LOAD RATING THE I. B. PERRINE BRIDGE

PHASE I

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16. Abstract (no more than 250 words)

The I.B. Perrine Bridge spans across the Snake River on US 93 north of Twin Falls, Idaho. The bridge carries a significant volume of truck traffic and the Idaho Transportation Department receives regular requests for permits to run overweight and other over-permit trucks across this span. Current bridge rating procedures make typical assumptions for load distribution factors and load paths that only approximately match the geometry of the bridge. To more accurately estimate load demands on the structure, the University of Idaho has created a three-dimensional, finite element model of the I.B. Perrine Bridge using LARSA 2000 Plus. Different vehicles can be simulated moving across the bridge deck and the resulting member reaction envelopes are calculated. In addition, an Excel worksheet has been created which imports analysis data generated in LARSA and calculates and summarizes load rating factors for the various structural components of the bridge. Load rating is performed using a LRFR strength limit state for permit vehicles. The finite element analysis and load rating program can be operated efficiently. A concise load rating summary report is generated so that an engineer can clearly determine if permit vehicles can safely cross the bridge. Once the finite element model is calibrated, the Idaho Transportation Department will be able to use this software to more accurately load rate the bridge for permit vehicles.

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ABSTRACT

The I.B. Perrine Bridge spans across the Snake River on US 93 north of Twin Falls, Idaho. The bridge carries a significant volume of truck traffic and the Idaho Transportation Department receives regular requests for permits to run overweight and other over-permit trucks across this span. Current bridge rating procedures make typical assumptions for load distribution factors and load paths that only approximately match the geometry of the bridge.

To more accurately estimate load demands on the structure, the University of Idaho has created a three-dimensional, finite element model of the I.B. Perrine Bridge using LARSA 2000 Plus. Different vehicles can be simulated moving across the bridge deck and the resulting member reaction envelopes are calculated. In addition, an Excel worksheet has been created which imports analysis data generated in LARSA and calculates and summarizes load rating factors for the various structural components of the bridge. Load rating is performed using a LRFR strength limit state for permit vehicles.

The finite element analysis and load rating program can be operated efficiently. A concise load rating summary report is generated so that an engineer can clearly determine if permit vehicles can safely cross the bridge. Once the finite element model is calibrated, the Idaho Transportation Department will be able to use this software to more accurately load rate the bridge for permit vehicles.

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1 INTRODUCTION

1.1 Background

1.1.1 I.B. Perrine Bridge

The I.B. Perrine Bridge spans 1500 feet across the Snake River on US 93 north of Twin Falls, Idaho and is composed of a 1000-foot steel trussed-arch main span with one- and two-span plate girder approach spans (see Figure 1 - 1). It is an important transportation connection between northern Nevada and I-84, the principal east-west interstate highway in southern Idaho. The bridge carries a significant volume of truck traffic and the Idaho Transportation Department (ITD) receives regular requests for permits to run overweight and other over-permit trucks across this span. Current bridge rating procedures make typical assumptions for load distribution factors and load paths that only approximately match the geometry of the bridge.

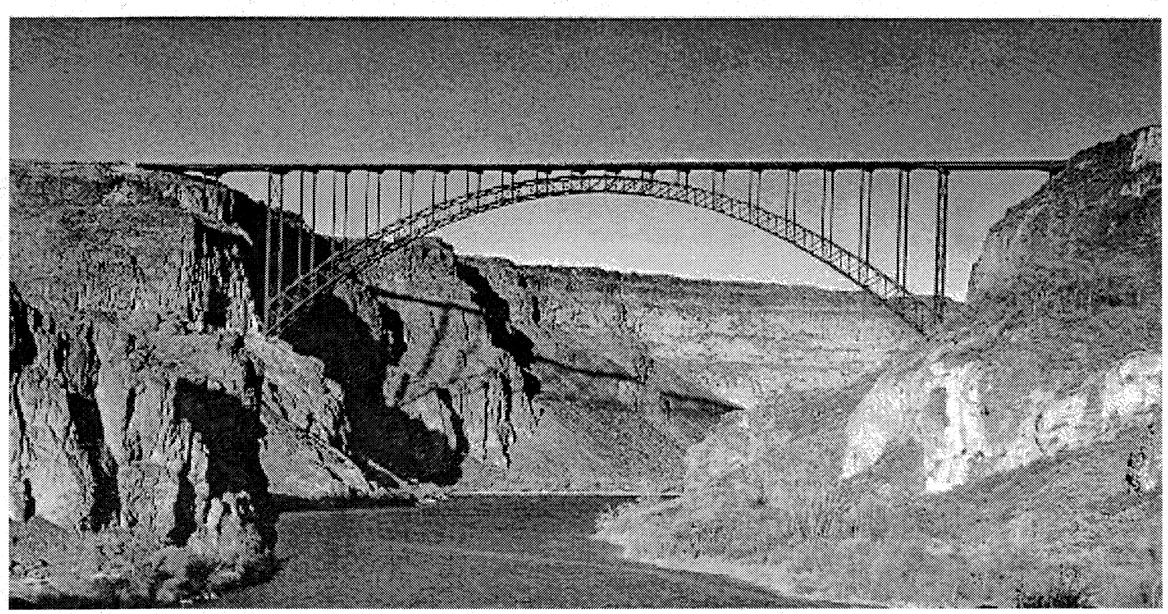


Figure 1 - 1: I. B. Perrine Bridge

1.1.2 Load and Resistance Factor Rating

Load and Resistance Factor Rating (LRFR) is a procedure adopted by the American Association of State Highway and Transportation Officials (AASHTO) for evaluating the effect of trucks on in-service bridges. LRFR is based on structural reliability concepts, similar to Load and Resistance Factor Design (LRFD), which are intended to ensure a more consistent level of reliability for bridge performance. There are three types of load rating: design, legal, and permit. This paper considers the permit load rating case. Permit load rating is required for vehicles that exceed the typical loads and axle configurations considered in bridge design.

1.2 Standard Live Load Distribution Factors

Vehicle loads are applied directly to the bridge deck and then are transferred to the balance of the bridge superstructure. Bridge engineers commonly use live load distribution factors to estimate live load effects in beams supporting bridge decks. A live load distribution factor simplifies structural analysis of vehicle loads by representing the transverse effect of vehicle wheel loads as a fraction of the wheel loads applied directly to beams or other supporting members. Customary design practice calculates transverse live load distribution factors according to either the AASHTO standard specifications or the AASHTO LRFD bridge design specifications (Huo et al., 2004). These two methods are discussed in Sections 1.2.1 and 1.2.2, respectively.

1.2.1 AASHTO Standard Specifications

The AASHTO standard specifications calculate live load distribution factors as a function of girder spacing and bridge type (Zokaie, 2000). These live load distribution factors are commonly represented as follows:

$$DF = S/D$$

where *DF* equals the live load distribution factor, *S* equals the girder spacing, and *D* equals a constant based on the type of bridge superstructure (Zokaie, 2000). Further investigation of these formulas determined that they generate valid results for bridges of typical geometry (i.e., girder spacing near 6 feet and span length of 60 feet), but lose accuracy rapidly when the bridge parameters are varied (e.g., when relatively short or long bridges were considered) (Zokaie, 2000). The geometry of the I.B. Perrine Bridge does not fall within the applicable range for these types of live load distribution factors.

1.2.2 AASHTO LRFD Bridge Design Specifications

Recognizing the need to gain higher accuracy, the distribution factors contained in the AASHTO LRFD bridge design specifications are more complex and include more parameters than the AASHTO standard specifications. The key parameters used to determine live load distribution factors are girder spacing (S), span length (L), girder stiffness (K_g), and slab thickness (t) (Zokaie, 2000). The new formulas are generally considered more accurate than the AASHTO standard specifications because their results have been verified with finite element analysis and have been calibrated against a database of existing bridges of varied bridge types with certain ranges of bridge span length, moment of inertia of beams, beam spacing, and so on (Huo et al., 2004). There are limitations to the new formulas that an engineer must consider. The database of bridges used to develop the formulas had uniform spacing, girder moments of inertia, and skew (Zokaie, 2000). Additionally, the AASHTO LRFD formulas are based on primary structural components; cross-frame effects were not considered (Zokaie, 2000).

1.3 Finite Element Modeling

1.3.1 Need for a Finite Element Model

1.3.1.1 Geometric Applicability

The parameters of the Perrine Bridge are outside of the range of applicability described for the AASHTO LRFD distribution factor formulas. The approach spans, comprising approximately one-third the total bridge length, are a structure type not considered by the AASHTO LRFD live load distribution factor formulas. Furthermore, the girder moments of inertias of primary structural beams in both the approach spans and main arch span are not uniform longitudinally. Plate girders of the approach spans have significant haunches near the bents and the girders of the main arch span contain haunches at spans adjacent to expansion joints. Likewise, the moments of inertia of the primary structural beams of the approach spans vary in the transverse direction. The significantly smaller stringers of the approach spans are ultimately supported by larger plate girder beams both of which directly support the deck. If the AASHTO LRFD live load distribution factors were used, an engineer would be required to make assumptions that exceed the limits of applicability of the distribution factors in order to estimate live load effects on the bridge (Huo et al., 2004).

By contrast, finite element analysis directly calculates the load effects of, say, a specific truck on bridge elements. Finite element analysis of a bridge is generally accepted as an accurate analysis method (Zookaie, 2000). For live load analysis of permit trucks, Zookaie suggests that finite element analysis can, in most cases, be performed to calculate more accurate distribution factors than the LRFD formula results.

1.3.1.2 Permit Load Applicability

Even on bridges which satisfy the assumptions used by the AASHTO LRFD live load distribution factors, permit loads may not be accurately represented by these live load distribution factors. Since the distribution factors assume that all lanes are loaded by similar trucks, they may be too conservative (Zookaie, 2000). It is possible that permit vehicles may have axle widths greater than 6 feet, which is not accounted for by the distribution factors (Tabsh and Tabatabai, 2000). As a result, a finite element analysis may be needed to verify the safety of the structure subjected to permit loads.

1.3.1.3 Advantage of Finite Element Model

It is known that frequent heavy and/or permit loads can reduce a bridge's life or cause permanent structural damage if not evaluated and regulated properly (Jaramilla and Huo, 2005). Additionally, overly conservative rating factors can lead to either an increase in costly traffic restrictions on the one hand or unnecessary bridge strengthening or repairs on the other (Jaramilla and Huo, 2005). Therefore the ability to determine accurate load rating factors appeals to both the safety and economic interests of bridge owners. A finite element model is used to estimate load effects throughout the bridge because the finite element method can provide more accurate results when AASHTO LRFD formulas are not directly applicable, especially for permit load analysis.

1.3.2 Finite Element Model Calibration

On complicated structures it is critical that the finite element model be verified with diagnostic testing to ensure that the finite element results are representative of the actual bridge response. Combining a diagnostic "semi-static" load test with finite element modeling can quantify bridge behavior for use in determining load ratings (Schulz et al., 1995). The calibration process uses measured structural response obtained from controlled diagnostic load testing to estimate boundary conditions related to support, geometry, and stiffness (Yost et al., 2005). Calibrated finite element models have been successfully implemented for the purpose of load rating in numerous states throughout the United States (Chajes et. al., 1997; Commander and Schulz, 1997; Phares et al., 2005). Typically, calibrated finite element models have been developed for simple span bridges ranging from short to medium spans. Although calibrated continuous, large span bridge models are not as common, prior successes in the area of finite element model calibration provide a broad range of applications to reference so that larger model calibration may be possible.

Controlled "semi-static" (low-speed) loads tests were conduced on the Perrine Bridge in October, 2005. Two sets of tests were conducted. During the first set of tests, strain gages were attached to key stringers, girders and arch members on the north approach span. During the second set of tests, strain gages recorded the strains in similar members near the center of the arch. Data from these tests has been briefly examined and archived by the authors. However, unexpected challenges in the development of the finite element model prevented the completion of the calibration process.

1.4 Report Outline

Section 2 describes the creation of the three-dimensional, finite element model of the Perrine bridge using LARSA 2000 Plus. All structural bridge components including the arch truss, diaphragms, and secondary bracing are included in the computer model. Unlike either of the previously discussed AASHTO distribution factors, the finite element model incorporates the effects of decks, diaphragms and secondary bracing while estimating bridge member response. LARSA 2000 Plus directly calculates static dead loads and moving load envelopes for all bridge members. Section 3 explains how member design capacities are calculated and load rating factors are determined. Section 4 presents the procedure for integrating the results of the finite element model with the load rating of the Perrine Bridge using Excel and Visual Basic. All bridge components are load rated using this method. Due to the large size of the bridge and number of structural components on the bridge, critical rating factors are sorted and summarized in a load rating report. Bridge engineers will be able to use the computer model and load rating program to load rate the bridge for permit vehicles on a routine basis.

2 FINITE ELEMENT MODEL

2.1 Model Mechanics

The LARSA finite element model formulates stiffness characteristics and equations of force equilibrium based on the bridge geometry presented in the I.B. Perrine Bridge plans provided by the Idaho Transportation Department (ITD). The LARSA model was checked by a second analyst to ensure the accuracy of the element geometry, material, orientation and loads. The various components of the finite element model and relevant assumptions used to model structural members are presented in Sections 2.1.1 through 2.1.6.

2.1.1 Material Properties

There are five isotropic materials used in the finite element model: A514 Steel, A588-Gr 50 Steel, 4000 psi Concrete, Rigid Link, and Lane. The first three materials are real materials, and the latter two are required for analysis purposes (i.e., virtual materials). Typical strength properties of A514 Steel, A588-Gr 50 Steel, and 4000 psi concrete are used in the finite element program (Gere, 2001). Both upper and lower arch truss chord members are A514 Steel; all other beam elements modeling structural members are A588-Gr 50 Steel. Plate elements modeling the concrete bridge deck are 4000 psi concrete. The concrete bridge deck contains steel reinforcement; however, the effects of the steel reinforcing have conservatively been neglected. There are two different types virtual beam elements in the finite element model which use the rigid link and lane materials. Virtual material properties are the same as that of A588-Gr 50 Steel, except their unit weight is equal to zero. The definition and function of both rigid link and lane members are discussed in Sections 2.1.6 and 2.3.3, respectively. Table 2 - 1 summarizes material properties used in the finite element program.

Table 2 - 1: Material Strength Properties

Material Property	Material Type				
	Steel A514	Steel A588-Gr 50	Concrete 4000 (psi)	Rigid Link	Lane
Modulus of Elasticity (kip/in^2)	29000	29000	3605	29000	29000
Poisson Ratio (-)	0.295	0.295	0.170	0.295	0.295
Shear Modulus (kip/in^2)	11200	11200	1541	11200	11200
Unit Weight (lb/in^3)	0.284	0.284	0.087	0.000	0.000
Yield Stress, F_y (kip/in^2)	100	50	0	50	50
Concrete f'_c28 / Steel F_u (kip/in^2)	110	70	4	70	50

2.1.2 Prismatic Beams

The section properties of a prismatic beam remain constant over the length of the beam. There are approximately three thousand beam elements modeling prismatic structural members on the Perrine Bridge. Prismatic beam sections include both rolled and built-up sections – a total of ninety-three section types. Section names and properties used for modeling prismatic beams are in Appendix A. The subsequent sections define a beam element and the varying applications of the beam element in the finite element model.

2.1.2.1 Beam Element

The beam element models a variety of structural members ranging from continuous bending members to two-force members. A beam element has six degrees of freedom (DOF) at each end joint: translational displacements in local *X*, *Y*, and *Z* directions and rotational displacements about local *X*, *Y*, and *Z* axes. Beam elements include axial and shear deformations, twisting about its x-axis, bending in two perpendicular planes, and associated shears (LARSA, 2004). These deformations are defined based on the local axes of the bending element.

The forces imparted from one member to another depend on the fixity of the connection between two members. Moment fixity can be quantified as the percent of applied moment transferred from the end of one beam element into the end of another beam element. Zero percent fixity corresponds to a perfect hinge (i.e., free rotation) and one hundred percent fixity means a fixed rotation connection. In reality, connections rarely act as either perfect hinges or fixed connections; their behavior lies somewhere between the two extremes. Beam element fixity assumptions are consistent with customary design practice. Unless stated otherwise, bolted connections are assumed to be pinned, i.e., no moments are transferred through these connections or zero percent fixity.

Beams in structures are often modeled as multiple beam elements connected along the length of the physical beam (see Section 4.2.1). These beam elements are known as analytical elements, as opposed to physical members because they are defined for the sake of analysis (LARSA, 2004). For example, numerous analytical elements are required to model a plate girder to allow for changes in section properties over the length of the girder; to provide connections for regularly spaced floor beams attached to the girder web; and to connect plate elements to girder joints at various intervals along the beam length. Where two or more analytical elements are used to model a real beam, the connection between the analytic elements is assumed fixed, i.e., internal beam moments are resisted. Bolted splices in beams are assumed to behave as fixed connections.

Beam elements for flexural beam modeling and truss modeling are discussed in subsections 2.1.2.2 and 2.1.2.3, respectively.

2.1.2.2 Flexural Beams

The following structural members require analytical elements to model the entire beam span:

- Plate girders
- Approach stringers

- Approach floor beams
- Main arch span stringers
- Main arch span floor beams
- Lateral cross bracing (south approach and main arch span)
- Spandrel columns C_o and C'_o
- Lateral cross bracing in columns C_o and C'_o
- Upper and lower arch struts

Figure 2 - 1 shows the analytical elements for a portion of a typical approach span; elements are reduced twenty percent to emphasize model geometry. Figure 2 - 1 indicates that numerous elements are required to model structural beams. Structural bending members of the main arch span are similarly partitioned in the finite element model.

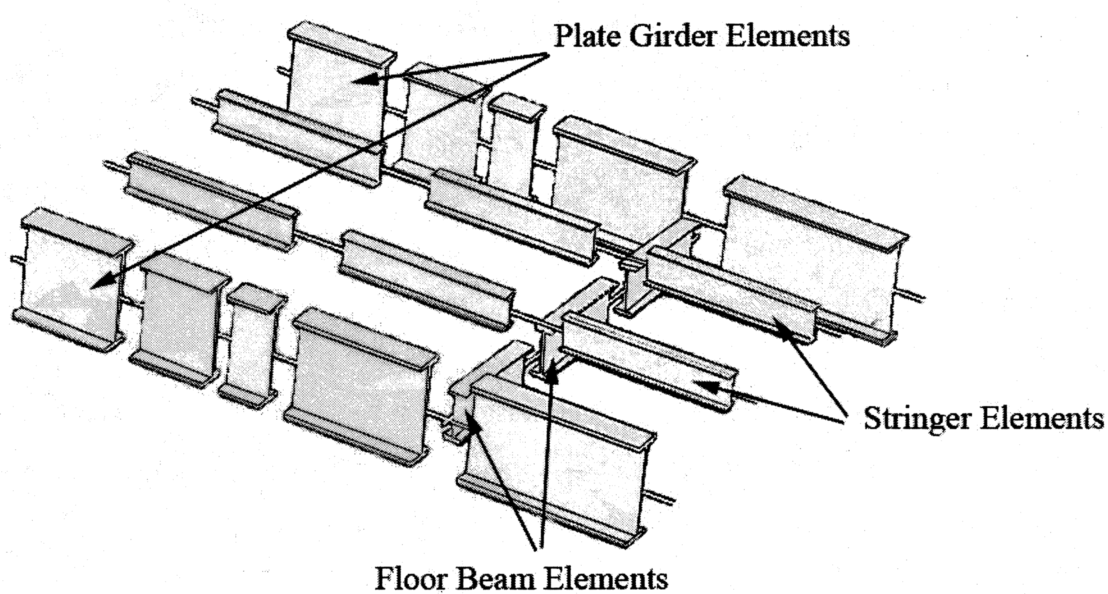


Figure 2 - 1: Bending Elements of Typical Approach Span

Figure 2 - 2 illustrates spandrel columns of the main arch span that are likewise modeled using multiple analytic elements. Lateral brace elements are also shown in Figure 2 - 2.

