,  $\Delta_{DL}$  remains 1.09 in.

This completes the demand analysis, and we will return to Figure 1.3-4 in the flow charts.

The next box in Figure 1.3-4 refers us to Article 4.4, Combine Orthogonal Displacements.

#### 5.1.2.3.3 Combination of Orthogonal Seismic Displacement Demands:

Article 4.4 defines two load cases. Load Case 1 is 100% of the displacement computed in the previous section for the longitudinal direction combined with 30% of the previously computed displacement in the transverse direction. This will be taken as the final displacement demand in the longitudinal direction.

$$\text{Case1} = \sqrt{\Delta_{DL}^2 + (0.3 \cdot \Delta_{DT})^2}$$

Substituting the previously computed values into this equation yields that the Case 1 deflection is

$$\text{Case1} = \sqrt{1.09^2 + (0.3 \cdot 1.17)^2} = 1.15 \text{ in.} \quad \text{Longitudinal Direction}$$

Load Case 2 is 100% of the displacement computed in the previous section for the transverse direction combined with 30% of the previously computed displacement in the longitudinal direction. This will be taken as the final displacement demand in the transverse direction.

$$\text{Case2} = \sqrt{\Delta_{DT}^2 + (0.3 \cdot \Delta_{DL})^2}$$

Substituting the previously computed values into this equation yields the Case 2 deflection as

$$\text{Case2} = \sqrt{1.17^2 + (0.3 \cdot 1.09)^2} = 1.21 \text{ in.} \quad \text{Transverse Direction}$$

This completes the combination of orthogonal displacements and we return to the flow chart in Figure 1.3-4.

The next box in that figure refers us to Article 4.8, Determine Displacement Demands Along Member Local Axis.

#### 5.1.2.3.4 Determine Displacement Demands Along Member Local Axis.

Referring to Article 4.8, we find that the needed values are the ones we just computed, namely,



$$\Delta_{DL} = 1.15 \text{ in.}$$

$$\Delta_{DT} = 1.21 \text{ in.}$$

Going back to Figure 1.3-4, the next box refers us back to Figure 1.3-2. The point we left off in figure 1.3-2 refers us back to Figure 1.3-1b.

The box after the one we last completed in Figure 1.3-1b (Displacement Demand Analysis) refers us to Figure 1.3-3 for the computation of capacity.

Turn to Figure 1.3-3. Following that figure through to the first location where an action is required, we arrive at the box titled “SDC B & C Determine Delta C – Implicit.” This box refers us to Article 4.8.1 for determination of displacement capacity.

#### 5.1.2.4 Displacement Capacity:

From Article 4.8.1, the equation for displacement capacity for SDC B is Equation 4.8.1-1, which is:

$$\Delta_C = (0.12H_0)(-1.27 \ln(x) - 0.32) \quad \text{Eq. 4.8.1-1}$$

$H_0$  = Column Height (ft.)

$$x := \frac{\Lambda B_0}{H_0} \quad \text{Eq. 4.8.1-3}$$

$\Lambda$  = a factor for column end restraint  
=1 for fixed free (the longitudinal direction)  
=2 for fixed top and bottom (the transverse direction)

$B_0$  = Column Diam. (ft.)

Substituting into this equation with  $H_0$  = 24 ft. and  $B_0$  = 3.5 ft. yields:

$\Delta_C$  = 6.12 in. in the longitudinal direction and 3.59 in. in the transverse direction.

$\Delta_C$  must be greater than or equal to  $0.12(H_0)$ , which is 2.88.

$\Delta_C$  is thus 6.12 in. in the longitudinal direction and 3.59 in. in the transverse direction.

Returning back to Figure 1.3-3 and following it through from the box where we departed from it returns us to Figure 1.3-1b.

In figure 1.3-1b the box after Displacement Capacity asks us to compare displacement capacity to displacement demand.

#### 5.1.2.5 Compare Displacement Capacity to Displacement Demand:

The equation is

$$\Delta_c \geq \Delta_D$$

In the longitudinal direction, the displacement capacity is 6.12 in., which is greater than the displacement demand of 1.15 in. The requirement of the equation is thus met.