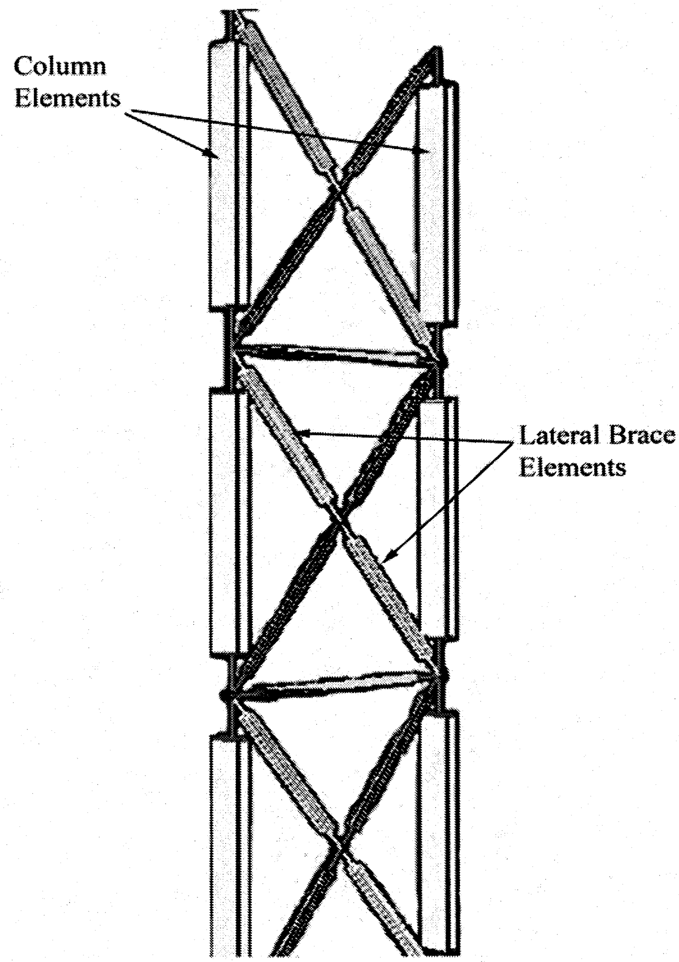


Figure 2 - 2: Analytical Elements of Spandrel Column

Non-prismatic beams also require analytical elements; they are described in further detail in Section 2.1.3.

2.1.2.3 Two-Force Members

All members of the arch truss are modeled as two-force members except the upper and lower struts. The end-loaded diaphragm members located in the bridge deck are also two-force members. Two-force members have pinned connections at both ends allowing rotations about the strong and weak axes. When a translation or rotation DOF is free, the corresponding internal reaction is equal to zero (LARSA, 2004). Therefore, two-force members do not support either strong or weak axis moments. Moments about the local x-axis (i.e., torsion) are resisted by two-force members. On the arch truss, where pin-pin members intersect at a joint, the joint is externally restrained against rotations about all axes since the members themselves do not necessarily restrain rotations.

Spandrel columns that predominately resist axial compression have pinned connections where columns intersect the arch truss and fixed connections where columns intersect the floor beams of the main arch span. Similarly, bent columns have pinned connections at the ground level and fixed connections where columns are attached to bent floor beams. Columns are the only structural members with bolted connections modeled as fixed connections because all four sides of the box sections are bolted to the floor beams.

2.1.3 Non-Prismatic Beams

There are four locations on the Perrine Bridge containing haunched girders: the south approach bent, north approach bent, and at both expansion joints between the main arch span and approach spans. The girders are I-sections whose web height varies parabolically over the haunched length. Stiffness characteristics of haunched girders continuously change along the length of the beam due to the varying web height. Parabolically haunched girders are modeled as stepped analytic elements since tapered elements were not available for modeling. A stepped element model divides a tapered member into constant-height analytic sub-elements using the average web-height of the tapered sub-elements to define the section properties of each prismatic sub-element. Flange thicknesses remain constant for each analytic element. Figure 2 - 3 shows the actual dimensions for an exterior south approach girder and Figure 2 - 4 illustrates the stepped-element representation of the same girder, with beam element lengths reduced twenty percent to emphasize model geometry.

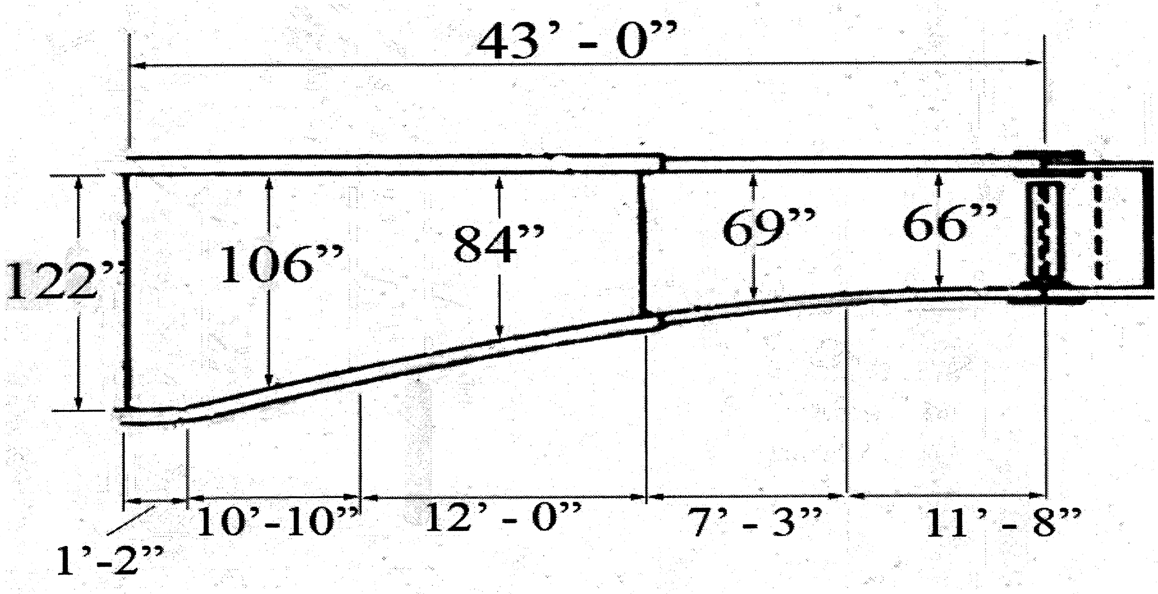


Figure 2 - 3: Exterior South Approach Haunch Girder

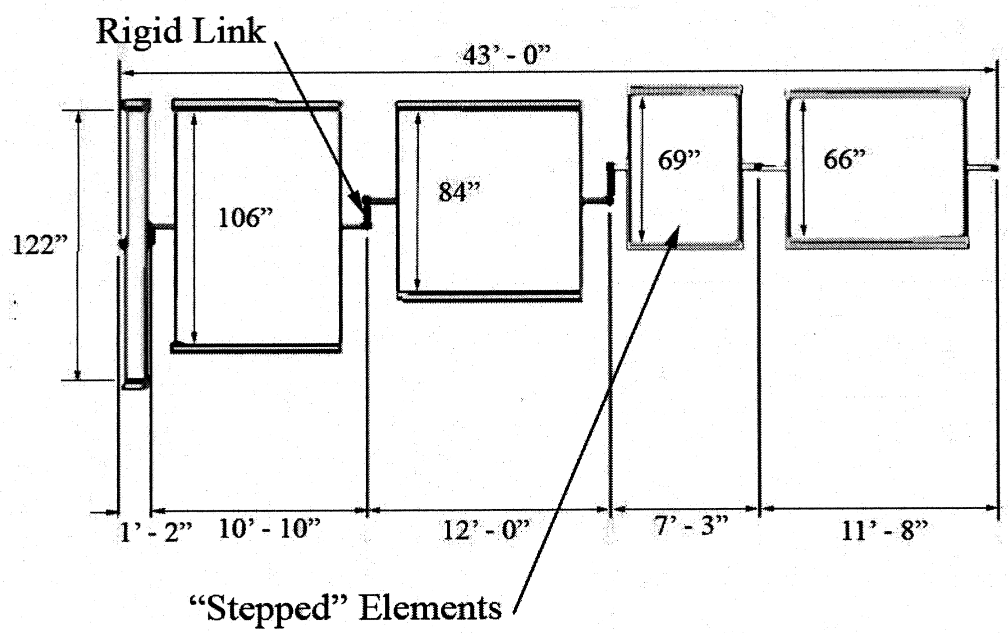


Figure 2 - 4: Stepped Element Model of Exterior South Approach Girder

The elevation of the top flange of the haunch girder must be nearly constant – flush with the deck bottom elevation – while the web height changes. Therefore, the elevation of the mid-height of the beam must change along the length of the beam (see Figure 2 - 4). Since all elements are connected at mid-height, rigid links are used to bridge the gaps between these analytic beam elements. Rigid link elements are discussed in Section 2.1.6.

The south approach, north approach, and main arch span haunch girders are modeled with five, four, and six sub-elements respectively. The range of web heights for parabolic haunches are summarized in Table 2 - 2. H_1 is the shortest web height, H_2 is the deepest web height, and ΔH is the change in web height.

Table 2 - 2: Actual Web Heights for Parabolic Haunch Plate Girders

Web Height (in)	Location of Haunch Girder					
	South Approach	North Approach	South Main Arch Span		North Main Arch Span	
			Interior	Exterior	Interior	Exterior
H_1	60.0	54.0	31.0	31.5	31.0	31.5
H_2	122.0	105.0	61.5	61.5	56.0	56
ΔH	62.0	51.0	30.5	30	25.0	24.5

The web heights of the prismatic sub-elements modeling these parabolic haunches are summarized in Table 2 - 3.

Table 2 - 3: Sub-Element Web Heights for Stepped Element Models

Location of Haunch Girder		Element Web Heights (in)					
		1	2	3	4	5	6
South Approach		66.09	69.19	84.19	106.25	122.00	-
North Approach		57.75	70.81	94.12	105.00	-	-
South Main Arch Span	Interior	31.50	32.09	35.78	42.41	53.72	61.50
	Exterior	31.00	31.59	35.34	42.09	53.59	61.50
North Main Arch Span	Interior	31.50	31.97	35.00	40.43	49.67	56.00
	Exterior	31.00	31.50	34.56	40.09	49.53	56.00

Sub-elements are numbered so that web heights increase in the Table 2 - 3. All section dimensions and section properties for sub-elements modeling haunch girders are provided in Appendix B.

2.1.4 Plate Elements and Bridge Deck Mesh

The bridge deck consists of a 7.5-inch thick reinforced concrete slab attached by shear studs to both stringers and girders. The concrete bridge deck is partitioned into 1240 quadrilateral plate elements. The following subsections describe the plate element, mesh constraints, and bridge deck mesh, respectively.

2.1.4.1 Plate Element

The concrete bridge deck performs two primary structural functions: supporting out-of-plane loads, such as dead load and vehicle loads; and providing in-plane support to resist lateral loads, such as wind. Therefore, both out-of-plane (bending) and in-plane (membrane) stiffness characteristics are required so that the deck model will appropriately distribute loads into the bridge superstructure.

A plate element is a planar element with constant thickness, either quadrilateral or triangular in shape, with isotropic material properties (LARSA, 2004). Plate elements can function as either membrane planar elements, bending planar elements, or a combination of both, known as shells. Membrane behavior is based on a quadratic displacement field for in-plane displacements in the element. Higher-order displacement functions require more degrees of freedom within the element but yield a more flexible element compared to elements using 1st order or linear shape functions (LARSA, 2004). Since finite elements are by definition slightly stiffer than the actual member, the more flexible quadratic element will be more accurate. Bending behavior utilizes one translational and two rotational degrees of freedom at each joint.

Shell element results for the deck are reported with respect to a local coordinate system for each element. Therefore, the local coordinate systems for all shell elements must be oriented in the same direction so that the stresses and strains will have consistent direction and algebraic signs across the entire deck mesh. For example, due to the self-weight of the bridge deck, a column will induce tensile stresses in the top surface of the bridge deck directly above the column. Local axes of the shell elements above the column must be oriented similarly to ensure that the tensile stresses always have the same algebraic sign.

The local axes of plates are determined using the sequence of the joints entered – *I*, *J*, *K*, and *L* defined in counterclockwise order. Furthermore (LARSA, 2004):

- The origin of the plate's coordinate system is at the *I* joint (see Figure 2 - 5).
- The local *x*-axis is along the line from the *I* joint to the *J* joint.
- The local *z*-axis is perpendicular to the plane of the plate, in the direction of the right-hand rule.
- The local *y*-axis is normal to *x*- and *z*-axes.
- The thickness of the plate extends thickness/2 into the positive-*z* direction and thickness/2 into the negative-*z* direction.

Figure 2 - 5 shows the local coordinate axes and joint labels for a typical plate element.

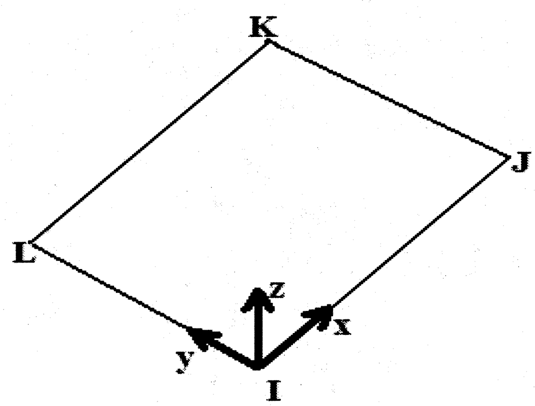


Figure 2 - 5: Local Plate Element Axes

All shell elements are 7.5 inches thick, which is the thickness of the concrete bridge deck. The mid surface of the shell elements is therefore 3.75 inches above the top flange of the supporting beam elements. The deck mesh is discussed in sub-sections Mesh Constraints and Deck Mesh, respectively.

2.1.4.2 Mesh Constraints

The deck mesh refers to the network of finite elements used to model bridge deck. The mesh of shell elements throughout the bridge deck is limited by a combination of physical and analytical constraints. Physical constraints are imposed by the need to position and connect the deck elements to the supporting beams. Analytic constraints result from the nature of the finite element calculations themselves. LARSA recommends that the shell element aspect ratio, defined as width-to-length should not be so extreme as to compromise the accuracy of the element (LARSA, 2004). To accommodate the geometry of the bridge deck and supporting elements, shell elements will inevitably have unequal sides. In order to preserve accuracy, shell elements in this model are limited by the following aspect ratios:

$$\frac{1}{2} \leq \frac{\text{width}}{\text{length}} \leq \frac{2}{1}$$

where *width* is the transverse dimension and *length* is the longitudinal dimension of the shell element, respectively. Note that for convenience, shell element dimensions are arbitrarily defined with respect to the orientation of the bridge deck. LARSA also recommends that interior angles of quadrilateral elements be as close as possible to ninety degrees (LARSA, 2004). In this model, shell element joints are arranged in a rectangular grid so that all interior angles of shell elements are exactly ninety degrees.

With very few exceptions, the shell element joints are located directly above the joint of a deck support element. In this context, a deck support refers to either stringers or girders because both support the bridge deck. Shell element joints which are aligned with deck support joints are constrained by a "slave-master" relationship to deform as the supporting stringer or plate girder bends thereby modeling the composite behavior indicated by the shear studs. Generally, a slave-master connection between the deck and deck support elements occurs at twelve-foot intervals. The definition of a slave-master constraint and its function are described in Section 2.1.5.

The widths of shell elements must span from one deck support to an adjacent deck support. Consequentially, shell element widths are dictated by the distances between deck supports. Figure 2 - 6 and Figure 2 - 7 illustrate the floor system and deck support layout for a typical approach span and main arch span, respectively.

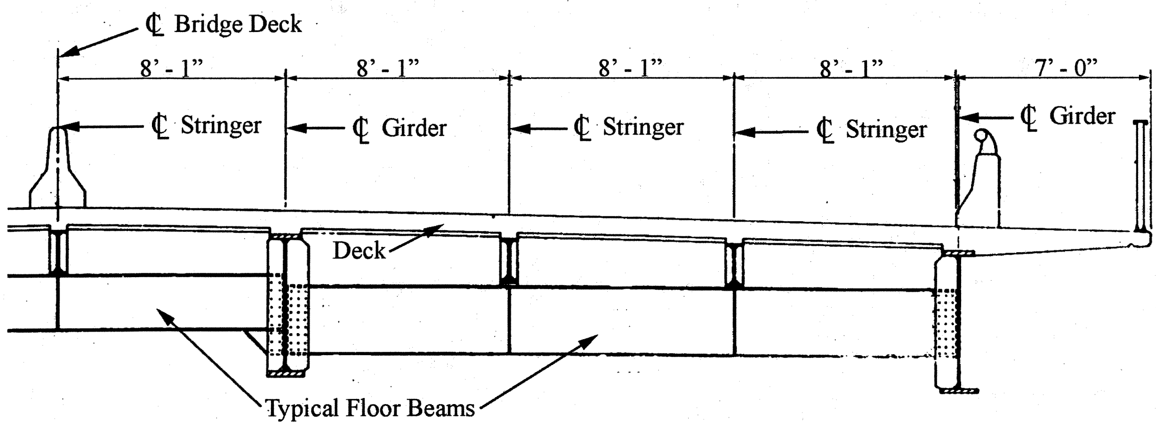


Figure 2 - 6: Typical Approach Span Floor System

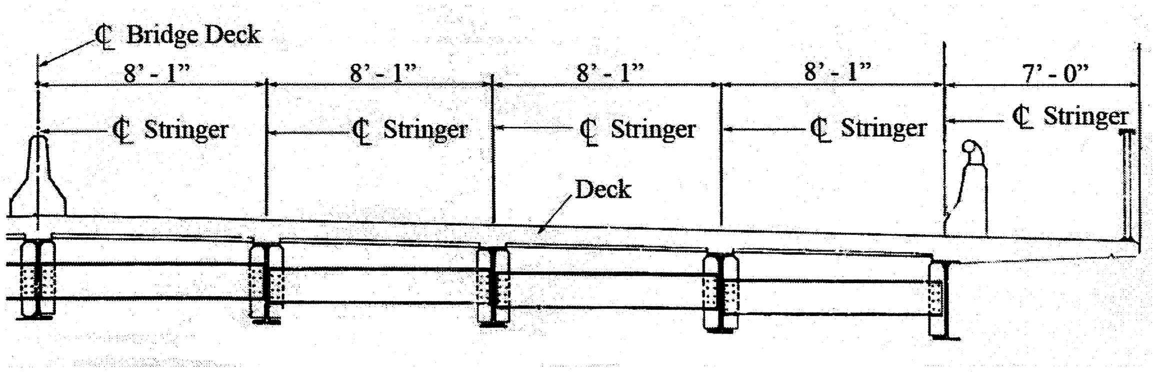


Figure 2 - 7: Typical Main Arch Span Floor System

For both Figure 2 - 6 and Figure 2 - 7, the bridge is symmetrical about the bridge deck centerline. Comparing both figures, it can be seen that the shell element width remains constant for the approach and main arch spans of the bridge. Therefore, interior shell widths between deck supports are eight feet, one inch and the exterior shell widths are seven feet.

2.1.4.3 Deck Mesh

Both shell lengths and corresponding beam lengths are selected to best suit the geometry of the bridge. For example, over the main arch span, plate and beam element lengths are selected to evenly divide the span between spandrel column supports. In regions where the deck is supported by haunched girders, analytic beam sub-element lengths dictate plate lengths because beam section properties model specific lengths of the haunched girder. In all cases, the aspect ratio of the shell element satisfies the aspect ratio limit given above. Table 2 - 4 shows the aspect ratios for shell elements modeling the bridge deck.