In the transverse direction, the displacement capacity is 3.59 in., which is greater than the displacement demand of 1.21 in. The requirement of the equation is thus also met in the transverse direction.

The capacity demand values determined above are a first trial and sufficient for the preliminary run. In the final design, the values will need to be corrected for shaft end displacements and rotations. The shaft displacement and rotation values will need to be obtained from the LPILE Analysis run by the Geotechnical Section. To insure that these values are obtained, the top of shaft loadings in both the transverse and longitudinal direction will need to be reported to the Geotechnical Section along with a request that the rotations and displacements be independently calculated in each direction.

Return now to the flow chart in Figure 1.3-1b. Since the displacement demand vs. capacity requirements have been met, we will proceed to the Satisfy Support Requirements box.

#### 5.1.2.6 Satisfy Support Requirements:

The first item in the box refers us to Article 4.12 for support lengths at expansion bearings. The pier bearings are fixed, so we will need to check support length at only the abutments.

##### Support Lengths at Abutments

The equation for support length presented in Article 4.12.2 is

$$N = (8 + 0.02 \cdot L + 0.08 \cdot H)(1 + 0.000125 \cdot S^2)$$

Following the definitions in the Article,

N = the support length (in.)

L = length of deck to adjacent expansion joint = 178.5 ft.

H = average height of columns (down to the point of fixity) supporting the bridge deck from the abutment to the next expansion joint = (8 ft. abutment + 24 ft. pier)/2 = 16 ft.

S = 0 degrees

Substituting these values into the equation, we get a support length requirement (N) of 12.85 in. Referring to Table 4.12.2-1 for bridges in SDC B, we need to multiply N by 150%. This results in a final support length requirement of 19.28 in.

Looking ahead to the preliminary abutment design on page 105 of this example, we will provide a support length of 24 in. to satisfy this requirement.

The next item in the box refers us to Article 4.14 for shear key design.

#### Shear Key

Shear key design will be undertaken during the final design phase.

Referring back to the flow chart in Figure 1.3.1-b, the next box refers us to Figure 1.3-5 for SDC B detailing.

#### **5.1.2.7 SBC B Detailing:**

Figure 1.3-5 defines three structure types. Examining the descriptions, it appears that Type 1 best fits our situation.

Following the Type 1 column of the flow chart, the box titled “Determine Flexure and Shear Demands” is the first box applicable to an SDC B bridge. The box refers us to Article 8.3.

#### Determine Flexure and Shear Demands

Article 8.3.2 addresses detailing requirements for SDC B bridges. That article states that the design forces for which detailing is to be determined shall be the lesser of the forces resulting from the plastic hinging moment capacity of the columns or the unreduced elastic seismic forces in the columns. Once the elastic seismic forces are calculated using RC-PIER, they will be compared to the plastic hinging moment capacity of the columns.

Article 8.3.2 also refers the reader to Articles 8.5 and 8.6 for a determination of column capacities. At this point in the design it would be premature to determine member capacities. That determination will be delayed until final design. For now, we will confine ourselves to the determination of design forces.

In our work under Displacement Demand, we determined  $p_e$  in both the transverse and longitudinal directions.

Those loads are

$$P_{e \text{ long}} = 6.92 \text{ kips/ft.}$$

$$P_{e \text{ trans}} = 6.50 \text{ kips/ft.}$$

The longitudinal and transverse seismic loads need to be combined using the same procedure from Article 4.4 as that for displacement.

$$\text{Case 1} = \sqrt{P_{e \text{ long}}^2 + (0.3 \cdot P_{e \text{ trans}})^2}$$

$$\text{Case 1} = \sqrt{6.92^2 + (0.3 \cdot 6.50)^2} = 7.19 \text{ kips / ft.}$$

Longitudinal Direction

$$\text{Case 2} = \sqrt{P_{e \text{ trans}}^2 + (0.3 \cdot P_{e \text{ long}})^2}$$

$$\text{Case 2} = \sqrt{6.50^2 + (0.3 \cdot 6.92)^2} = 6.82 \text{ kips / ft.}$$

Transverse Direction

In the longitudinal direction,

$$P_e = 7.19 \text{ kips/ft.}$$

The portion of this force distributed to the piers is based on stiffness.