Table 2 - 4: Plate Element Aspect Ratios

	Aspect Ratio					
	South Approach		North Approach		Main Arch	
Deck Supported by	**Exterior**	**Interior**	**Exterior**	**Interior**	**Exterior**	**Interior**
Prismatic Beam	0.582	0.665	0.683	0.780	0.536	0.613
Non-Prismatic Beam	0.582	0.665	0.564	0.645	0.509	0.582

The minimum shell aspect ratio is recorded in shell elements located above haunched girders.

The deck mesh is divided into three separate segments: the south approach span, the north approach span, and the main arch span. On the bridge, each of these segments is divided by an expansion joint. Since the expansion joint physically divides the deck, stresses in one deck segment are not transferred to an adjacent deck segment. In the finite element model, expansion joints are modeled by excluding shell elements that would bridge between deck segments ensuring that deck stresses cannot be transferred between these deck segments.

2.1.5 Slave-Master Constraint

As previously mentioned in Section 2.1.4, the bridge deck is connected to stringers and plate girders by shear studs. Shear studs are assumed to ensure composite bending action between supporting deck members and the bridge deck. Consequently, the displacements of both the bridge deck and supporting deck members will be compatible.

In the finite element model, the bridge deck is connected to supporting deck members by a slave/master constraint. Slave/master constraints enforce equal displacements for specified degrees of freedom at any two joints (LARSA, 2004). The slave joint will move independently in DOF where DOF of a master joint are not specified. Since the deck and both stringers and girders act as one composite bending member, almost all shell element joints are slaved for all six DOF to corresponding master joints of supporting stringers and girders.

Eighteen shell joints are not slaved to bending elements. These occur at locations where the corresponding shell elements were divided so that shell aspect ratios would be within the specified limit. New joints were created as the shells were divided and there were no corresponding supporting bending element joints to which the shell joints could be slaved. These joints are located in the center of the south approach deck near the bent.

2.1.6 Rigid Link Elements

Rigid links connect beam elements where a connection cannot be made through a mutual joint between two beam elements. This is common when intersecting beam element centerlines do not align, e.g., the stepped element model of the haunched girders. Rigid links transfer reactions and displacements from one beam element to another across the offset joints between beam elements. There are approximately twelve hundred rigid link elements in the finite element model. The following subsections describe section properties and applications for the rigid link.

2.1.6.1 Rigid Link Element

Rigid link stiffness coefficients cannot be so small that rigid links deform, erroneously adding to beam displacements. Conversely, rigid link stiffness coefficients cannot be so great that beam element stiffness coefficients become so small in comparison that valuable analysis information is lost due to round-off error in the computer analysis. Therefore, the stiffness coefficients for rigid link elements are targeted to be approximately three to four orders of magnitude greater than the largest beam stiffness coefficient.

The axial stiffness coefficient is calculated by the following equation:

$$K = \frac{AE}{L^3}$$

where K, A, E, and L represent the axial stiffness coefficient, the element cross-sectional area, the modulus of elasticity, and element length, respectively (McGuire et al., 2000). Stiffness for the axial stiffness coefficient is in terms of axial force per unit displacement.

There are three stiffness formulas corresponding to bending stiffness about an axis of rotation:

$$K = \frac{12EI}{L^3}$$

$$K = \frac{6EI}{L^2}$$

$$K = \frac{4EI}{L}$$

where E, I, and L represent modulus of elasticity, moment of inertia, and element length, respectively (McGuire et al., 2000). The first bending stiffness coefficient defines the transverse bending force (shear force) per unit translation. The second bending stiffness coefficient describes the transverse bending force per unit rotation or the bending moment per unit translation. Finally, the third bending stiffness coefficient denotes the bending moment per unit rotation. The exterior haunch girder of the south approach is used to scale stiffness coefficients of the rigid link element because this girder has the greatest section properties i.e., cross-section area, strong axis moment of inertia, and weak axis moment of inertia, and therefore the greatest stiffness for any structural member requiring rigid links. Rigid link stiffness coefficients are scaled based on section properties rather than material properties because each stiffness coefficient (i.e., axial, strong axis bending, and weak axis bending stiffness coefficients) can be scaled independently. Cross section area is scaled to meet axial stiffness criteria. Similarly, strong and weak axis moments of inertia are scaled to meet corresponding bending stiffness criteria. When scaling is based on adjusting the material properties, only the modulus of elasticity can be adjusted to meet the stiffness criteria for three different structural responses. Inevitably only one stiffness coefficient can be accurately scaled to the appropriate stiffness, while the remaining stiffness coefficients cannot. The section properties and stiffness coefficients for both the rigid link element and the deepest haunch beam element, south approach gird 5, are listed in Table 2 - 5.

Table 2 - 5: Section Property & Stiffness Coefficients For Rigid Link & South Ap. Gird 5

Element Section	Section Property			Stiffness Coefficient		
	A_{cs} (in^2)	I_{zz} (in^4)	I_{yy} (in^4)	Axial (kip/in)	Strong Bending (kip-in)	Weak Bending (kip-in)
rigid link	82000	$2.49 \cdot 10^8$	$1.53 \cdot 10^6$	$2.96 \cdot 10^8$	$3.59 \cdot 10^{12}$	$2.20 \cdot 10^{10}$
s. approach girder 5	147	$4.46 \cdot 10^5$	$2.70 \cdot 10^3$	$2.96 \cdot 10^5$	$3.59 \cdot 10^9$	$2.20 \cdot 10^7$

Cross-section area, strong axis moment of inertia, and weak axis moment of inertia are constant for all rigid link elements; however, bending stiffness coefficients vary depending on the length of the element. Rigid link stiffness coefficients listed in Table 2 - 5 are for the rigid link connecting the bending elements of south approach girder five and south approach girder four. The stiffness analysis scaling all rigid link section properties is presented in Appendix C.

2.1.6.2 Rigid Link Applications

As mentioned previously, rigid links connect members that cannot be connected through a mutually shared joint. The end conditions of the rigid link element vary depending on the type of application the rigid link serves.

Figure 2 - 6 shows a typical floor system for an approach span. Stringers bear on floor beams that are connected to plate girders. The bottom flanges of the approach stringers are connected to the top flanges of the floor beams through a bolted connection. A rigid link connects the stringer mid-height to the corresponding floor beam mid-height. The rigid link that connects these two members has a pinned connection at the stringer joint because internal moments in the stringer are not assumed to be transferred through this connection into the floor beam; only forces are transferred. The fixed end of this rigid link – at the floor beam mid-height – preserves internal moments in the floor beam generated by eccentricity between the stringer and the floor beam. Figure 2 - 8 illustrates a typical approach floor system represented in the finite element model. Beam and shell elements are shrunk twenty percent to clarify model geometry.

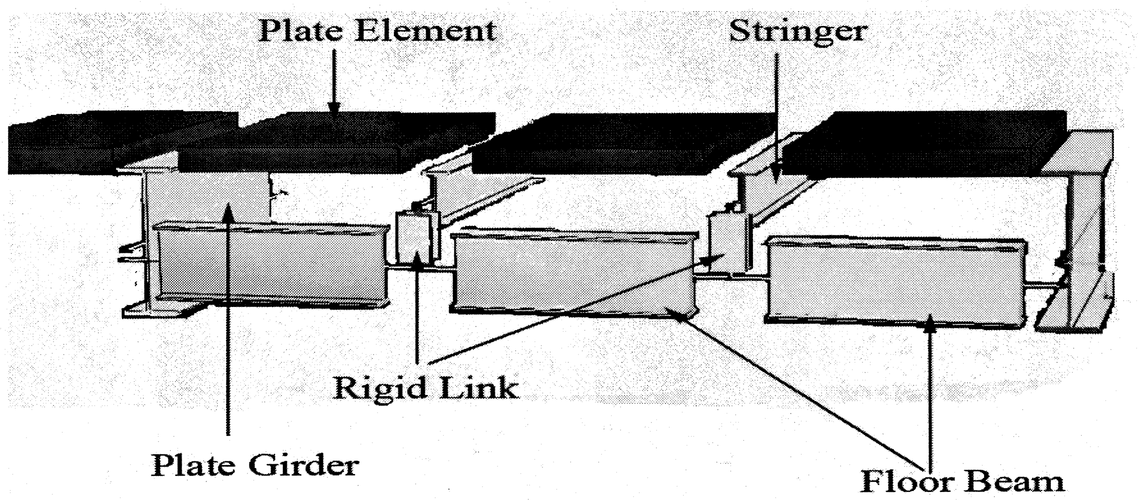


Figure 2 - 8: Finite Element Model of Typical Approach Floor System

Similar to the approach stringer and floor beam configuration, plate girders of the approach spans and main arch span bear on the box floor beam supporting the expansion joint between approach and main arch spans. Stringers are attached eccentrically to the floor beams, and floor beams are attached eccentrically to the plate girders. In the finite element model, rigid links connect the joints of stringers to the joints of the floor beam; additional rigid links connect the floor beams to the plate girders. These rigid links are fixed at the floor beam joint and pinned at the girder joint.

The most common application for a rigid link element connects intersecting structural elements when a mutually shared joint is not available. For example, diaphragm members are attached at the upper and lower ends of a plate girder's web. Additionally, floor beams are also connected to the web of the plate girder. The location of the joints varies along the web height of the plate girder. The rigid links connecting the diaphragm and floor beam elements to the plate girders are fixed at both ends. Figure 2 - 9 illustrates a connection between two different haunched girder elements and a floor beam. The upper, lighter shaded rigid link connects left hand plate girder to the floor beam. The lower, darker shaded rigid link connects the floor beam to the right hand plate girder. All beam elements are shrunk twenty percent to clarify model geometry.

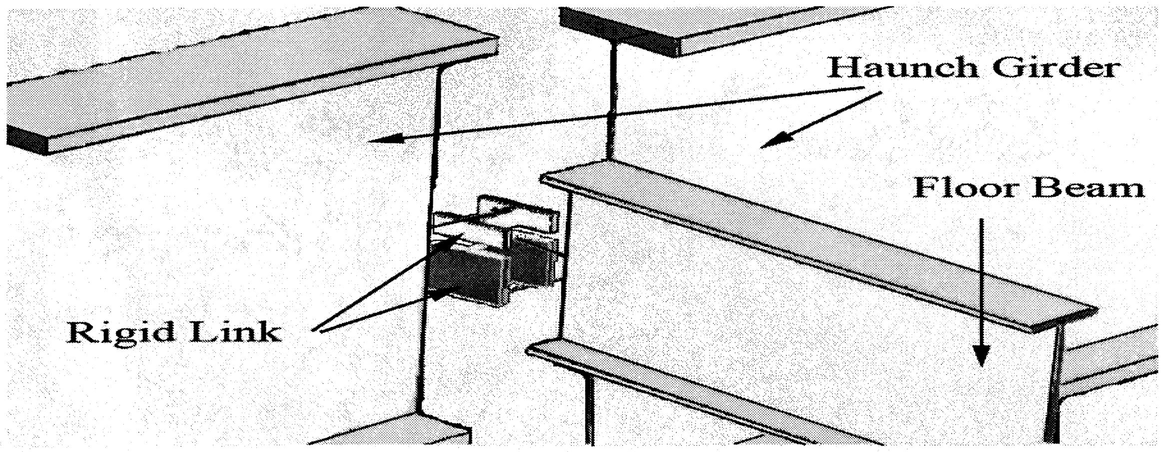


Figure 2 - 9: Web Connection for Haunch Girder/Floor Beam Intersection

2.2 Static Load Analysis

Three load cases are considered when calculating LRFR rating factors: dead load due to self-weight; dead load due to the wearing surface; and live loads resulting from moving vehicles. Self-weight and wearing surface loads are static loads. The two static load cases must be considered separately because different load magnification factors are applied to each. Assumptions for static load analyses are presented in Section 2.2.1, and both self-weight and wearing surface load cases are presented in Sections 2.2.2 and 2.2.3, respectively. Permanent load effects calculated by the finite element model are compared to the permanent load effects presented in the bridge plans in Section 2.2.4.

2.2.1 Static Load Analysis Assumptions

Static loads are analyzed using a linear static analysis. A linear static analysis is a first-order analysis that excludes geometric and material nonlinearity. The following basic assumptions are consistent with a linear static analysis (LARSA, 2004):

- Equations of motion are formulated on the geometry of the unloaded structure.
- First-order, infinitesimal, and linear strain approximations can be used.
- Material behavior of the elements is linear.

2.2.2 Self-Weight

The dead load for structural elements as well as concrete parapets and center median consists of the elements' self-weight. The weight of beam elements is calculated using the cross-sectional area, element length, and the unit weight assigned to the element. For beam elements, self-weight is applied as a uniform load along the length of the element producing the appropriate bending and shear effects in the nodal load vector. Plate element weight is calculated using the element volume and weight density assigned to the plate. The self-weight for plate elements is applied uniformly over the area of the plate. The dead loads for concrete parapets are applied to exterior girder bending elements as uniformly distributed loads based on the cross-section geometry and weight density of the parapet. The concrete median dead loads are applied to center stringer elements in a similar fashion. The dead load effects for all structural members are calculated and stored in an Excel spreadsheet.

2.2.3 Wearing Surface

The load effects for the wearing surface are calculated and accounted for separately from the self-weight of the structure. Per standard engineering practice, the bridge plans specify twenty-two pounds per square foot as an allowance for a future wearing surface, which is applied as a uniform load to the plate elements in the finite element model. The load effects of the wearing surface are calculated and stored in an Excel spreadsheet.

2.2.4 Comparing Permanent Load Effects

Permanent load effects are the sum of self-weight and the wearing surface loads on the structure. The I.B. Perrine Bridge plans present the total dead load effects (i.e., permanent load effects), used for design of the principal structural members of the arch truss and spandrel columns. Table 2 - 6 summarizes the relative differences in percent between the total permanent load effects calculated by the finite element model and the design load resultants presented in the bridge plans. Values presented in Table 2 - 6 show the most extreme differences between the two permanent dead load calculations. Positive differences indicate that the permanent load effects of the finite element model exceeded the original design dead load calculations. Conversely, a negative difference means that the finite element model underestimated the magnitude of permanent load effects compared to the original design dead load calculations. For the arch posts and diagonals near the center arch, the negative difference suggests that the finite element model dead load effects are opposite from the design dead load effects. For example, all arch diagonals are designed to resist tensile forces; however, near the center arch, the finite element model calculates compression forces. Figures indicating the nodal numbering scheme, arch members, and element locations near the center arch are provided in Appendix D.

Table 2 - 6: Relative Dead Load Difference Between the Finite Element Model and Original Design Loads for Principal Arch Members

Structural Member	Excluding Center Arch Min/Max (%)		Near Center Arch Min/Max (%)	
Top Chords	1.40	5.90	0.90	5.30
Bottom Chords	2.60	5.20	-9.00	1.80
Posts	-10.80	-8.70	-113.90	52.40
Diagonals	-1.00	14.90	-121.90	-14.50
Columns	4.50	11.20	7.20	7.60

Table 2 - 6 indicates that permanent load effects for principal arch members and spandrel columns agree within ten percent for most elements. The largest differences occur in the post and diagonal members located near the center arch. Two principal reasons have been suggested for these differences. At the time the Perrine Bridge was designed and constructed, two-dimensional models were commonly used for analyzing structures. The finite element model used for load rating is a three-dimensional model. Differences in load paths between the two-dimensional and three-dimensional models may be most pronounced near the center of the arch. Furthermore, the original designs may have only included lateral braces to provide structural stability within the arch truss and to resist lateral loads such as wind. In the finite element model, lateral brace members in the arch truss may be resisting more load than originally expected due the significant stiffness the brace members. Anticipated model calibration studies will either validate the finite element results for these members or suggest needed changes.

2.3 Moving Load Analysis

The LRFR procedure uses the maximum live load response (i.e. maximum positive moment, negative moment, shear, and axial force) to calculate LRFR factors (AASHTO, 2003). The finite element model envelopes the maximum live load response for all structural members and all load positions using the results of a moving load analysis. The moving load analysis is described in Section 2.3.1. The mechanics of the load path and description of the lane element are described in Sections 2.3.2 and 2.3.3, respectively. Finally, the vehicle configuration used in a moving load analysis is defined in Section 2.3.4.

2.3.1 Moving Load Analysis Assumptions

A moving load analysis generates the load cases that simulate the movement of a vehicle(s) traveling along a user-defined path (LARSA, 2004). Automatically generated load cases correspond to different vehicle positions at user-specified increments along the load path. Moving load cases are generated starting with the first axle located at the first joint of the first element defining the lane and proceeding until the final axle of the vehicle configuration reaches the last joint of the last element defining the lane. A linear static analysis is performed for every load case generated in a moving load analysis. Therefore, the analysis assumptions for linear static analyses (Section 2.2.1) apply to moving load analyses. Since both static and moving load analyses assume linear elastic behavior, the results from both analyses are superimposed to estimate the total load effects for structural members.

2.3.2 Defining Moving Load Path

The I.B. Perrine Bridge accommodates a four-lane highway with two striped lanes traveling in either direction. However, since a vehicle may travel along a variety of paths, the finite element model defines several lanes in addition to the striped lanes. AASHTO design requirements prescribe design lanes that are offset from the striped lanes. For instance, in order to produce maximum load effects in the exterior girders, the wheel loads in the outside design lane are placed six inches from the parapet curb, whereas for the striped lane they are approximately two feet from the curb. In another scenario, a trip permit may require an overweight vehicle to straddle the striped lanes to minimize live load demands on specific members. Therefore, the finite element model allows the user to choose from ten travel paths for a moving load analysis. For each travel direction there are two design lanes – each at the outside edge of the striped lanes; two load paths, centered on each of the striped lanes; and a load path straddling the two striped lanes.

A lane is defined in the computer model by selecting a beam element and declaring it a lane member. Multiple beam elements are typically combined to declare one lane which spans an entire structure. Structural beam elements are rarely coincidentally aligned with travel lanes. Therefore virtual beam elements (lane elements) are added to conveniently define moving load paths. These longitudinal elements are located in the mid-plane of the plate elements defining the bridge deck. The virtual longitudinal elements are connected to virtual transverse beams which are connected to actual plate element joints. This is illustrated in Figure 2 - 10, with the plate elements reduced twenty percent to avoid obscuring the lane elements.

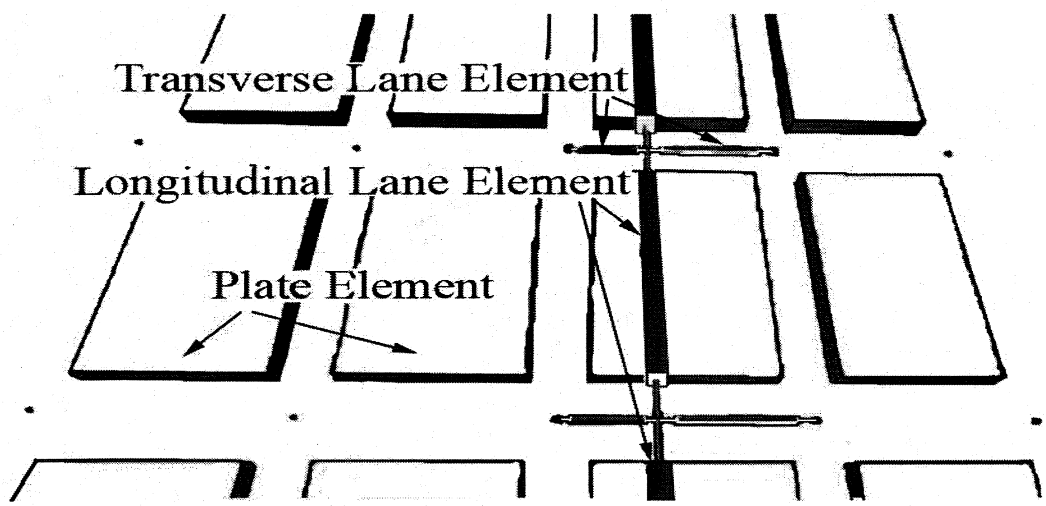


Figure 2 - 10: Lane Connectivity to Deck Super Structure

Vehicle loads are initially applied to the virtual longitudinal elements defining an individual lane. These live loads are transferred through the virtual transverse members into the (real) bridge deck at the shell element joints. The shell (deck) element joints are in turn supported by the longitudinal stringers or girders and the balance of the bridge superstructure. All virtual longitudinal and transverse lane elements have reduced stiffness characteristics so they do not significantly influence the stiffness characteristics of the deck. Lane elements are discussed in Section 2.3.3.

2.3.3 Lane Element

As mentioned in Section 2.3.2, virtual lane elements are used to define the load paths for moving load analyses. Transverse lane elements intersect one another at both plate element joints and longitudinal lane element joints, essentially creating a network of virtual lane elements between load paths. For indeterminate structures, such as the network of lane elements, load effects are distributed among elements in proportion to the relative stiffness of the elements. Since lane elements all have similar stiffness coefficients, they can effectively transfer load effects among one another to nearby plate element joints. However, lane element stiffness coefficients are substantially smaller than the plate element properties. As are result, significant load effects are transferred from loaded lane elements to plate elements and not into adjacent lane elements. This appropriately models the (real) bridge, where tire loads create concentrated local stresses in the deck. Section properties for lane elements are summarized in Table 2 - 7.

Table 2 - 7: Lane Element Section Properties

	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsion Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
lane element	0.10	0.10	0.10	0.01	5.00	5.00

2.3.4 Vehicle Configuration

A moving load pattern (i.e. vehicle configuration) defines the location of wheel contact points with respect to the load path and the magnitude of the forces at the contact points. The user can select a vehicle configuration from a database of pre-defined American standard vehicle configurations or create a load configuration representing the exact axle geometry and wheel contact loads. Since the finite element model and load rating software are designed to rate bridge members for over-permit vehicle loads, the latter case may be more common. Figure 2 - 11 illustrates the moving load pattern used to model a dump truck in field tests. The downward arrows indicate wheel contact points and their corresponding forces (in kips).

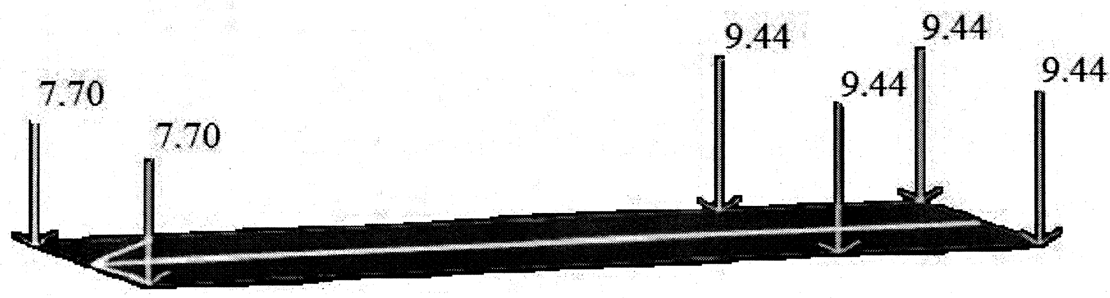


Figure 2 - 11: Dump Truck Load Pattern

The instructions for entering a custom vehicle configuration in the Larsa model are presented in Appendix E.

3 LOAD AND RESISTANCE FACTOR RATING

The following subsections discuss: (1) AASHTO Load and Resistance Factor Design (LRFD) capacities calculated for bridge members and (2) AASHTO Load and Resistance Factor Rating (LRFR) procedures.

3.1 Load and Resistance Factor Design (LRFD) Capacity

The LRFR procedure determines member rating factors from the LRFD strength capacity. In this study, all capacity terms are calculated in accordance with the AASHTO LRFD Bridge Design Specifications 3rd ed., 2005 Interim Revision, referred to hereafter as the LRFD code. Specific capacity terms are calculated for each bridge member depending on the type of resistance a member provides. Since not all bridge members provide the same structural function (i.e., bending resistance or axial force resistance), not all bridge members have the same types of capacity terms. For example, end-loaded truss members may have either axial compression capacity, tensile capacity, or both, depending on the type of truss member. However, end-loaded truss members do not have bending capacity terms, i.e., moment and shear capacities, because these members are primarily loaded in axial compression or tension. Subsections 3.1.1, 3.1.2, and 3.1.3 present the conditions used to calculate the LRFD design capacities for axial force members, prismatic bending members, and haunch bending members, respectively. All prismatic member capacities are presented in Appendix F and all haunched member capacities are presented in Appendix G.

3.1.1 End-Loaded Axial Force Members

End-loaded axial force members include columns, arch truss members, and various diaphragm members. Excluding self-weight, loads are only applied through the end-connections of these members. Axial force members may resist compression only, tension only, or both tension and compression forces. Where live loads may cause stress reversals in bridge members, both axial compression and tensile capacities are calculated. Both axial capacities are calculated for all diaphragm members in the bridge deck. Both axial capacities are also calculated for both upper and lower chevron braces and cross-bracing members in the arch truss.

Principal structural components, such as columns and arch truss chords, posts, and diagonals, predominately resist static dead loads. Therefore, stress reversals are highly unlikely to occur in these types of members. Where static dead loads govern the axial reaction of a member, only the applicable capacity corresponding to the static dead load reaction is calculated and used in LRFR load rating. Columns, posts, and both upper and lower chords of the arch truss have only axial compression capacities. Conversely, only tensile capacities are calculated for the diagonals of the arch truss. The following subsections – Tensile Capacity and Axial Compression Capacity – describe how these two types of capacities are calculated.

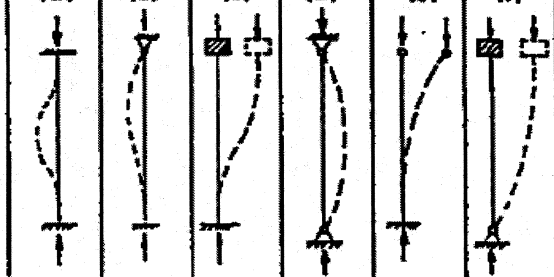
3.1.1.1 Tensile Capacity

Tensile capacities are estimated in accordance with Article 6.8 of the LRFD code. A sample spreadsheet calculating tensile capacities for bridge members is located in Appendix H.

3.1.1.2 Axial Compression Capacity

Axial compression capacities are estimated in accordance with Article 6.9 of the LRFD code. Buckling is a common failure mode controlling compression member design. Therefore compression strength is a function of a member's effective length. The effective length is the actual length times K, the effective length factor. The effective length factor accounts for the effects of the end restraints on the deformed shape of a compression member. Table 3 - 1 illustrates the deformed shapes corresponding to the various effective length factors.

Table 3 - 1: Effective Length Factor, K (AASHTO, LRFD Table C4.6.2.5-1)

EFFECTIVE LENGTH FACTORS, K						
	(a)	(b)	(c)	(d)	(e)	(f)
BUCKLED SHAPE OF COLUMN IS SHOWN BY DASHED LINE						
THEORETICAL K VALUE	0.5	0.7	1.0	1.0	2.0	2.0
DESIGN VALUE OF K WHEN IDEAL CONDITIONS ARE APPROXIMATED(1)	0.65	0.80	1.2	1.0	2.1	2.0
END CONDITION CODE	ROTATION FIXED TRANSLATION FIXED ROTATION FREE TRANSLATION FIXED ROTATION FIXED TRANSLATION FREE ROTATION FREE TRANSLATION FREE					

Spandrel and bent columns are assumed to buckle in shape (b), from Table 3 - 1. The column is considered fixed at the bent cap and pinned at the bottom where the column intersects the arch truss. The effective length factor is the same for both strong and weak axis buckling. Compression members in the arch truss are assumed to be pinned at both ends. The LRFD code permits pinned truss members to use an effective length factor of 0.875, which is slightly reduced from the effective length factor suggested for buckled shape (d) (AASHTO, 2005). Finally, all deck bracing members (vertical diaphragms and horizontal wind bracing) conservatively use an effective length factor equal to 1.0, corresponding to buckled shape (d) in Table 3 - 1. A sample spread-sheet calculating compression member design capacities is presented in Appendix I.

3.1.2 Prismatic Bending Members

Plate girders, stringers, approach floor beams, main arch span floor beams, and arch struts are the only bridge members analyzed as bending members. Shear and both positive and negative moment capacities are calculated for all sections of all bending members. Chevron bracing is connected to the web of arch struts causing weak axis bending; therefore, both strong and weak axis bending moments are calculated for arch struts. The following subsections describe the calculation of prismatic bending capacity terms.

3.1.2.1 Composite Positive Bending Moment Capacity

Stringers and plate girders are considered fully composite with the bridge deck. Shear studs are detailed in these members ensuring that tensile or compressive stresses in the top flange of the stringers and girders are transmitted to the deck. For composite bending members in positive flexure, the contribution of the longitudinal slab reinforcement to the positive moment capacity is conservatively neglected (AASHTO, 2005). The contribution of the concrete parapet to exterior girder strength is also conservatively neglected. Positive moment capacities for composite sections in positive flexure are calculated in accordance with Article 6.10.7 of the LRFD code. Spreadsheets calculating composite moment capacity follow flowchart C6.4.5 of the LRFD code. A sample spreadsheet calculating composite moment capacity is attached in Appendix J.

3.1.2.2 Negative Composite & Non-Composite Bending Moment Capacity

Plate girders and stringers that are considered composite in positive flexure are considered composite in negative flexure. Approach floor beams, main arch span floor beams, and arch struts are non-composite bending members because they are not attached to the bridge deck. Moment capacities for composite bending members in negative flexure and non-composite bending members are calculated in accordance with Article 6.10.8 of the LRFD code. Longitudinal slab reinforcement located within the effective slab width is used to calculate negative bending capacity. The contribution of longitudinal slab reinforcement to the negative composite moment capacity is based on the slab geometry presented in Figure 3- 1.

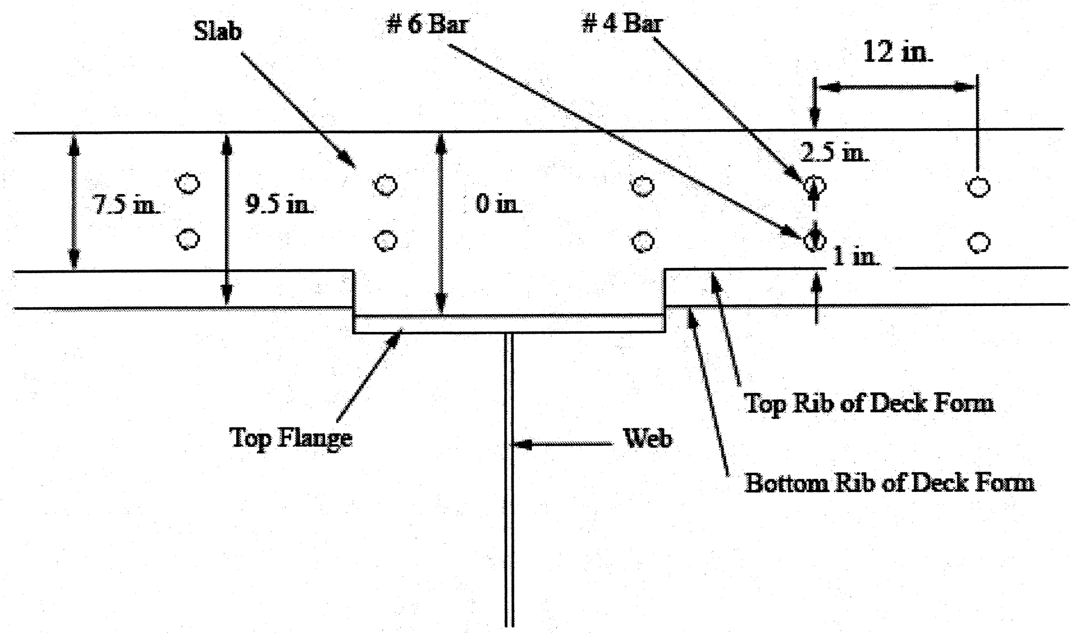


Figure 3- 1: Vertical Dimensions for Longitudinal Deck Reinforcement

Spreadsheets calculating negative composite and non-composite moment capacity follow flowchart C6.4.6 of the LRFD code. A sample spreadsheet calculating negative composite and non-composite moment capacity is attached in Appendix K.

3.1.2.3 Shear Capacity

The shear capacities of all prismatic bending members are calculated according to Article 6.10.9 of the LRFD code. Plate girders and main arch span floor beams have both interior panel and end panel shear capacities. Both approach and arch span stringers (rolled I-sections) are unstiffened. Approach floor beams are also considered unstiffened. Spreadsheets calculating shear capacity follow the Flowchart for Shear Design of I-Sections, Figure C6.10.9.1-1 of the AASHTO LRFD code. A sample spreadsheet calculating shear capacity for prismatic I-sections is presented in Appendix L.

3.1.3 Haunched Bending Members

As discussed in Section 2.1.3, haunched bending members are modeled as multiple prismatic bending elements (i.e., "stepped" elements). Positive moment, negative moment, and shear capacities are calculated for each analytic element (i.e., for each "step"). Bending section capacities correspond to the haunched section capacity at mid-span of the analytic element.

Similar to prismatic bending members, composite positive and negative moment capacities are calculated according to Article 6.10.7 and 6.10.8 of the LRFD code. Haunched members are considered fully composite with the deck slab. The contribution of longitudinal slab reinforcement to composite negative moment capacity is based on the same slab geometry illustrated in Figure 3- 1. The resultant yield force of the inclined bottom flange in haunched sections has both vertical and horizontal components, in contrast with prismatic sections where flange forces are entirely horizontal. Only the horizontal component of the resultant bottom flange force is considered when calculating section moment resistance. See Appendix J for samples of composite positive bending moment capacity calculations. See Appendix K for samples of composite negative bending moment capacity calculations.

3.1.3.1 Haunched Member Shear Capacity

The flexural stresses in the bottom flange of a haunched section are parallel to the inclined bottom flange. Therefore the resultant force in the bottom flange has both a horizontal component that resists moment reactions and a vertical force component that affects the shear capacity of the section. Haunched girders are typically located in regions where permanent loads induce negative bending moment or flexure resulting in compression on the bottom flange. For parabolic haunched girders, the vertical component of the compression force in the inclined bottom flange reduces the shear stress in the girder web (Blodgett, 1966). Figure 3- 2 illustrates the section forces for a continuous parabolic haunched girder in negative flexure.

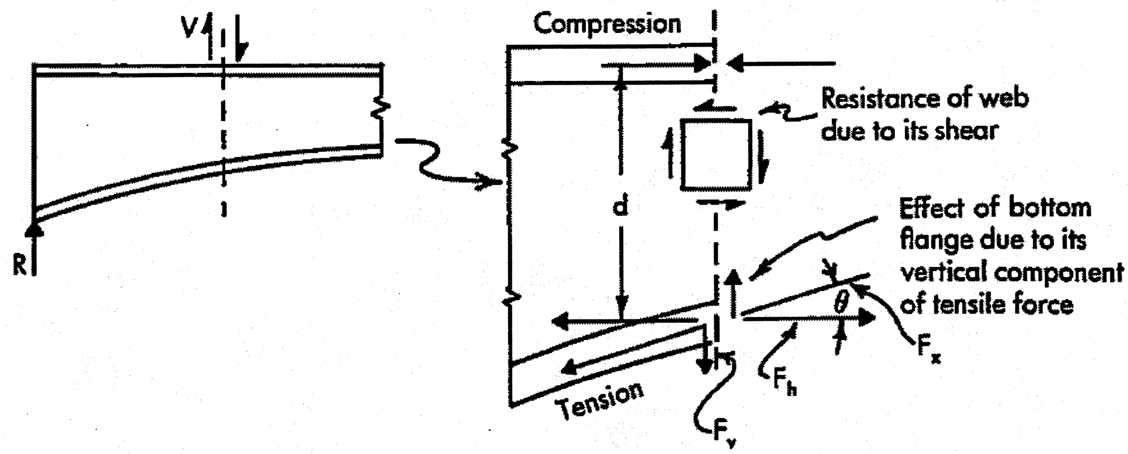


Figure 3- 2: Haunch Section Forces for Negative Bending Moment (from Blodgett, 1966)

The effect of the applied moment on a section's shear capacity depends on the geometry of the haunch. For example, as the inclination of the bottom flange increases, the vertical component of the compressive bottom flange force increases as well, effectively increasing the shear capacity of the section. Conversely, as the inclination of the bottom flange approaches zero (i.e., horizontal bottom flange) the section behaves as a prismatic I-section in shear, where significant vertical flange forces are not transferred into the web of the girder.

The plastic shear capacity for haunched bending sections assumes that the critical stress calculated according to the Huber-von Mises formula equals the yield stress of steel (A588-Gr50). The plastic shear capacity for haunched sections is calculated from the section geometry, yield strength of steel, and the applied moment at the section (Blodgett, 1966). Haunch geometry (i.e., degree of haunch and radius of curvature) were measured from scaled drawings. The initial inclination of the haunch (θ) could be accurately measured to the nearest half degree; however, measurements for the radius of curvature are more approximate. Figure 3- 3 illustrates that as the radius of curvature (r) increases, the plastic shear capacity decreases. Therefore, radii of curvature are conservatively exaggerated from their initial measurement so that the plastic shear capacity is not erroneously overestimated.

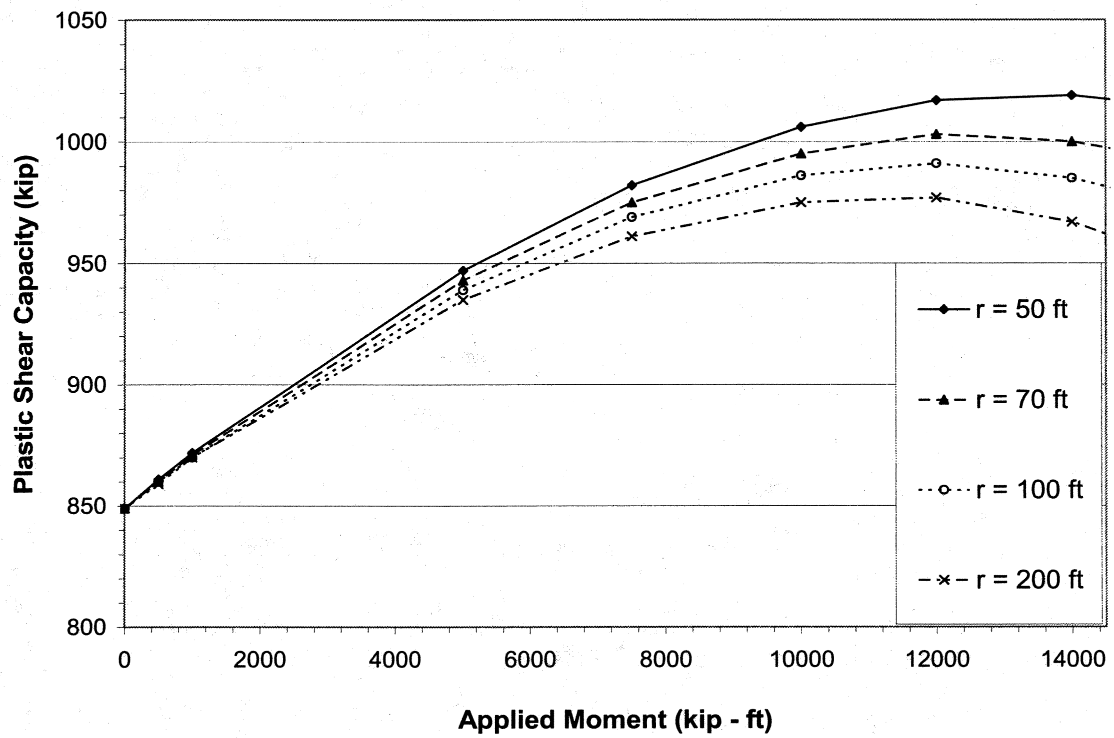


Figure 3- 3: Applied Moment vs. Plastic Shear Capacity

The negative moments in Figure 3- 3 are reported as absolute values to agree with structural mechanics presented by Blodgett (1966). The Mathcad worksheet calculating plastic shear capacity is presented in Appendix M.