$$P_{\text{piers}} = 7.19 \text{ kips/ft.} \times 178.5 \text{ ft.} \times (145.12 \text{ kip/in.} / 1129.68 \text{ kip/in.}) = 164.87 \text{ kips}$$

The force per pier is

$$P_{\text{pier}} = 164.87 \text{ kips} / 2 \text{ piers} = 82.43 \text{ kips/pier.}$$

Going back to our seismic design model in RC-PIER, input  $P_{\text{pier}}$  (82.43 kips) as Global Z direction bearing loads of 13.74 kips/ brng (82.43 kips / 6 bearing lines) under earthquake loads. Input 1.05 in the Multiplier for Loads box to account for the “Essential” importance factor.

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

**Loads: Load data**

**Bearing / Girder loads**

Bearing Line	Bearing Point#	Dir	Load (kips)
1	1	Z	13.7400
1	2	Z	13.7400
1	3	Z	13.7400
1	4	Z	13.7400
1	5	Z	13.7400
1	6	Z	13.7400
*			

Insert Copy Delete Delete All

**Column Loads / Settlement**

Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units

Insert Copy Delete Delete All

**Cap Loads**

Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units

Insert Copy Delete Delete All

**Strain Load**

Unit: 0.

+ Expansion - Contraction

Name: EQ1 Seismic

Description: Bearing Loads

**Factors**

Multiplier for Loads: 1.05

**Auto Generation**

Generate

**Import Loads**

Import

Note: Vertically downward loads be added as negative loads in Y direction.

OK Cancel

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

The following Analysis screen shows analysis results for the Extreme Event Seismic Group load case.

Membr	Node	Fx	Fy	Fz	Mx(Max/Min)	My	Mz
1	1	1.332	542.3	-35.54	-1171	-1.184	-13.76
		1.256	450.5	35.5	1039	1.224	-12.92
1	2	-1.181	-442	-35.5	-861.6	-1.224	6.682
		-1.257	-530.6	35.54	993.8	1.184	7.138
2	2	1.257	530.6	-35.54	-993.8	-1.184	-7.138
		1.181	442	35.5	861.6	1.224	-6.682
2	3	-0.5098	-426.8	-35.5	-116.2	-1.224	-8.547
		-0.5858	-509.4	35.54	247.5	1.184	-9.687
3	4	-0.7199	527.3	-33.54	-1147	-0.6429	7.974
		-0.7213	427.4	33.62	1016	0.6406	7.993
3	5	0.7213	-418.9	-33.62	-847.9	-0.6406	-4.386
		0.7199	-515.5	33.54	979.4	0.6429	-4.374
4	5	-0.7199	515.5	-33.54	-979.4	-0.6429	4.374
		-0.7213	418.9	33.62	847.9	0.6406	4.386
4	6	-0.2148	-468.5	-33.19	-148.5	0.2443	-2.959
		-0.2162	-559.2	33.12	281.6	-0.2421	-2.977
5	7	-0.6695	541.9	-35.51	-1171	1.109	8.068
		-0.5921	450.2	35.47	1039	-1.153	7.222
5	8	0.5921	-441.7	-35.47	-861.3	1.153	-4.261
		0.6695	-530.1	35.51	993.5	-1.109	-4.72
6	8	-0.6695	530.1	-35.51	-993.5	1.109	4.72
		-0.5921	441.7	35.47	861.3	-1.153	4.261
6	9	0.5921	-426.5	-35.47	-116.4	1.153	8.172
		0.6695	-509	35.51	247.7	-1.109	9.34
7	10	0	0	0	0	0	0
		0	0	0	0	0	0
7	11	0	8.19	0	0	0	-12.78
		0	8.19	0	0	0	-12.78

Note that the maximum moment at the top of the shaft is 994 ft.-kips. This is accompanied by a horizontal load of 36 kips and an axial load of 531 kips. These loads will be reported to the Geotechnical Section in the request for final foundation recommendations.

To determine which design forces will govern the detailing, the plastic hinging forces must be calculated according to Article 4.11.2.