Figure 3- 3 indicates that the plastic shear capacity depends on the applied moment. A shear force envelope combining both static and moving load analyses also provides applied moments occurring coincidentally with the maximum shear loading for a single moving load analysis. A range of applied moments was created by enveloping shear forces from numerous moving load analyses performed for different travel paths and vehicle configurations. The maximum and minimum applied moments from these analyses define the range of possible plastic shear capacity values. This range of plastic shear capacity values for this range of applied moments is illustrated in Figure 3- 4 for the same section analyzed in Figure 3- 3. Again, negative moments are reported as absolute values.

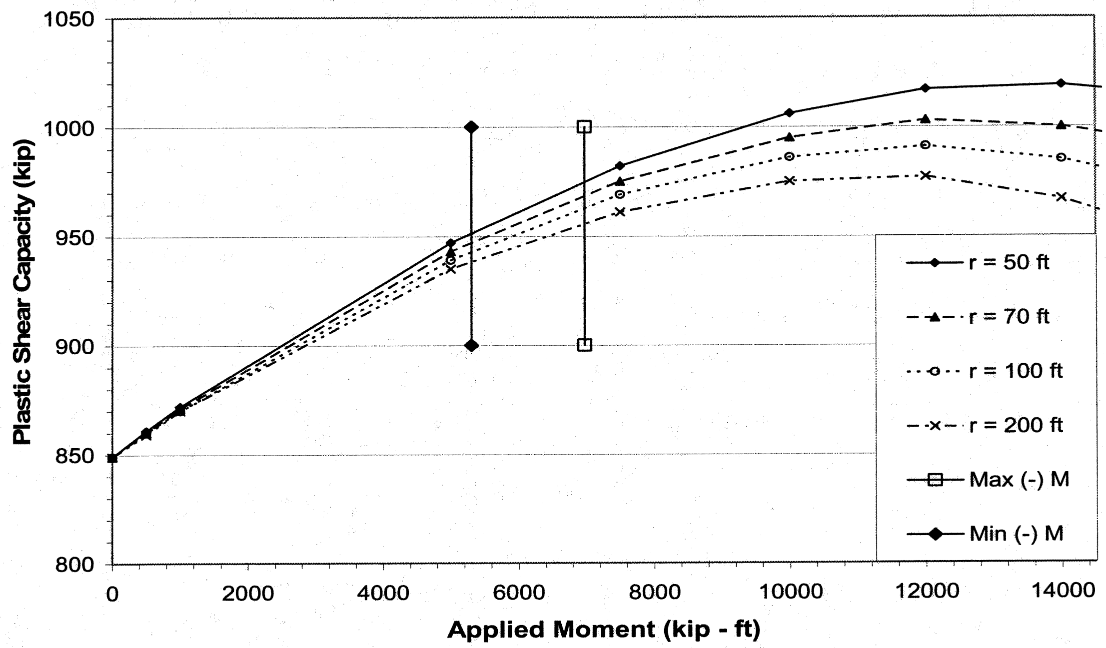


Figure 3- 4: Plastic Shear Capacity Range

The smallest plastic shear capacity occurring within the range of applied moments and radii of curvature is conservatively used as the section's plastic shear capacity. For example, the section shown in Figure 3- 4 has a plastic shear capacity of 938 kips for a radius of curvature of 200 feet. Once the plastic shear capacity is estimated for a section, the shear design capacity is calculated according to Article 6.10.9 of the LRFD code. Haunched sections are considered stiffened, interior panels for shear design capacity calculations. See Appendix N for plastic shear capacity ranges of elements modeling haunched members.

3.2 Load and Resistance Factor Rating (LRFR)

This section describes the load and resistance parameters used in the LRFR calculation. The LRFR analysis must be modified to reflect current system conditions and vehicle loads as both system characteristics and over-permit vehicle types vary. For example, the default condition factor assumes that regular bridge inspections reveal that all bridge components are in good condition. The default LRFR analyses assume that vehicles are single-trip permits traveling in mixed traffic and that vehicles travel at posted speed limits (speeds are not reduced). All structural members are assumed to be in good condition. Section 3.2.1 describes the LRFR equation and Section 3.2.2 defines the individual parameters used in the rating factor calculation.

3.2.1 General Rating Factor Equation

The analysis spreadsheet load rates bridge members for permit loads at the Strength II limit state corresponding to, "Owner-specified … evaluation permit vehicles." Service and Fatigue limit states are not considered. The general rating factor used to determine the load rating of each component subjected to a single force effect (i.e., axial force, flexure, or shear) can be described by the following equation (AASHTO, 2003):

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot R_n - \gamma_{DC} \cdot DC - \gamma_{DW} \cdot DW \pm \gamma_P \cdot P}{\gamma_L \cdot (LL + IM)}$$

Where:

RF = Rating factor

φ_c = Condition factor

φ_s = System factor

φ = LRFD resistance factor

γ_{DC} = LRFD load factor for structural components and attachments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

γ_L = Evaluation live-load factor

γ_P = LRFD load factor for permanent loads other than dead loads = 1.0

DC = Dead-load effect due to structural components and attachments

DW = Dead-load effect due to wearing surface and utilities

IM = Dynamic load allowance

LL = Live-load effect

P = Permanent loads other than dead loads

R_n = Nominal member resistance (as-inspected)

No other permanent loads other than structural components, attachments, wearing surface, and utilities are considered (i.e., P equals zero). Therefore, the general rating factor equation simplifies to:

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot R_n - \gamma_{DC} \cdot DC - \gamma_{DW} \cdot DW}{\gamma_L \cdot (LL + IM)}$$

Both the dead load effects of the components and attachments (*DC*) and wearing surface and utilities (*DW*) are calculated prior to a moving load analysis. The results from both dead load analyses are included in data worksheets within the load rating spreadsheet. Once a moving load analysis is performed, live load effects (*LL*) are enveloped for each bridge member in LARSA and imported into the load rating spreadsheet. The organization of the load rating spreadsheet and Visual Basic programming that imports reaction data; envelopes reaction data over bridge spans; and sorts rating factors is discussed in Section 4.

The rating factor represents the ratio of the member's live load capacity divided by the maximum live load demand from the rating vehicle. Rating factors greater than one are satisfactory since the live load capacity is greater than the demand. Rating factors between zero and one indicate that the live load capacity is less than required for the rating vehicle. There are some negative rating factors for the uncalibrated bridge model. Negative rating factors indicate that the self-weight and wearing surface load effects exceed a member's capacity. Members with negative rating factors are misleading because the bridge can obviously support self-weight and wearing surface loads, in addition to live load demands. The uncalibrated bridge model apparently overestimates the load effects for members with negative rating factors. Calibrating the model response to match the actual bridge response requires further investigation of the initial structural modeling assumptions (see Section 1.3.2).

One possible reason that secondary brace members have significantly larger dead load effects calculated by the finite element model than expected is that the finite element model calculates the self-weight of the bridge as it is built (i.e., staged construction is not considered). If diaphragms and cross-bracing were constructed after the principal structural members, the dead load effects of principal members would not be transferred to the secondary braces, resulting in lower load demands. The finite element model calculates structural response of the members based on the relative stiffness of all the finite elements. Some secondary brace members may have relative stiffness values that result in greater calculated load effects than were originally designed. In fact, the stiffness of the secondary braced members may have been neglected in the original design. This is a subject of further research.

3.2.2 Input Parameters for Rating Factor Equation

Input parameters characterize the bridge and its condition when the loads are applied, e.g., dead load effects, live load effects, and nominal member resistances. The following subsections describe the factors considered as input parameters. Unless otherwise stated, all section, table, and page numbers referenced in the subsequent subsections are from the AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (2003).

3.2.2.1 Permanent Load Factors: γ_{DC} & γ_{DW}

The dead load factor (γ_{DC}) for the self-weight of structural components and attachments is 1.25 for the Strength II limit state (AASHTO, 2003). Load factors are taken from the LRFR Manual Table 6-1, pg. 6-14. The self-weight load factor will not be modified for routine analyses. The wearing surface load factor (γ_{DW}) is 1.50 for the Strength II limit state. However, the load factor for DW at the Strength II limit state may be adjusted to 1.25 where the thickness of the wearing surface has been field measured (AASHTO, 2003).

3.2.2.2 Condition Factor: φ_c

According to the LRFR Manual, the condition factor is considered at the discretion of the governing agency (AASHTO, 2003). It has been included for the LRFR calculations. The LRFR Manual states, "The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles" (AASHTO, 2003). Damage due to vehicle accidents, collisions, or any other accident is not considered by this factor.

Table 3 - 2 (Table 6-2 from the LRFR Manual) defines the condition factor, φ_c, (AASHTO, 2003). All members are assumed to be in satisfactory condition; however, individual condition factors can be changed to reflect the structural condition determined by bridge inspection.

Table 3 - 2: Condition Factor: φ_c

Structural Condition of Member	φ_c
Good or Satisfactory:	1.00
Fair:	0.95
Poor:	0.85

The LRFR spreadsheet includes condition factors for nine types of structural components: (1) stringers and girders, (2) floor beams, (3) arch chords, (4) arch posts, (5) arch diagonals, (6) chevron braces, (7) arch struts, (8) columns, and (9) bracing. These components are labeled: (1) Strngr, (2) Fl. Beam, (3) A. Chord, (4) A. Post, (5) A. Diag. (6) Chevron, (7) Arch Strut, (8) Column, (9) Bracing, respectively. For these condition factors, the structural types "stringers and girders" include main arch stringers and north and south approach plate girders and stringers. Floor beams include the plate floor beams supported by columns in the main arch span and the floor beams supporting stringers that are attached to plate girders in both approach spans. Arch chords, diagonals, chevron braces, and struts consider both upper and lower structural components for the arch truss. Columns include both spandrel columns supporting the main arch span and bent columns supporting both approaches. Finally, bracing considers any diaphragm, lateral, or transverse brace anywhere in the entire bridge.

3.2.2.3 System Factor: φ_s

The system factor (φ_s) for bridges is discussed in Section 6.4.2.4, page 6-16 of the LRFR Manual. This factor adds reserve capacity such that the overall system reliability is increased from approximately an operating level (for redundant systems) to a more realistic target for non-redundant systems corresponding to Inventory levels (AASHTO, 2003). Since the Perrine Bridge is a nine-stringer bridge over the arch main span (> four parallel girders or stringers), the stringers over the main arch span are assumed to be a redundant system with a system factor of 1.0 (see Table 6-3 of the LRFR Manual, AASHTO, 2003). Likewise, in the approach spans, the stringer subsystem between the floor beams is assumed to provide adequate redundancy such that the system factor equals 1.0.

3.2.2.4 Resistance Factor: φ

Resistance factors are found in AASHTO LRFD Bridge Design Specifications 2005 Interim Revision, Section 6.5.4.2. For shear and flexural resistance, φ equals 1.0. The compression resistance factor (φ equal to 0.90) is applied in the capacity spreadsheet and not in the load rating spreadsheet.

3.2.2.5 Live Load Factor: γ_L

Live load factors for permit load rating are presented in the LRFR Manual Section 6.4.5.4, page 6-26. For the I.B. Perrine Bridge, the average daily truck traffic (ADTT) for one direction is assumed to be greater than five thousand vehicles per day. Live load factors are presented in the LRFR Manual, Table 6-6. The portion of Table 6-6 pertaining to ADTT greater than five thousand vehicles per day is summarized in Table 3 - 3 (AASHTO, 2003).

Table 3 - 3: Permit Live Load Factors for ADTT > 5000

Permit Type	**Frequency**	**Load Condition**	**DF[a]**	**ADTT 1 Direction**	**Load Factor by Permit Weight[b]**	
					< 100 kips	**≥ 150 kips**
Routine or Annual	Unlimited Crossings	Mix w/ Traffic	Two or more lanes	> 5000	1.80	1.30
					All Weights	
Special or Limited Crossing	Single Trip	Escorted	One lane	N/A	1.15	
	Single Trip	Mix w/ Traffic	One lane	> 5000	1.50	
	Multiple <100	Mix w/ Traffic	One lane	> 5000	1.85	

Notes:

[a] DF = LRFD-live load distribution factor. When a one-lane live load distribution factor is used, the built-in multiple presence factor should be divided out.

[b] For routine (inventory) permits between 100 kips and 150 kips, the load factor is interpolated by weight and ADTT value using only axle weights on the bridge.

Operators should select the applicable permit live load distribution factor. The default permit live load factor is 1.50, corresponding to a single trip, traveling in mixed traffic.

3.2.2.6 Dynamic Load Allowance: *IM*

The dynamic load allowance increases the static loads of trucks for strength and service limit states to account for the dynamic effects resulting from moving vehicles. Under typical traveling conditions all bridge members, except main arch span stringers, receive a thirty-three percent dynamic load allowance (i.e., IM equals 0.33). Main arch span stringers span approximately fifty-two feet. For longitudinal members spanning greater than forty feet the dynamic load allowance may be reduced depending on the riding surface conditions. Table 3 - 4 summarizes the dynamic load allowance from Table C6-3 of the LRFR Manual (AASHTO, 2003).

Table 3 - 4: Dynamic Load Allowance: Arch Span Stringers

Riding Surface Conditions	IM
Smooth riding surface at approaches, bridge deck, and expansion joints	0.10
Minor surface deviations or depressions	0.20
Vehicles traveling < 10 mph	0.00

Arch stringers have a dynamic load allowance of 1.20 which corresponds to minor surface deviations on the wearing surface. This factor may be reduced if a new (smooth) riding surface is added at a later time. Section 6.4.5.5 of the LRFR Manual (AASHTO, 2003) indicates that the dynamic load allowance can be eliminated (i.e., IM equals 0.00) for all structural components when vehicles are restricted to traveling at speeds less than ten miles per hour, which is reflected in the third entry in Table 3-4.

4 LOAD RATING THE I. B. PERRINE BRIDGE

The Perrine Bridge is load rated using Excel spreadsheets. The Excel load rating file is organized into forty-three worksheets where reaction data is imported from LARSA and used to load rate bridge members. This section explains both the organization of the load rating spreadsheet and programming functions so that all bridge members are correctly load rated and analysis results are summarized. The operator instructions for performing a basic load rating analysis are attached in Appendix O. Section 4.1 describes the organization of I. B. Perrine Bridge model and load rating spreadsheet. Section 4.2 describes how load effects are selected from the load envelopes and used in the LRFR analysis. Section 4.3 summarizes the load rating procedure of the I.B. Perrine Bridge. The member capacity and load rating worksheets were checked by a second analyst.

4.1 Load Rating Organization

4.1.1 Structure Groups

Structure or Geometry Groups allow for efficient selection of a group of similar structural components to retrieve analysis results from LARSA. Related structural components of the bridge are organized into fifty-one Structure Groups in the Perrine Bridge model. For example, stringers of the south approach, north approach, and main arch spans are organized in three separate structure groups. Similarly, each diaphragm of the bridge deck is in a separate structure group. Structure groups are labeled according to the structure type they represent (e.g. "south approach stringer", "ext. S. approach plate girder," "arch span floor beam," or "spandrel column"). Rating factors are summarized according to the structure groups in the Excel load rating spreadsheet.

4.1.2 Control Panel

In this worksheet, the operator activates the LRFR analysis by selecting the "Populate Results" button. The "Populate Results" program performs the following functions:

- Delete previous load rating analysis data
- Import current moving load envelope data from LARSA
- Sort moving load data
- Select applicable load effect to load rate bridge components
- Load rate all bridge members for their principal structural resistances (i.e., beams are load rated for bending and shear reactions and truss members are load rated for axial response)
- Summarize load rating factors according to structure group and lowest rating factor by reaction type (e.g., positive moment, shear, or axial compression)

Sorting moving load data, selecting loads for load rating, and summarizing the load rating analysis are discussed in Sections 4.1.5, 4.2, and 4.1.6 respectively.

The "Data Import" function selects reaction envelopes based on criteria provided in Control Panel worksheet. Figure 4 - 1 displays criteria used by the "Data Import" function to import moving load data from LARSA.

	A	B	C	D	E	F	G
1	**Control Panel**						
2	Result Case	Result Type	Geometry Group	Envelope Column	Target Worksheet	Target Row	Target Col
3	moving load envelope	member sectional forces	S. App. Stringer	9	strngr data M	6	2
4	moving load envelope	member sectional forces	N. App. Stringer	9	strngr data M	6	13
5	moving load envelope	member sectional forces	Exterior Stringer	9	strngr data M	6	24
6	moving load envelope	member sectional forces	Interior Stringer	9	strngr data M	6	35
7	moving load envelope	member sectional forces	S. App. Stringer	5	strngr data V	6	2
8	moving load envelope	member sectional forces	N. App. Stringer	5	strngr data V	6	13
9	moving load envelope	member sectional forces	Exterior Stringer	5	strngr data V	6	24
10	moving load envelope	member sectional forces	Interior Stringer	5	strngr data V	6	35

Figure 4 - 1: Criteria for Importing Moving Load Analysis Data

Each row imports a reaction envelope for a specific structure group and pastes it into a specific worksheet, at a specific row and column location. Ninety-four reaction envelopes are imported from LARSA during a load rating analysis. The following sub-sections describe the criteria listed in each column shown in Figure 4 - 1. The Visual Basic programming for both the "Populate Results" and "Import Data" programs are provided in Appendix P.

4.1.2.1 Column 1: Result Case

LARSA performs numerous types of structural analyses (e.g., linear static, moving load, or dynamic). The Result Case column indicates what type of analysis results to select. Only the moving load envelope result case is imported. To reduce the importing time, dead load effects are permanently saved in the load rating spreadsheet.

4.1.2.2 Column 2: Result Type

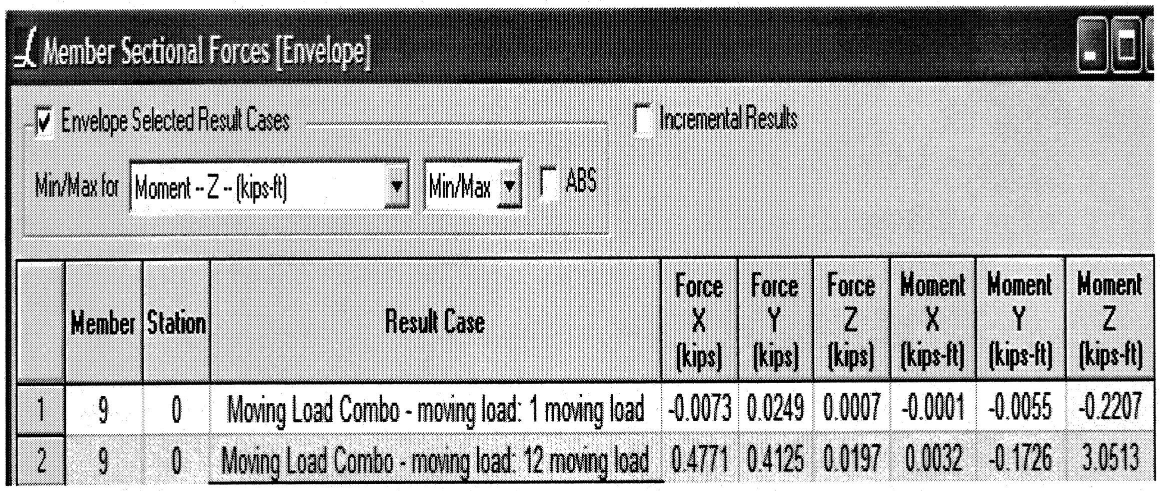
Either member sectional forces or member end forces are selected depending on the reaction envelope needed. Member sectional forces are imported for bending response and member end forces are imported for axial response. The differences between the two result types are described in Section 4.1.5.