Assuming  $\epsilon_{\text{plas.}} = 0.003$ ,

$$M_p = \frac{I_g \cdot E \cdot \epsilon}{y} = \frac{152745 \cdot 3320.56 \cdot 0.003}{21} = 72,457 \text{ kip} - \text{in.} = 6038 \text{ kip} - \text{ft.}$$

This value is greater than the maximum moment in the top of the shaft generated by RC-PIER. As mentioned above, the lesser of the plastic hinging forces and the unreduced elastic seismic forces will be used for detailing. Therefore, the elastic seismic forces govern the detailing.

Next, we will check the transverse direction.

$$P_e = 6.82 \text{ kips/ft.}$$

The portion of this force distributed to the pier is based on stiffness.

$$P_{\text{piers}} = 6.82 \text{ kips/ft.} \times 178.5 \text{ ft.} \times (1/3) = 405.79 \text{ kips/pier}$$

Going back to our seismic design model in RC-PIER, input  $P_{\text{pier}}$  as a Global X direction cap load under earthquake loads.

The maximum moment at the top of the shaft is 330 ft.-kips. This is accompanied by a horizontal load of 139 kips and an axial load of 659 kips. These loads will be reported to the Geotechnical Section in the request for final foundation recommendations.

#### **5.1.2.8 Foundation Design:**

Looking over Figure 1.3-6 reveals that the only applicable requirement is Article 6.5, Drilled Shaft.

##### *Drilled Shaft*

Article 6.5 lists the following requirements for drilled shafts. The disposition of the requirement as it pertains to this example will follow the presentation of each requirement.

- Drilled shaft design is to conform to the seismic design requirements for columns. Shaft design for this example will conform to those requirements.
- The effects of streambed aggradation or degradation shall be taken into account when establishing point of fixity in the shaft. Neither aggradation nor degradation are anticipated at this location.
- The effects of liquefaction are to be considered. As discussed earlier, liquefaction will not be a consideration at this site.
- A stable length will be established for the shaft. This is a geotechnical consideration that will be met.
- The ultimate geotechnical capacity of the shaft will not be exceeded by seismic loads. Again, this is a geotechnical consideration that will be met.

With that discussion, we return to the flow charts (Figure 1.3-1b), which indicate that we are through, at least for preliminary design, with seismic considerations.

#### **5.1.2.9 Summary of Preliminary Seismic Design:**

The focus of our work so far in seismic design has been to check the adequacy of the preliminary pier geometry we have selected to resist earthquake loading, and to determine seismic forces and displacement with sufficient accuracy to allow the Geotechnical Section to complete their analysis.

Seismic work still to be completed under final design will be to add the effects of shaft end displacements and rotation to the displacement demand figure we have calculated, reassess seismic forces if necessary based on a revised point of fixity, and check the requirements for SDC B level of detailing.

With that, we will leave seismic design for now and return to determining forces for the 500-year flood event.

### **5.1.3 Run for 500-Year Flood (Extreme Event Group II)**

The last step in the preliminary pier design is to check the ability of the pier to withstand the loading of the 500-year flood. From Table 3.4.1-1 of the *LRFD Specifications*, the Extreme Event Group II load case will be used.

The stream flow data for the 500-year flood are as follows:

$V = 12 \text{ ft./sec.}$

Scour depth = 15 ft.

High water elevation = bottom of pier cap

The load combination will be Extreme Event Group II. To verify the adequacy of the design, an RC-PIER run will be made with the appropriate load combination, and the resultant forces will be compared to the ultimate capacities of the members.

Referring to Article 3.7.3 of the *LRFD Specifications*, the stream flow pressure acting on the structure in ksf is given by

$$p = \frac{C_D V^2}{1,000}$$

The drag coefficient for the columns and shafts is 0.7 from Table 3.7.3.1-1.

Using this value, the pressure acting on the columns is 0.101 ksf or 0.35 kips/ft.

Likewise, the stream flow load acting on the shaft is 0.404 kips/ft.

The pressure acting on debris lodged against the column is calculated using Figure C3.7.3.1-1 and the associated commentary. In the commentary,  $C_D=0.5$ . With the 500-yr flow velocity, we get a stream flow pressure of 0.072 ksf.

We will turn this pressure into a force acting at the centroid of the debris raft. To do this, we will need to calculate dimensions A and B from Figure C3.7.3.1-1. The depth, A, should be half the water depth, but not greater than 10 ft. The width, B, should be the length of the superstructure tributary to the pier, but not greater than 45 ft.



$$A = \frac{(19 + 15)}{2} = 17 \text{ ft.} > 10 \text{ ft.}$$

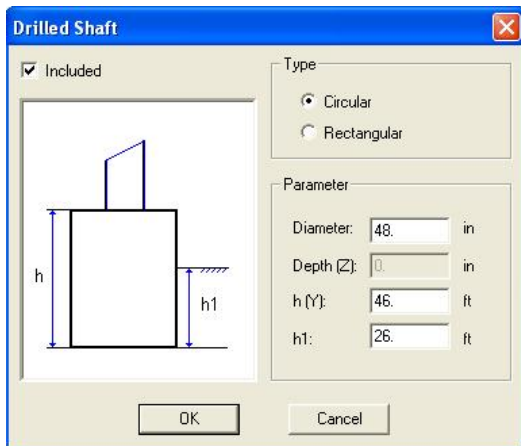
$$B = \frac{(43.75 + 88)}{2} = 65.875 \text{ ft.} > 45 \text{ ft.}$$

Since A and B are both greater than their allowable lengths, we will use A = 10 ft. and B = 45 ft. The result is a force of 16.2 kips acting at the centroid of the debris raft, which is 3.33 ft. from the top of the column.

For this run, the location of maximum moment in the drilled shaft will be assumed at 5 ft. below the 500-year scour depth or 20 ft. below streambed elevation.

With that information, we are ready to go to RC-PIER again.

Going back to the Column Geometry, ensure that the column diameter is again set to 42 in. Next, in the Drilled Shaft screen for each column, change the location of the point of fixity to 20 ft. below streambed as shown below. Click OK.



**EXAMPLE NO.1: Concrete Bridge  
LRFD Specifications**

Bring up the Loads screen and input the stream flow loads by editing the WA1 load to match the screen shown below.

**Loads: Load data**

**Bearing / Girder loads**

Bearing Line	Bearing Point#	Dir	Load (kips)
▶			

Insert Copy Delete Delete All

**Strain Load**

Unit: 0.

+ Expansion - Contraction

Name: WA1 Water

Description: 500-yr flood

Factors: Multiplier for Loads: 1.

Auto Generation: Generate

Import Loads: Import

Note: Vertically downward loads be added as negative loads in Y direction.

OK Cancel

**Column Loads / Settlement**

Col Nr	Load Type	Dir	Mag1	y1/L	Mag2	y2/L	Units
1+Shaft	Pressure	X	0.1010	0.1300	0.0000	1.0000	ksf
▶ 1+Shaft	Force	X	16.2000	0.9150	0.0000	0.0000	kips
*							