4.1.2.3 Column 3: Geometry Group

The envelope reaction data is calculated for the structure group in the geometry group column. Figure 4- 1 lists stringer structure groups; similar calculations are performed for plate girder, floor beam, spandrel column and other groups.

4.1.2.4 Column 4: Envelope Column

Figure 4 - 2 illustrates LARSA analysis results for member section forces. Member end forces are presented with the same column organization.



	Member	Station	Result Case	Force X (kips)	Force Y (kips)	Force Z (kips)	Moment X (kips-ft)	Moment Y (kips-ft)	Moment Z (kips-ft)
1	9	0	Moving Load Combo - moving load: 1 moving load	-0.0073	0.0249	0.0007	-0.0001	-0.0055	-0.2207
2	9	0	Moving Load Combo - moving load: 12 moving load	0.4771	0.4125	0.0197	0.0032	-0.1726	3.0513

Figure 4 - 2: LARSA Format for Presenting Member Sectional Forces

The envelope column number in Figure 4 - 1 corresponds to a result column in LARSA shown in Figure 4 - 2. For example, 9 in the envelope column of the Control Panel worksheet instructs the import sub-program to envelope reactions for Moment-*Z* because it is the ninth column of the member sectional forces envelope in LARSA. Similarly, a column envelope value of 4 envelopes Force-*X* and a column envelope value of 5 envelopes Force-*Y*. Only one reaction column is enveloped at a time. Bridge members requiring multiple load rating factors such as bending members, require multiple reaction envelopes to retrieve all necessary moving load data. Reaction data is described further in Section 4.1.5.

4.1.2.5 Columns 5, 6, & 7: Target Worksheet, Target Row, & Target Column

Moving load envelopes are pasted in the target worksheet. The target worksheet corresponds to the name of the load rate worksheet in which that moving load data is stored. Typically, reaction data is stored in worksheets with names ending in "data" such as arch chord data. The target row and target column specify the paste point within the target worksheet.

4.1.3 User Input

The User Input worksheet contains all of the input parameters used in the rating factor equation. Input parameters are defined in Section 3.2.2. Load rating factors are calculated in multiple worksheets throughout the load rating file; however, all load rating formulae reference the User Input worksheet for input parameter values. Input parameters used by the LRFR analysis must be modified to reflect current system conditions and vehicle loads. Changes to input parameters in the User Input worksheet are applied to load rating equations throughout the entire load rating file.

4.1.4 Member Capacity

The Member Capacity worksheet contains all of the structural capacities for all bridge components that are load rated. Similar to the User Input worksheet, all load rating formulae reference the Member Capacity worksheet for member capacity values. Member capacities are pasted into the load rate file from a separate Excel file.

4.1.5 Load Rating by Structure Group

4.1.5.1 Worksheet Organization

Most worksheets in the load rating file either contain reaction data or calculate load rating factors for bridge members. Typically, structure groups are combined into larger supergroups for load rating to minimize the number of worksheets used in the load rating file. For example, all stringer groups (south approach, north approach, main arch exterior, and main arch interior stringers) are load rated using the same worksheets. Reaction data for either a structure group or supergroup is organized in worksheets designated to facilitate data storage. The corresponding rating factors are calculated in separate worksheets using the data stored in the designated data worksheets.

4.1.5.2 Member Reaction Types

As discussed in Section 4.1.2, there are two different types of reaction envelopes imported from LARSA: member sectional forces and member end forces. Both types of member forces are reported with respect to local element coordinate systems. Furthermore, when a reaction is enveloped for either result type, the other five concurrent reactions are also returned. Member sectional forces provide enveloped forces at ten evenly spaced stations along the member length. Bending rating factors (moment and shear) require enveloping member sectional forces because the maximum load effect may occur anywhere along a member's length. When a specific reaction is enveloped in

LARSA for member sectional forces, the minimum and maximum reactions occurring during the moving load analysis are returned for each station of the member.

Member end forces provide the enveloped resultant section forces acting at the ends of a member. Member end forces are enveloped for axial rating factors (compression and tension). Neglecting the effects of self-weight, the axial response of end-loaded elements remains constant over the length of the element. In addition, the axial effects of self-weight will be greatest at either end of a member. Therefore, it is sufficient to envelope the member end forces for axial load rating factors. For member end forces, the two extreme reactions are returned for each end of a member.

4.1.5.3 Sorting Static & Moving Load Analysis Data

When structure group envelopes are imported to Excel, members within the structure group are arranged in no particular order. A sorting program automatically sorts members into a sequential order so that the span envelope program can correctly obtain the largest load effect occurring on all members. Where multiple analytic elements model a single beam span, spans are numbered and sorted in sequential order so that the greatest load effect occurring on any of the analytic elements within the span will be selected when rating that beam. Selecting load effects for load rating is presented in Section 4.2. A sample sorting sub-program is provided in Appendix P.

4.1.6 Load Rating Analysis Summary

The load rating analysis summary worksheet lists the lowest five rating factors and corresponding members for each reaction of each structure group. The sorting program that selects the lowest rating factors for each structure group is attached in Appendix P. The load rating summary worksheet prints as a five-page report. An operator is required to check this summary report to determine if a permit vehicle can safely cross the bridge.

4.2 Selecting Load Effects for Load Rating

One load effect is selected from the self-weight, wearing surface, and moving load envelopes to calculate one representative rating factor for each reaction of each bridge component. The greatest dead load, wearing surface load, and moving load effects of the corresponding member load envelopes are used to calculate load rating factors. This may be conservative in some scenarios because this assumes that the greatest load effects for all three load analyses occur at the same location on the member. Since bridge components are modeled with varying numbers of analytic elements with sometimes varying section capacities, the process for selecting load effects and determining load rating factors varies depending on the type of structural component. Section 4.2.1 discusses how load effects are selected and load rating factors are calculated for prismatic bending members. Section 4.2.2 describes the load rating of haunched bending members. Finally, Section 4.2.3 explains how load effects are selected and used to load rate axial force members.

4.2.1 Prismatic Bending Members

4.2.1.1 Stringers & Floor Beams

Stringer capacities remain constant through their entire span (from support to support). Numerous analytic elements comprise stringers spans (see Figure 4-3 and Section 2.1.2). The enveloping program searches through the analytic elements comprising a stringer's span and selects the greatest load effect and therefore the most conservative rating factor for that stringer span. One rating factor is calculated per deck stringer span. An identical procedure is performed for other bending members where capacity remains constant over the entire component length (e.g., approach floor beams and arch struts).

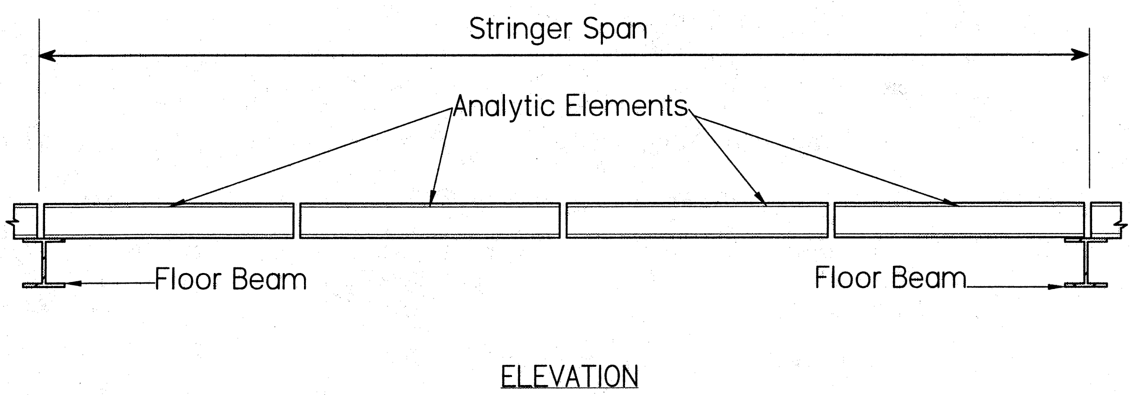


Figure 4 - 3: Stringer Analytic Elements and Stringer Span

4.2.1.2 Plate Girders

Plate girder capacities vary along the girder span. Consequently, analytic elements modeling plate girders do not have the same section properties throughout the girder span. A section span is a span on a plate girder that has constant capacity (i.e., constant section properties). Varying numbers of analytic elements model section spans on plate girders (see Figure 4-4). For example, there may be twenty analytic elements modeling a plate girder span grouped into five constant-capacity section spans. For the plate girder application, the enveloping program searches through the analytic elements defining a section span and selects the greatest load effect and therefore the most conservative rating factor for that section span. The lowest rating factor out of the section spans represents the rating factor for the entire plate girder span. A sample of the prismatic bending span envelope program is presented in Appendix P.

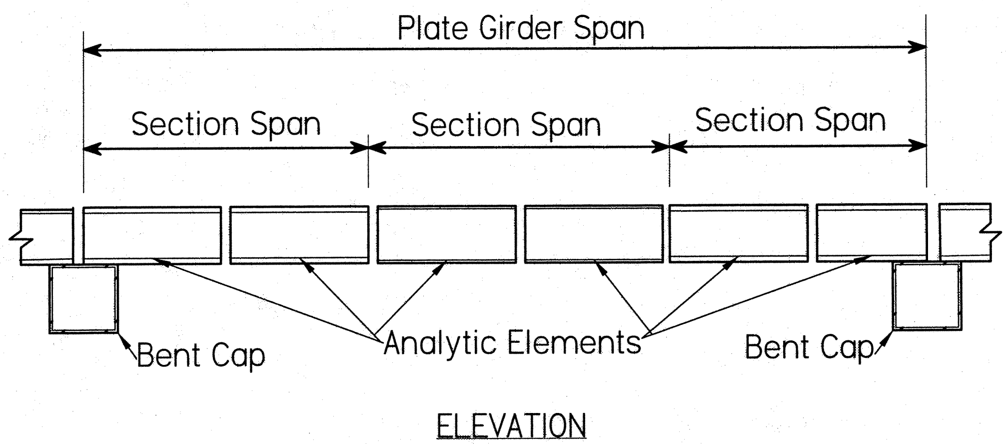


Figure 4 - 4: Plate Girder Analytic Elements, Section Spans, and Plate Girder Span

4.2.2 Haunched Bending Members

Similar to plate girders, the capacity in haunched girders varies along the span length (see Section 3.1.3). However, for haunched spans the calculated capacity is representative of discrete locations in the span due to the changing web height, whereas for plate girders even though the capacity varies along the girder span, the capacity is constant over the section span (see Section 4.2.1). Analytic elements modeling haunched bending members have section capacities calculated based on the average web height occurring approximately at mid-span. Therefore, bending and shear load effects are selected at station five (mid-span) for analytic members of haunch sections. The greatest load effect occurring anywhere along the analytic element span should not be used because the capacity is only accurate at one discrete location (i.e., mid-span). The lowest rating factor of the analytic elements modeling the haunched span represents the load rating factor for the entire haunched span. For example, there are five analytic elements modeling each of the haunch plate girders in the south approach. Therefore, the south approach haunch spans are load rated at five discrete locations, with the lowest rating factor serving as the rating factor for the entire haunch span. A sample of the haunch envelope program is presented in Appendix P.

4.2.3 Axial Force Members

Axial load factors are calculated for each element of an axial member. Some bridge members are modeled with one element and others require multiple analytic elements. Axial capacities are constant along the member length. Therefore, this capacity can be used to calculate rating factors for any element comprising a member. The lowest rating factor of the elements modeling a member span represents the rating factor for the entire member. Enveloping member end forces requires selecting the greatest axial load effect that occurs at the ends of a member. A sample of the axial span envelope program is presented in Appendix P.

4.2.4 Span and Element Numbering

Each member span is assigned a unique number in LARSA. Those span numbers are transferred to the Excel rating spreadsheet and can be used to locate critical spans in LARSA once they have been identified in the Excel load rating spreadsheet. The member data worksheets in the load rating spreadsheet (e.g., "strngr data M") also identify which analytic members comprise a physical member span. (See Section 5 – "Locating Elements within the Perrine Bridge Model" in the *Instruction Manual for Load Rating the I.B. Perrine Bridge*.)

4.3 I.B. Perrine Bridge Load Rating Summary

The I.B. Perrine Bridge is load rated for any vehicle configuration by performing a moving load analysis in LARSA and then load rating it in Excel (see Appendix E and Appendix O). Static self-weight and wearing surface load effects for all bridge components are permanently stored in the load rating file. The Excel load rating procedure automatically performs a series of algorithms. First, previous load rating data is cleared from the Excel load rating spreadsheets. Moving load analysis envelopes are copied from LARSA databases and pasted into Excel. Once in Excel, moving load envelopes are sorted so that load effects can be selected and used to calculate LRFR load rating factors for all bridge components (see Section 4.2). Load rating factors correspond to a Strength II limit state; service and fatigue are not considered. Finally, the lowest five rating factors for the principal reactions of each structure group are summarized in a five-page summary report.

5 SUMMARY & PRELIMINARY CONCLUSIONS

Existing bridge rating procedures make assumptions for live load distribution factors and load paths that only approximately match the geometry of the I.B. Perrine Bridge. This report presents a finite element model which directly calculates dead and live load response without resorting to distribution factors. Section 5.1 summarizes the analysis and load process. Section 5.2 describes preliminary load rating results calculated by the computer model. Future research to calibrate the computer model and verify load rating method presented in this thesis is discussed in Section 5.3.

5.1 Summary

Both load distribution factors of the AASHTO standard specifications and AASHTO LRFD bridge design specifications have geometric ranges of applicability that ensure accurate live load estimations. Even though the AASHTO LRFD formulas consider a greater number of bridge parameters to provide accurate results over a broader range of bridges, the geometry of I.B. Perrine Bridge exceeds these ranges of applicability for multiple parameters. Furthermore, load rating methods using live load distribution factors and simplified structural analyses typically only provide a load rating factor for the primary structural components directly supporting the bridge deck. Rather than requiring bridge engineers to make assumptions and approximations about the bridge geometry in order to apply the live load distribution formulas, a finite element model directly calculates live load effects considering the interaction of all structural members.

A detailed finite element analysis can more accurately calculate live load response and enables a broader range of bridge components to be load rated. Arch truss members, columns, and secondary bracing members are load rated that would otherwise be excluded from customary load rating analyses. It is not expected that load rating factors for principal arch truss members and columns will control the bridge rating since they predominately resist the structure's self-weight; however, this can be verified by load rating analyses.

The three-dimensional finite element model of the I.B. Perrine Bridge was created using LARSA 2000 Plus. In this model, similar bridge components are organized into structure groups so that load rating is performed efficiently. The computer model allows the moving load analysis of any user-specified axle configuration (i.e., any number of axles, axle weight, axle spacing, or axle width). Therefore, any permit vehicle can be analyzed using this model. Moving load analyses can be performed along five different load paths for each travel direction. The computer model is used in conjunction with an Excel database and accompanying Visual Basic programming to load rate the bridge according the AASHTO LRFR procedure for permit vehicles. Load rating is examined for the Strength II limit state. Load rating in Excel consists of performing the following operations:

- Clear previous load rating analysis data
- Import current moving load envelopes from LARSA
- Sort moving load data so that loads can be selected and used for load rating
- Select applicable load effect for load rating bridge components
- Load rate all bridge members for their principal structural resistances (i.e., beams are load rated for bending and shear reactions and truss members are load rated for axial response)
- Summarize load rating factors according to structure group and lowest rating factor by reaction type (e.g., positive moment, shear, or axial compression)

The load rating procedure is automated so that bridge engineers need only to complete a moving load analysis and select a single button in Excel to perform a routine load rating analysis on the entire bridge.

5.2 Preliminary Results

Preliminary load rating analyses show that load rating factors for primary structural components are reasonable. Rating factors for columns and principal arch truss members do not vary significantly with load path or vehicle configuration. This is expected because the dead load resisted in these members is significantly greater than the corresponding live load component of the total load effect. The live load component of the total load effect resisted by principal arch truss members and spandrel columns is approximately five percent of the total load. The dead loads calculated by the finite element model for primary column and principal arch members agree within ten percent of the design dead loads for most members.

Some secondary brace members that are indirectly loaded have rating factors less than zero. This would imply that the dead loads calculated by the computer model exceed the member's capacity. Clearly, the structure does support its self-weight in addition to live loads. Generally, bracing members are not designed to resist direct loads; rather they ensure stability in primary structural components. Standard load rating practice does not consider secondary bracing members for the load rating of a bridge. However, that does not limit the possibility that secondary bracing members will be overloaded during permit vehicle loadings. Further model calibration is required to accurately load rate these members.

5.3 Future Research

Diagnostic tests have been run on the I.B. Perrine Bridge, which provide section strain data for determining member forces incurred during a series of semi-static moving load tests. The results of diagnostic tests will be compared to the results of the uncalibrated computer model for the same vehicle configuration and load paths. There will likely be differences between the actual measured response and modeled analytical response due to modeling assumptions and simplifications inherent in the computer model. The computer model will be calibrated using the measured structural response from diagnostic testing by adjusting boundary conditions related to support, as well as geometry, and stiffness of members. The load rating results calculated using the calibrated computer model will be compared to empirical load rating methods, providing a comparison between a detailed structural analysis and typical, simplified methods.

The completed research project will provide the Idaho Transportation Department (ITD) with a more accurate load rating method that is simple to use. Costly traffic restrictions and unnecessary bridge strengthening or repairs can be minimized with a more accurate load rating method. The increased load rating accuracy will benefit the state by getting the most utility from the state's investment in the bridge without damaging the bridge by imposing excessive loads.

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Appendix A. PRISMATIC ELEMENT SECTION PROPERTY SUMMARY

Table A - 1: Rolled Stringer & Approach Floor Beam Element Section Properties

Sections	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
Rolled Stringers*						
South Approach W21x55 stringer	15.99	7.80	7.15	1.14	1123.30	48.40
North Approach W18x46 (AISC)	13.50	6.50	6.11	1.22	712.00	22.50
Arch Interior W33x118 (AISC)	34.70	18.07	14.16	5.30	5900.00	187.00
Arch Exterior W33x130 (AISC)	38.30	19.19	16.40	7.37	6710.00	218.00
Approach Floor Beams						
S. Approach Ext. W30x124 (AISC)	36.50	17.65	16.30	7.99	5360.00	181.00
S. Approach Int. W24x76 (AISC)	22.40	10.52	10.19	2.68	2100.00	82.50
S. Approach Bent Ext. W10x72	20.97	5.36	13.70	4.04	416.60	141.75
S. Approach Bent Int. W10x49 (AISC)	14.40	3.39	9.33	1.39	272.00	93.40
N. Approach Ext. W30x116 (AISC)	34.20	16.96	14.87	6.43	4930.00	164.00
N. Approach Int. W24x68 (AISC)	20.10	9.85	8.74	1.87	1830.00	70.40
N. Approach Bent Ext. W10x68(AISC)	20.00	4.89	13.00	3.56	394.00	134.00
N. Approach Bent Int. W10x49(AISC)	14.40	3.39	9.33	1.39	272.00	93.40

* Section properties presented in the table are non-composite section properties.