Insert Copy Delete Delete All

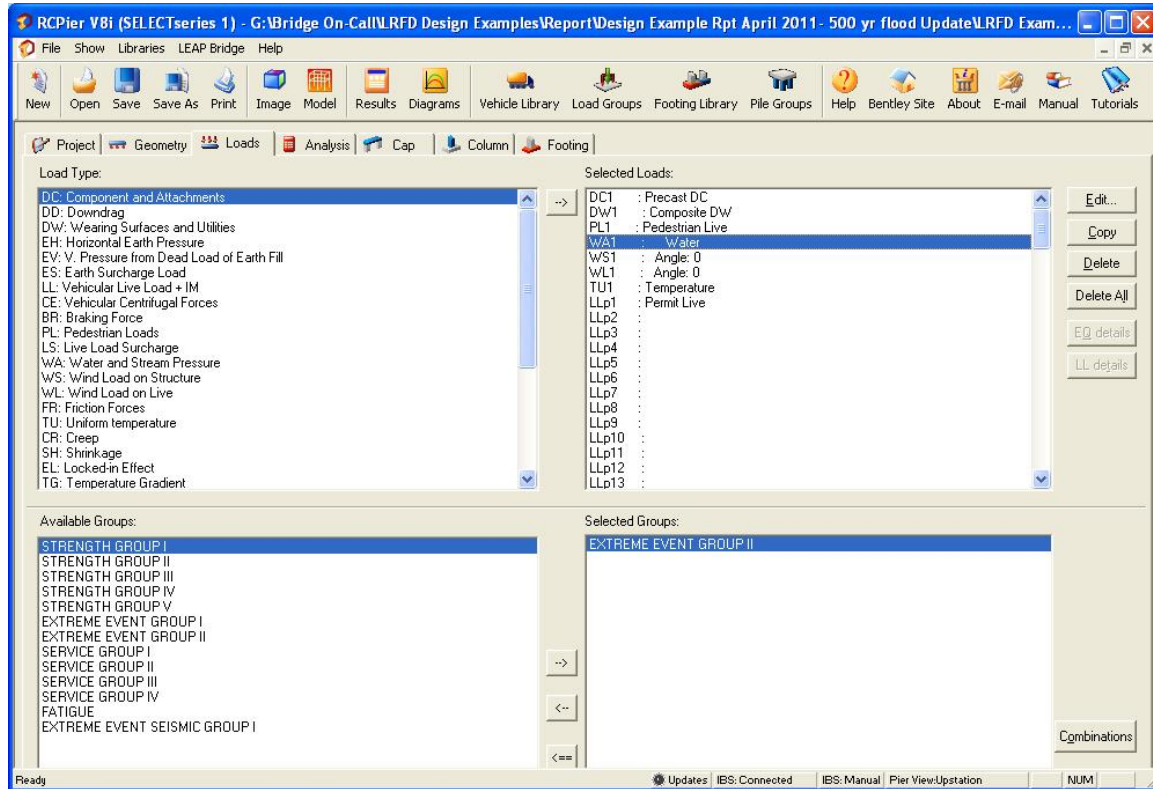
**Cap Loads**

Load Type	Dir	Arm (ft)	Mag1	x1/L	Mag2	x2/L	Units
▶							

Insert Copy Delete Delete All

## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

Modify the Loads screen as shown below so that only Extreme Event Group II will be calculated.



## EXAMPLE NO.1: Concrete Bridge LRFD Specifications

Click on the Analysis tab and run the analysis. The results are shown below. An examination of these forces reveals that all members are well below their capacity.

Memb	Node	Fx(Max/Min)	Fy	Fz	Mx	My	Mz
1	1	-15.78	475.4	6.612	383.3	-0.9256	266.4
		-12.81	553.8	6.619	409.4	-0.9358	220.6
1	2	6.878	-506.7	-6.619	-277.1	0.9358	-7.998
		9.854	-441.5	-6.612	-251.1	0.9256	5.793
2	2	-9.854	441.5	6.612	251.1	-0.9256	-5.793
		-6.878	506.7	6.619	277.1	-0.9358	7.998
2	3	-16.75	-472.4	-6.619	-138.1	0.9358	2.046
		-13.77	-416.8	-6.612	-112.2	0.9256	78.33
3	4	-9.055	438.7	-4.836	358.5	0.01478	206.9
		-6.53	613.2	-4.847	377.9	0.01527	158.6
3	5	6.53	-566.1	4.847	-282.6	-0.01527	-28.01
		9.055	-404.8	4.836	-262.9	-0.01478	-25.82
4	5	-9.055	404.8	-4.836	262.9	0.01478	25.82
		-6.53	566.1	-4.847	282.6	0.01527	28.01
4	6	6.53	-531.8	4.847	-182.5	-0.01527	109.1
		9.055	-380.1	4.836	-162.6	-0.01478	164.3
5	7	-8.951	570.4	6.616	409.4	0.968	199.8
		-5.978	492	6.61	383.2	0.9564	154.1
5	8	5.978	-458.1	-6.61	-251	-0.9564	-34.52
		8.951	-523.3	-6.616	-277	-0.968	-20.77
6	8	-8.951	523.3	6.616	277	0.968	20.77
		-5.978	458.1	6.61	251	0.9564	34.52
6	9	5.978	-433.4	-6.61	-112.2	-0.9564	91.02
		8.951	-489	-6.616	-138.1	-0.968	167.2
7	10	0	0	0	0	0	0
		0	0	0	0	0	0
7	11	0	8.19	0	0	0	-12.78
		0	8.19	0	0	0	-12.78

### 5.1.4 Conclusion of Preliminary Design

With the check of the 500-year flood, the preliminary design work for the pier is concluded, and we are now ready to transmit shaft loads to the Geotechnical Section for final foundation recommendations. These loads, acting at the top of shaft, are as follows (referring to the Load summary for each RC-PIER run):

#### Strength Groups

- Axial Load = 749 kips
- Horizontal load = 20 kips
- Moment = 923 ft.-kips

Scour depth is 10 ft.

Extreme Event Seismic

Transverse

- Axial Load = 659 kips
- Horizontal load = 139 kips
- Moment = 330 ft.-kips

Longitudinal

- Axial Load = 531 kips
- Horizontal load = 36 kips
- Moment = 994 ft.-kips

Scour depth is 0 ft.

Extreme Event II (500-yr flood)

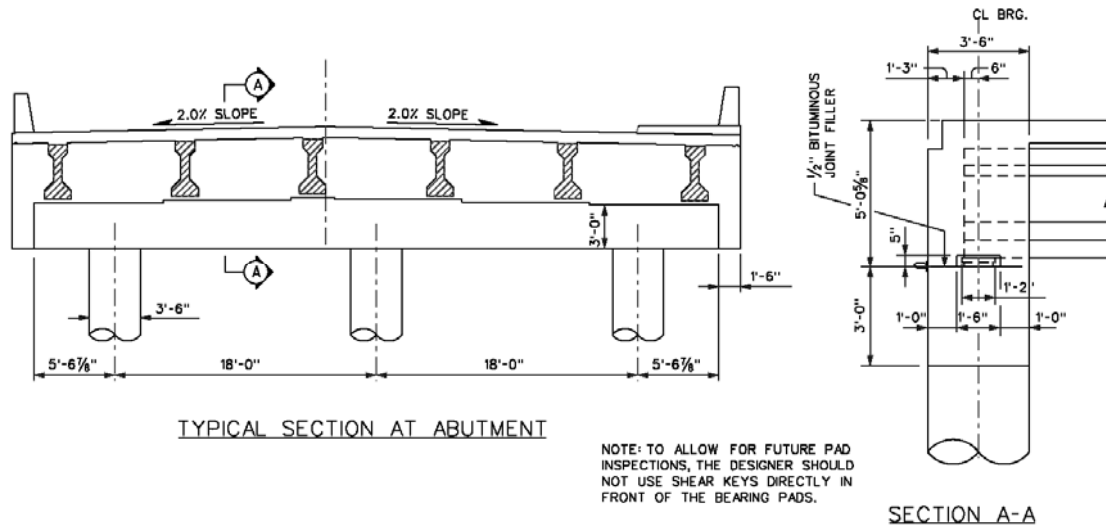
- Axial Load = 566 kips
- Horizontal load = 7 kips
- Moment = 283 ft.-kips

Scour depth is 15 ft.

In the transmittal to the Geotechnical Section we will need to state that (1) these loads are factored loads, (2) they are acting at the top of shaft elevation and (3) we want a recommendation on shaft length and an LPILE analysis for each load case.

### 5.1.2 Preliminary Abutment Design

Preliminary abutment configuration is shown below.



Abutment analysis and design, like the pier analysis, can readily be completed using RC-PIER. The process would very closely follow the one used for pier design, so closely, in fact, that presenting it here would be quite redundant. For that reason, a complete abutment design will not be presented. Rather, the differences between an abutment and a pier design will be discussed. Those differences are as follows:

1. For stream crossing bridges, it is generally accepted that the abutments are protected from scour. For that reason stream flow forces will not be applied to abutments in most cases. Since the high water elevation is at the bottom of the pier cap, we will not need to investigate stream flow forces.
2. The depth to fixity (location of maximum moment) will be much shallower than for a pier. A good first assumption would be 10 ft. below the bottom of the abutment cap.
3. Depending on the abutment configuration, earth pressure loads may need to be applied.
4. Seismic analysis will be required since we have included contribution from the abutments.

In an actual design situation, though, loading similar to that computed for the pier would be determined and submitted to the Geotechnical Section for their use in preparing the foundation report.