Table A - 2: Prismatic Plate Girder Element Section Properties

*Sections**	Section Area (in²)	Shear Area yy (in²)	Shear Area zz (in²)	Torsional Constant (in⁴)	Inertia I_{zz} (in⁴)	Inertia I_{yy} (in⁴)
Exterior South Approach Girder						
south g. 1	59.00	23.25	30.00	12.85	40739.9	1048.9
south g. 2	86.50	23.25	60.00	65.90	67900.0	1846.0
south g. 3	94.50	23.25	60.00	88.32	75966.0	2016.2
south g. 4	107.00	23.25	60.00	149.52	89017.6	2432.9
south g. 5	99.00	23.25	56.25	139.64	80952.9	2262.2
south g. 6	89.34	23.25	56.25	83.03	70471.3	1928.9
Interior South Approach Girder						
south g. 1	59.00	23.25	30.00	12.85	40739.9	1048.9
south g. 2	86.50	23.25	60.00	65.90	67900.0	1846.0
south g. 3	94.50	23.25	60.00	88.32	75966.0	2016.2
south g. 4	107.00	23.25	60.00	149.52	89017.6	2432.9
south g. 5	99.00	23.25	56.25	139.64	80952.9	2262.2
south g. 6	89.34	23.25	56.25	83.03	70471.3	1928.9
Exterior North Approach Girder						
N. ap. g. 1	50.25	21.09	24.38	18.39	27840.9	451.2
N. ap. g. 2	73.25	21.75	43.33	87.37	46735.7	787.9
N. ap. g. 3	67.25	21.56	37.92	77.87	41947.2	715.9
Interior North Approach Girder						
N. ap. g. 1	50.25	21.09	24.38	18.39	27840.9	451.2
N. ap. g. 2	73.25	21.75	43.33	87.37	46735.7	787.9
N. ap. g. 3	67.25	21.56	37.92	77.87	41947.2	715.9
Exterior South Arch Girder						
S str. h. arch 6	33.88	11.63	19.17	8.21	5939.0	253.6
Interior South Arch Girder						
S str. h. arch' 6	28.31	11.63	14.38	3.78	4749.2	190.2
Exterior North Arch Girder						
north str. hnch 6 o	34.63	12.38	19.17	8.25	6820.8	253.6
Interior North Arch Girder						
north str. hnch 6 i	29.06	12.38	14.38	3.81	5462.8	190.2

* All plate girder section properties presented in the table are non-composite section properties.

Table A - 3: Arch Chord Element Section Properties

Sections	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
Upper Chords:						
U_o-U_2	67.25	37.38	29.25	6456.92	5583.04	3682.55
U_2-U_4	77.50	43.13	33.75	7439.62	6518.72	4177.83
U_4-U_6	83.25	46.00	36.00	7677.34	6772.20	4540.11
U_6-U_8	92.25	51.75	40.50	9095.38	8178.49	4783.11
U_8-U_{10}	98.00	54.63	42.75	9372.61	8431.97	5134.07
U_{10}-U_{12}	102.50	57.50	45.00	10060.26	9156.35	5255.57
U_{12}-U_{14}	102.50	57.50	45.00	10060.26	9156.35	5255.57
U_{14}-U_{16}	98.00	54.63	42.75	9372.61	8431.97	5134.07
U_{16}-U_{18}	93.50	51.75	40.50	8641.93	7721.79	5012.57
U_{18}-U_{19}	93.50	51.75	40.50	8641.93	7721.79	5012.57
Lower Chords:						
L_o-L_2	179.63	92.81	86.63	20239.00	16667.43	11639.96
L_2-L_4	158.38	82.47	76.13	18622.00	15395.60	10290.90
L_4-L_6	142.20	73.94	68.25	16568.30	13341.50	9463.30
L_6-L_8	126.00	65.41	60.38	14490.60	11355.60	8623.10
L_8-L_{10}	115.25	60.38	55.13	13824.70	11121.80	7736.95
L_{10}-L_{12}	104.75	54.63	49.88	12190.90	9414.84	7351.10
L_{12}-L_{14}	99.00	51.75	47.25	11756.50	9161.35	6852.40
L_{14}-L_{16}	93.75	48.88	44.63	10921.40	8332.80	6659.47
L_{16}-L_{18}	88.00	46.00	42.00	10472.70	8079.33	6147.33
L_{18}-L_{19}	88.00	46.00	42.00	10472.70	8079.00	6147.00

Table A - 4: Arch Diagonal Element Section Properties

Sections	**Section Area (in^2)**	**Shear Area yy (in^2)**	**Shear Area zz (in^2)**	**Torsional Constant (in^4)**	**Inertia I_{zz} (in^4)**	**Inertia I_{yy} (in^4)**
Diagonals:						
U_o-L_1	26.38	12.00	12.50	2.95	2540.06	180.24
U_1-L_2	26.38	12.00	12.50	2.95	2540.06	180.24
U_2-L_3	22.06	10.50	10.00	1.67	2100.59	144.16
U_3-L_4	22.06	10.50	10.00	1.67	2100.59	144.16
U_4-L_5	22.06	10.50	10.00	1.67	2100.59	144.16
U_5-L_6	22.06	10.50	10.00	1.67	2100.59	144.16
U_6-U_7	22.06	10.50	10.00	1.67	2100.59	144.16
U_7-L_8	22.06	10.50	10.00	1.67	2100.59	144.16
U_8-L_9	22.06	10.50	10.00	1.67	2100.59	144.16
U_9-L_{10}	22.06	10.50	10.00	1.67	2100.59	144.16
U_{10}-L_{11}	22.06	10.50	10.00	1.67	2100.59	144.16
U_{11}-L_{12}	23.50	12.00	10.00	2.00	2163.96	144.24
U_{12}-L_{13}	23.50	12.00	10.00	2.00	2163.96	144.24
U_{13}-L_{14}	28.88	12.00	14.58	3.28	2881.63	286.07
U_{14}-L_{15}	28.88	12.00	14.58	3.28	2881.63	286.07
U_{15}-L_{16}	28.88	12.00	14.58	3.28	2881.63	286.07
U_{16}-U_{17}	28.88	12.00	14.58	3.28	2881.63	286.07
U_{17}-L_{18}	28.88	12.00	14.58	3.28	2881.63	286.07
U_{18}-L_{19}	35.06	15.00	17.50	5.89	3432.20	343.46

Table A - 5: Arch Post, Strut, Chevron, & Cross Brace Element Section Properties

Sections	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
Posts:						
U_o - L_o	108.91	86.13	39.00	33052.36	48681.80	11627.40
U_1-L_1	43.88	18.00	22.50	8.44	4361.98	729.79
U_2-L_2	59.53	21.00	33.75	22.45	6052.58	1094.71
U_3-L_3	38.06	15.00	20.00	6.45	3837.76	512.46
U_4-L_4	45.41	15.00	26.25	9.99	4786.99	850.95
U_5-L_5	31.72	15.00	14.58	4.23	3004.29	286.30
U_6-L_6	39.56	15.00	21.25	6.73	4040.54	614.58
U_7-L_7	31.72	15.00	14.58	4.23	3004.29	286.30
U_8-L_8	39.56	15.00	21.25	6.73	4040.54	614.58
U_9-L_9	28.88	12.00	14.58	3.28	2881.63	286.07
U_{10}-L_{10}	31.72	15.00	14.58	4.23	3004.29	286.30
U_{11}-L_{11}	26.38	12.00	12.50	2.95	2540.06	180.24
U_{12}-L_{12}	31.72	15.00	14.58	4.23	3004.29	286.30
U_{13}-L_{13}	23.50	12.00	10.00	2.00	2163.96	144.24
U_{14}-L_{14}	31.72	15.00	14.58	4.23	3004.29	286.30
U_{15}-L_{15}	23.50	12.00	10.00	2.00	2163.96	144.24
U_{16}-L_{16}	31.72	15.00	14.58	4.23	3004.29	286.30
U_{17}-L_{17}	23.50	12.00	10.00	2.00	2163.96	144.24
U_{18}-L_{18}	26.38	12.00	12.50	2.95	2540.06	180.24
U_{19}-L_{19}	19.81	11.50	7.29	1.52	1564.99	73.15
Struts:						
U Strut	15.78	7.36	7.29	0.90	1011.48	73.00
L Strut	19.81	11.50	7.29	1.52	1564.99	73.15
Chevron Braces:						
U diagonal	18.98	7.36	10.00	1.34	1299.45	144.08
L_1 diagonal	37.78	14.38	20.42	8.12	3499.62	400.60
L` diagonal	28.38	11.50	14.58	3.24	2619.59	286.06
X- Braces:						
2L8x4x7/16	10.12	7.00	2.92	2.01	68.22	19.09

Table A - 6: Column & Column Brace Element Section Properties

Sections	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
Spandrel Columns						
C_o	108.91	86.13	39.00	33052.36	48681.80	11627.40
C_2	134.75	67.81	66.94	49848.00	34765.00	31847.00
C_4	92.25	46.50	45.75	21760.90	15255.70	13837.60
C_6	62.50	31.56	30.94	9741.70	6856.20	6172.76
C_8	57.50	29.06	28.44	7582.40	5362.55	4784.10
C_{10}	57.50	29.06	28.44	7582.40	5362.55	4784.10
C_{12}	57.50	29.06	28.44	7582.40	5362.55	4784.10
C_{14}	57.50	29.06	28.44	7582.40	5362.55	4784.10
C_{16}	68.25	34.50	33.75	7983.94	6281.70	5505.60
C_{18}	68.25	34.50	33.75	7983.94	6281.70	5505.60
South Bent Columns						
bent col. 1	120.00	66.25	60.00	17186.38	12451.25	10651.25
North Bent Columns						
bent col. 2	87.06	45.06	45.06	13467.82	8989.66	8989.66
Spandrel Column Bracing						
HP14x73 w/ .5PL	33.82	14.60	13.00	1172.56	1003.38	899.50
L_o lower strut	28.11	20.13	8.75	1267.89	1903.45	515.79
HP14x73(AISC)	21.40	6.87	12.28	2.01	729.00	261.00
South Bent Bracing						
WT9x23(AISC)	6.77	3.25	3.06	0.61	52.10	11.30
WT12x65.5(AISC)	19.30	7.41	10.28	4.74	238.00	170.00
North Bent Bracing						
HP14x73(AISC)	21.40	6.87	12.28	2.01	729.00	261.00
HP14x73 w/ .5PL	33.82	14.60	13.00	1172.56	1003.38	899.50

Table A - 7: Deck Bracing Element Section Properties

Sections	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
South Approach Lateral Bracing						
L6x4x5/16(AISC)	3.03	1.56	1.04	0.11	11.40	4.18
South Bent Diaphragm						
2L7x4x3/8(AISC)	7.97	4.38	2.50	0.00	41.10	0.00
2L7x4x7/16	9.24	6.13	2.92	1.79	47.30	19.03
2L3.5x2.5x5/16	3.58	2.19	0.65	0.31	3.42	0.47
WT4x17.5(AISC)	5.14	1.26	3.31	0.39	4.81	21.30
South Abutment Diaphragm:						
MC12x35(AISC)	10.30	5.60	4.39	1.25	216.00	12.70
South Diaphragm at F_o:						
W12x53(AISC)	15.60	4.16	9.58	1.58	425.00	95.80
North Bent Diaphragm:						
2L7x4x3/8(AISC)	7.97	4.38	2.50	0.00	41.10	0.00
2L7x4x7/16	9.24	6.13	2.92	1.79	47.30	19.03
2L3.5x2.5x5/16	3.58	2.19	0.65	0.31	3.42	0.47
WT4x17.5(AISC)	5.14	1.26	3.31	0.39	4.81	21.30
North Abutment Diaphragm:						
MC12x35(AISC)	10.30	5.60	4.39	1.25	216.00	12.70
North Diaphragm at F'_o:						
W12x53(AISC)	15.60	4.16	9.58	1.58	425.00	95.80
Arch Span Lateral Bracing						
WT12x65.5(AISC)	19.30	7.41	10.28	4.74	238.00	170.00
Arch Span Floor Beam Diaphragm:						
5/16" Bent Plate	6.99	4.69	2.08	0.23	212.20	8.83
Arch Span Diaphragm A:						
W12x53(AISC)	15.60	4.16	9.58	1.58	425.00	95.80
Arch Span End Diaphragm:						
W12x40(AISC)	11.80	3.52	6.87	0.95	310.00	44.10
L6x3½x3/8(AISC)	3.42	1.88	1.09	0.17	12.90	3.34
L3x2½x3/8(AISC)	1.92	0.94	0.78	0.09	1.66	1.04
Arch Span Traction Bracing:						
L3x2½x3/8(AISC)	1.92	0.94	0.78	0.09	1.66	1.04
L3½x3x3/8(AISC)	2.30	1.09	0.94	0.11	2.72	1.85

Appendix B. HAUNCH SUB-ELEMENT DIMENSION & PROPERTY SUMMARY

Table B - 1: South Approach Haunch Sub-Element Section Dimensions

Section	Upper Flange Width (in)	Upper Flange Thickness (in)	Lower Flange Width (in)	Lower Flange Thickness (in)	Web Height (in)	Web Thickness (in)
South Approach Exterior Girder						
South ap. gird 1	16.00	2.63	20.00	2.13	61.84	0.38
South ap. gird 2	16.00	2.63	20.00	2.13	69.19	0.38
South ap. gird 3	16.00	3.13	20.00	2.50	84.19	0.38
South ap. gird 4	16.00	3.13	20.00	2.50	106.25	0.38
South ap. gird 5	16.00	3.13	20.00	2.50	122.00	0.38
South Approach Interior Girder						
South ap. gird 1	16.00	2.63	20.00	2.13	61.84	0.38
South ap. gird 2	16.00	2.63	20.00	2.13	69.19	0.38
South ap. gird 3	16.00	3.13	20.00	2.50	84.19	0.38
South ap. gird 4	16.00	3.13	20.00	2.50	106.25	0.38
South ap. gird 5	16.00	3.13	20.00	2.50	122.00	0.38

Table B - 2: South Approach Haunch Sub-Element Section Properties

Section*	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
South Approach Exterior Girder						
So. ap. gird 1	107.69	24.79	71.25	146.99	94549.66	2312.94
So. ap. gird 2	110.45	25.95	71.25	147.11	118571.95	2312.97
So. ap. gird 3	131.57	31.57	86.25	244.25	207940.48	2733.70
So. ap. gird 4	139.84	39.84	86.25	244.60	334918.11	2733.80
So. ap. gird 5	145.75	45.75	86.25	244.86	446268.43	2733.87
South Approach Interior Girder						
So. ap. gird 1	107.69	24.79	71.25	146.99	94549.66	2312.94
So. ap. gird 2	110.45	25.95	71.25	147.11	118571.95	2312.97
So. ap. gird 3	131.57	31.57	86.25	244.25	207940.48	2733.70
So. ap. gird 4	139.84	39.84	86.25	244.60	334918.11	2733.80
So. ap. gird 5	145.75	45.75	86.25	244.86	446268.43	2733.87

* All section properties presented in the table are non-composite section properties.

Table B - 3: North Approach Haunch Sub-Element Section Dimensions

Section	Upper Flange Width (in)	Upper Flange Thickness (in)	Lower Flange Width (in)	Lower Flange Thickness (in)	Web Height (in)	Web Thickness (in)
South Approach Exterior Girder						
N. h. ap. g. 1	12.00	2.5	14.00	2.125	53.13	0.375
N. h. ap. g. 2	12.00	2.5	14.00	2.125	66.19	0.375
N. h. ap. g. 3	12.00	2.5	14.00	2.125	89.50	0.375
N. h. ap. g. 4	12.00	2.5	14.00	2.125	102.69	0.375
South Approach Interior Girder						
N. h. ap. g. 1	12.00	2.5	14.00	2.125	53.13	0.375
N. h. ap. g. 2	12.00	2.5	14.00	2.125	66.19	0.375
N. h. ap. g. 3	12.00	2.5	14.00	2.125	89.50	0.375
N. h. ap. g. 4	12.00	2.5	14.00	2.125	102.69	0.375

Table B - 4: North Approach Haunch Sub-Element Section Properties

Section*	Section Area (in²)	Shear Area yy (in²)	Shear Area zz (in²)	Torsional Constant (in⁴)	Inertia I_{zz} (in⁴)	Inertia I_{yy} (in⁴)
North Approach Exterior Girder						
N. h. ap. g. 1	79.7	21.98	50.10	108.2	50631.40	846.20
N. h. ap. g. 2	84.6	26.55	50.10	108.4	79186.62	846.2
N. h. ap. g. 3	93.3	35.30	50.10	108.9	148349.40	846.3
N. h. ap. g. 4	98.3	40.24	50.10	109.1	198563.9	846.4
North Approach Interior Girder						
N. h. ap. g. 1	79.7	21.98	50.10	108.2	50631.40	846.20
N. h. ap. g. 2	84.6	26.55	50.10	108.4	79186.62	846.2
N. h. ap. g. 3	93.3	35.30	50.10	108.9	148349.40	846.3
N. h. ap. g. 4	98.3	40.24	50.10	109.1	198563.9	846.4

* All section properties presented in the table are non-composite section properties.

Table B - 5: South Arch Girder Sub-Element Section Dimensions

Section	Web Height (in)	Flange Width* (in)	Flange Thickness** (in)	Web Thickness (in)
South Arch Exterior Girder				
S. str. h. arch 1	61.50	11.50	1.00	0.38
S. str. h. arch 2	53.59	11.50	1.00	0.38
S. str. h. arch 3	42.09	11.50	1.00	0.38
S. str. h. arch 4	35.34	11.50	1.00	0.38
S. str. h. arch 5	31.59	11.50	1.00	0.38
S. str. h. arch 6	31.00	11.50	1.00	0.38
South Arch Interior Girder				
S. str. h. arch' 1	61.50	11.50	0.75	0.38
S. str. h. arch' 2	53.72	11.50	0.75	0.38
S. str. h. arch' 3	42.41	11.50	0.75	0.38
S. str. h. arch' 4	35.78	11.50	0.75	0.38
S. str. h. arch' 5	32.09	11.50	0.75	0.38
S. str. h. arch' 6	31.50	11.50	0.75	0.38

* Both upper and lower flange widths are the same dimension.
** Both upper and lower flange thicknesses are the same dimension.

Table B - 6: South Arch Girder Sub-Element Section Properties

Section*	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
South Arch Exterior Girder						
S. str. h. arch 1	45.31	23.06	19.17	8.75	27631.01	253.74
S. str. h. arch 2	42.35	20.10	19.17	8.61	20198.18	253.71
S. str. h. arch 3	38.04	15.79	19.17	8.41	11726.02	253.66
S. str. h. arch 4	35.50	13.25	19.17	8.29	7942.47	253.63
S. str. h. arch 5	34.10	11.85	19.17	8.22	6192.10	253.61
S. str. h. arch 6	33.88	11.63	19.17	8.21	5939.07	253.61
South Arch Interior Girder						
S. str. h. arch' 1	39.75	23.06	14.38	4.32	22666.36	190.37
S. str. h. arch' 2	36.78	20.10	14.38	4.18	16460.58	190.34
S. str. h. arch' 3	32.47	15.79	14.38	3.97	9462.61	190.29
S. str. h. arch' 4	29.94	13.25	14.38	3.86	6373.07	190.26
S. str. h. arch' 5	28.53	11.85	14.38	3.79	4953.82	190.24
S. str. h. arch' 6	28.31	11.63	14.38	3.78	4749.28	190.24

* All section properties presented in the table are non-composite section properties.

Table B - 7: North Arch Girder Sub-Element Section Dimensions

Section	Web Height (in)	Flange Width* (in)	Flange Thickness** (in)	Web Thickness (in)
North Arch Exterior Girder				
north str. hnch 1 o	58.00	11.50	1.00	0.38
north str. hnch 2 o	51.53	11.50	1.00	0.38
north str. hnch 3 o	42.09	11.50	1.00	0.38
north str. hnch 4 o	36.56	11.50	1.00	0.38
north str. hnch 5 o	33.50	11.50	1.00	0.38
north str. hnch 6 o	33.00	11.50	1.00	0.38
North Arch Interior Girder				
north str. hnch 1 i	57.50	11.50	0.75	0.38
north str. hnch 2 i	51.16	11.50	0.75	0.38
north str. hnch 3 i	41.94	11.50	0.75	0.38
north str. hnch 4 i	36.50	11.50	0.75	0.38
north str. hnch 5 i	33.47	11.50	0.75	0.38
north str. hnch 6 i	33.00	11.50	0.75	0.38

* Both upper and lower flange widths are the same dimension.
** Both upper and lower flange thicknesses are the same dimension.

Table B - 8: North Arch Girder Sub-Element Section Properties

Section*	Section Area (in^2)	Shear Area yy (in^2)	Shear Area zz (in^2)	Torsional Constant (in^4)	Inertia I_{zz} (in^4)	Inertia I_{yy} (in^4)
North Arch Exterior Girder						
north str. hnch 1 o	44.00	21.75	19.17	8.69	24171.67	253.73
north str. hnch 2 o	41.57	19.32	19.17	8.57	18481.22	253.70
north str. hnch 3 o	38.04	15.79	19.17	8.41	11725.86	253.66
north str. hnch 4 o	35.96	13.71	19.17	8.31	8564.12	253.63
north str. hnch 5 o	34.81	12.56	19.17	8.26	7052.10	253.62
north str. hnch 6 o	34.63	12.38	19.17	8.25	6820.89	253.62
North Arch Interior Girder						
north str. hnch 1 i	38.25	21.56	14.38	4.25	19377.48	190.36
north str. hnch 2 i	35.87	19.18	14.38	4.13	14784.20	190.33
north str. hnch 3 i	32.41	15.73	14.38	3.97	9382.92	190.29
north str. hnch 4 i	30.38	13.69	14.38	3.88	6852.30	190.26
north str. hnch 5 i	29.24	12.55	14.38	3.82	5638.41	190.25
north str. hnch 6 i	29.06	12.38	14.38	3.81	5462.82	190.25

* All section properties presented in the table are non-composite section properties.

Appendix C. RIGID LINK STIFFNESS SCALING ANALYSIS

The following Mathcad sheets describe the rigid link stiffness calculations.

Nick McDowell **Rigid Link Stiffness Analysis**

Define System Properties:

Determine appropriate stiffness coefficients for rigid link elements of the Perrine Bridge model. The exterior haunch girder of the south approach will be used to calibrate stiffness characteristics of rigid link element because this girder has the greatest stiffness characteristics for any structural member requiring rigid links in the finite element model. Stiffness coefficients for the rigid link shall be three to four orders of magnitude larger than that of the bending elements. The haunch girder is divided into five sub-elements. Calibrating rigid links according to the stiffest structural member in the model will ensure that rigid links will be adequately stiff for all other bending elements in the finite element program.

$E_i := 29000 \cdot ksi$

$n_e := 8$ number of elements — $i := 1 .. n_e$ counting index for elements

$n_b := 5$ number of beam elements — $b := 1 .. n_b$ counting index for beam elements

$r := 6 .. n_e$ counting index for rigid elements

Define Element Properties:

Sub-element Properties

$L_1 := 1.2 \cdot ft$ $L_2 := 10.83 \cdot ft$ $L_3 := 12.03 \cdot ft$ $L_4 := 7.25 \cdot ft$ $L_5 := 11.69 \cdot ft$

$I_{xx_1} := 446270 \cdot in^4$ $I_{xx_2} := 334290 \cdot in^4$ $I_{xx_3} := 207940 \cdot in^4$ $I_{xx_4} := 118570 \cdot in^4$ $I_{xx_5} := 95850 \cdot in^4$

$I_{yy_1} := 2733.87 \cdot in^4$ $I_{yy_2} := 2733.87 \cdot in^4$ $I_{yy_3} := 2733.87 \cdot in^4$ $I_{yy_4} := 2313.97 \cdot in^4$ $I_{yy_5} := 2313.97 \cdot in^4$

$A_1 := 147.1 \cdot in^2$ $A_2 := 141.2 \cdot in^2$ $A_3 := 132.9 \cdot in^2$ $A_4 := 109.7 \cdot in^2$ $A_5 := 108.5 \cdot in^2$

Rigid link element lengths

$L_6 := 0.67 \cdot ft$ $L_7 := 0.92 \cdot ft$ $L_8 := 0.89 \cdot ft$

Only three rigid links are used to connect consecutive sub-elements in the stepped element model. Other sub-element center lines are close enough that sub-elements are connected through a mutually shared joint.

The deepest web height is in element 1 and the shortest web height is in element 5. Rigid link element 6 connects elements 1 and 2. Rigid link element 7 connects elements 2 and 3. Rigid link element 8 connects elements 3 and 4.
Find cross sectional area such that rigid link axial stiffness coefficient is 1000 time greater than greatest bending element axial stiffness coefficient.

Axial Stiffness Coefficient:

$$A_6 := \frac{1000 \cdot A_1 \cdot L_6}{L_1} \qquad A_7 := A_6$$

$A_8 := A_6$ $\qquad A_6 = 82130.8 \cdot in^2$

Calculate the ratio of rigid link stiffness coefficient to that of sub-elements in stepped element model of exterior south approach girder.

$$\frac{\frac{A_6 \cdot E_6}{L_6}}{\frac{A_1 \cdot E_1}{L_1}} = 1000 \qquad \frac{\frac{A_7 \cdot E_7}{L_7}}{\frac{A_2 \cdot E_2}{L_2}} = 6847.19 \qquad \frac{\frac{A_8 \cdot E_8}{L_8}}{\frac{A_3 \cdot E_3}{L_3}} = 8353.28$$

$$\frac{A_6 \cdot E_6}{L_6} = 2.96 \times 10^8 \frac{kip}{in} \qquad \frac{A_1 \cdot E_1}{L_1} = 2.96 \times 10^5 \frac{kip}{in}$$

Using rigid link cross-section area equal to 82130.8 in^2 will provide an axial stiffness coefficient three orders of magnitude greater than the stiffest sub-element of the finite element program. Relative stiffness between rigid links and bending elements will be greater for bending elements with lesser stiffness coefficients.

Strong Axis Bending Stiffness Coefficient:

Rigid section moments of inertia will be adjusted provide adequate stiffness approximately 1000 times greater than the stiffest beam element. There are three different bending stiffness coefficients. Each coefficient will be considered to determine the greatest moment of inertia required for rigid link elements.

using $K = 12\ EI/L^3$

$$I_{xx_6} := 1000 \cdot \frac{I_{xx_1}}{(L_1)^3} \cdot (L_6)^3$$

$$I_{xx_6} = 7.77 \times 10^7 \, in^4$$

using $K = 6\ EI/L^2$

$$I_{xx_6} := 1000 \cdot \frac{I_{xx_1}}{(L_1)^2} \cdot (L_6)^2$$

$$I_{xx_6} = 1.39 \times 10^8 \, in^4$$

USE $K = 4\ EI/L$:

$$I_{xx_6} := 1000 \cdot \frac{I_{xx_1}}{L_1} \cdot L_6$$

$$I_{xx_6} = 2.49 \times 10^8 \, in^4 \qquad I_{xx_7} := I_{xx_6} \qquad I_{xx_8} := I_{xx_6}$$

Calculate ratio rigid link stiffness coefficient to that of sub-elements in stepped element model of exterior south approach girder.

$$\frac{\frac{I_{xx_6}}{L_6}}{\frac{I_{xx_1}}{L_1}} = 1000 \qquad \frac{\frac{I_{xx_7}}{L_7}}{\frac{I_{xx_2}}{L_2}} = 8774.22 \qquad \frac{\frac{I_{xx_8}}{L_8}}{\frac{I_{xx_3}}{L_3}} = 16196.79$$

Strong axis rigid link bending stiffness

$$\frac{4 I_{xx_6} \cdot E_6}{L_6} = 3.59 \times 10^{12}\, kip \cdot in$$

Strong axis south approach girder bending stiffness

$$\frac{4 \cdot I_{xx_1} \cdot E_1}{L_1} = 3.59 \times 10^{9}\, kip \cdot in$$

Weak Axis Bending Stiffness Coefficient:

Rigid section moments of inertia will be adjusted to provide stiffness coefficients approximately 1000 times greater than the stiffest beam element adequate stiffness. There are three different bending stiffness coefficients. Each coefficient will be considered to determine the greatest moment of inertia required for rigid link elements.

using $K = 12\, EI/L^3$

$$I_{yy_6} := 1000 \cdot \frac{I_{yy_1}}{(L_1)^3} \cdot (L_6)^3 \qquad I_{yy_6} = 4.76 \times 10^5\, in^4$$

using $K = 6\, EI/L^2$

$$I_{yy_6} := 1000 \cdot \frac{I_{yy_1}}{(L_1)^2} (L_6)^2 \qquad I_{yy_6} = 8.52 \times 10^5\, in^4$$

USE $K = 4\, EI/L$:

$$I_{yy_6} := 1000 \cdot \frac{I_{yy_1}}{L_1} \cdot L_6 \qquad I_{yy_6} = 1.53 \times 10^6\, in^4$$

$$I_{yy_8} := I_{yy_6} \qquad I_{yy_7} := I_{yy_6}$$

Calculate the ratio of rigid link stiffness coefficient to that of sub-elements in stepped element model of exterior south approach girder.

$$\frac{\frac{I_{yy_6}}{L_6}}{\frac{I_{yy_1}}{L_1}} = 1000 \qquad \frac{\frac{I_{yy_7}}{L_7}}{\frac{I_{yy_2}}{L_2}} = 6572.55 \qquad \frac{\frac{I_{yy_8}}{L_8}}{\frac{I_{yy_3}}{L_3}} = 7546.91$$

Strong axis rigid link bending stiffness

$$\frac{4 I_{yy_6} \cdot E_6}{L_6} = 2.2 \times 10^{10}\, kip \cdot in$$

Strong axis south approach girder bending stiffness

$$\frac{4 \cdot I_{yy_1} \cdot E_1}{L_1} = 2.2 \times 10^{7}\, kip \cdot in$$

Appendix D. ARCH TRUSS NODAL NUMBERING SCHEME

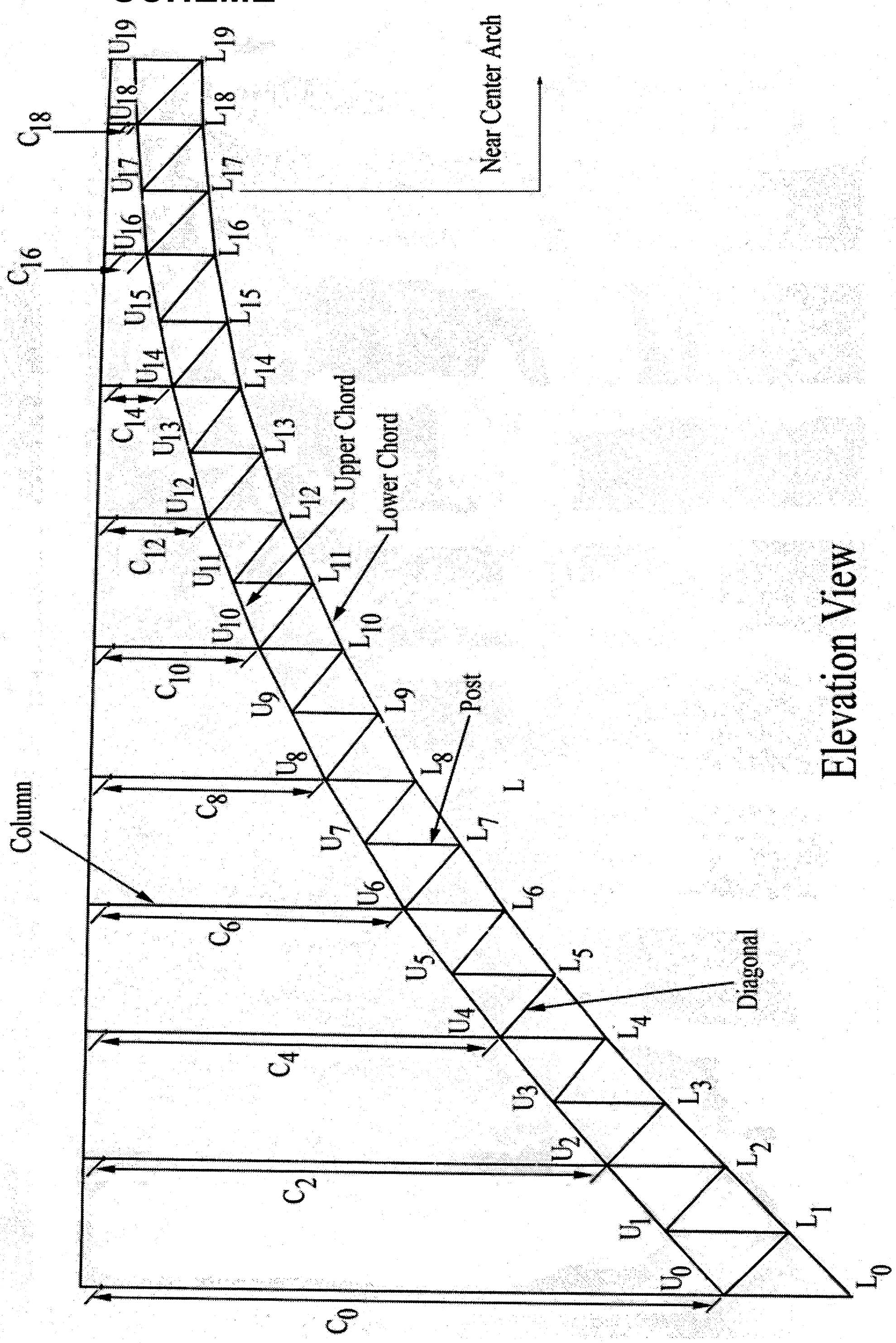


Figure D - 1: Arch Truss Nodal Numbering (Sym. About Node 19)

Appendix E. PROCEDURE FOR ADDING NEW VEHICLE CONFIGURATION

This appendix describes how to input a new moving load pattern into LARSA's moving load database so that it can be used for a moving load analysis.

E.1 Instructions: Adding New Vehicle Pattern

Over-permit vehicles will likely require a new vehicle load pattern to run a moving load analysis. A vehicle load pattern is a data file that indicates the location of wheel contact points with respect to the load path and the magnitude of the forces at the contact points. The moving load database contains pre-defined American standard vehicle load patterns as well as custom load patterns. A new vehicle load pattern is added to the moving load database as follows.

1. Open LARSA 2000 Plus.
2. Open "PERRINE BRIDGE MASTER COPY" file.
   a. File → Open → Local Disk (D:) → Perrine Bride Project folder → "PERRINE BRIDGE MASTER COPY"
3. Under the Input Data Menu, located at the top of the screen, select **Edit Databases**.

   Comments:
   After selecting the Edit Databases tab, the attached databases are shown in the Database Editor screen. The only database that is attached is "movedata_Perrine_Bridge.dml." Only one moving load pattern database can be attached at a time, so additional vehicle load patterns **MUST** be added to this database file.

   When LARSA is run for the first time on a new machine, the program will ask for the location of the moving loads database. Select movedata_Perrine_Bridge.dml from the appropriate directory in the dialog box. Otherwise, once LARSA is running, Under the **Input Data** menu located at the top of the screen, select **Connect Databases...** and select "movedata_Perrine_Bridge.dml." in the dialog box.

4. Select the **(+) Button** located on the left hand side of the screen, at mid-height.

   Comments:

   This creates a new moving load pattern record. The name will remain "new record" until it is renamed. Single click the left mouse button over the "new record" to rename a record.

5. Select the new record to show load pattern information.

   Comments:

   At this point, this is a new load pattern file. No information will be displayed.

6. Under the **Spreadsheet** tab, select the first cell under the **Type** column and **press enter**.

   Comments:

   A load pattern consists of individual wheel loads, rather than axle loads, with the wheel locations defined as longitudinal distances measured rearward from the front axle and transverse distances measured from the vehicle centerline. The following steps describe how to enter the load pattern data for a permit vehicle. The options for entering wheel loads are displayed in Figure E - 1.

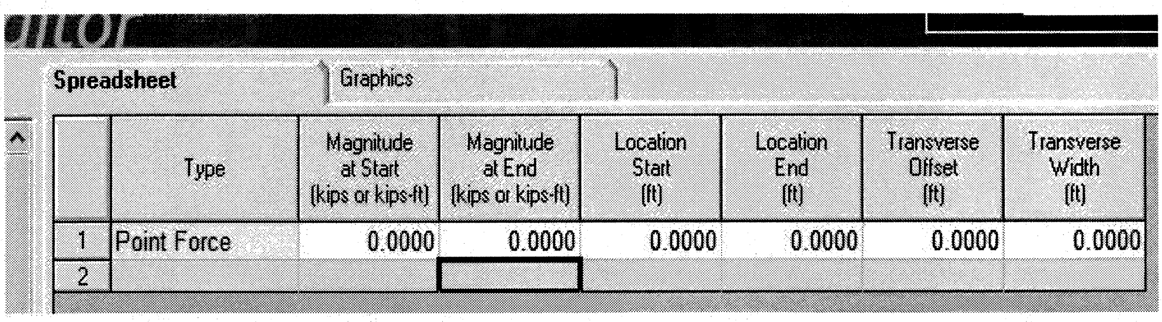


Figure E - 1: Options For Entering Wheel Load

   a. All Contact points are entered as **POINT FORCE**.

   b. For an individual point force, Start and End Magnitudes are identical and represent the magnitude of the force (in kips) at the specific wheel location, i.e., half the axle weight . Two contact points are required per axle, one corresponding to each side of the axle.

Comments:

LARSA allows an option of entering distributed forces where the Start and End Magnitudes may differ depending on the nature of the distributed force. Since all wheel loads are entered as Point Forces, as opposed to Distributed Forces, both Magnitudes are identical. Using half the axle weight as the magnitude of the force at a wheel contact point assumes that the vehicle load is evenly distributed along the axle. If additional information is provided concerning an unbalanced vehicle load, individual wheel loads may be entered to more accurately model an unbalanced load.

c. Start and End Locations are also identical and represent the longitudinal distance (in feet) from the front axle to the axle in question (i.e., the first axle is entered as 0.000 ft for start location and 0.000 ft for end location).

d. Transverse Offset is the lateral spacing of the centroid of the tire footprint with respect to the centerline of the vehicle load pattern. Note: two offsets are entered per axle: one places a wheel load at a transverse offset equal to (+) width/2 and the other at (–) width/2.

e. Transverse Width should be equal to zero for all Point Force loads entered.

Comments:

The Transverse Width corresponds to the width of the tire footprint. When the Transverse Width is entered as zero, the wheel contact load is simplified to a point load. If more accuracy is desired and the width of the tire footprint is known, the wheel contact loads can be distributed along the Transverse

Width. Typically, wheel contact loads are entered as point loads, neglecting the width of the tire footprint.

f. Repeat steps a – e for all wheel contact points to finish the custom load pattern.

Comments:

The lane selection in LARSA does not check for interference between the moving load and the median, or pedestrian barriers. The operator must verify that the vehicle width defined here does not exceed the width of the lane selected for the analysis.

7. Once the load pattern is named (step 4) and created (step 6), click the **Save Database Button**. This is located near the top of the database editor screen.

Comments:

This step adds the vehicle configuration to the "movedata_Perrine_Bridge.dml" moving load pattern database file.

8. Once a new load pattern is added to the moving load pattern database, the LRFR moving load analysis and rating described in Appendix O can be performed using the new load pattern.

9. Moving load patterns can be removed from the database by selecting a load pattern (as in step 5) and selecting the **(-) Button** located next to the (+) Button of step 4. Select **Save Database** to save changes to database.

This concludes the instructions to enter a custom moving load pattern to the moving load pattern database.

E.2 Example: Adding New Vehicle Configuration

This example describes how a custom moving load pattern is entered for a three-axle dump truck. The dump truck has the following information (obtained from weigh station):

- Gross Vehicle Weight (GVW): 53.16 kips
- Drive axle (1): Width 6.8 feet, Load 15.4 kips
- Axle (2): Width 7.2 feet, Load 18.88 kips
- Axle (3): Width 7.2 feet, Load 18.88 kips
- Longitudinal Axle Spacing: (1-2) 15.1 feet, (2-3) 4.4 feet

Once LARSA 2000 Plus and "PERRINE BRIDGE MASTER COPY" is open, select the **Edit Database Button**. This is illustrated in Figure E - 2.

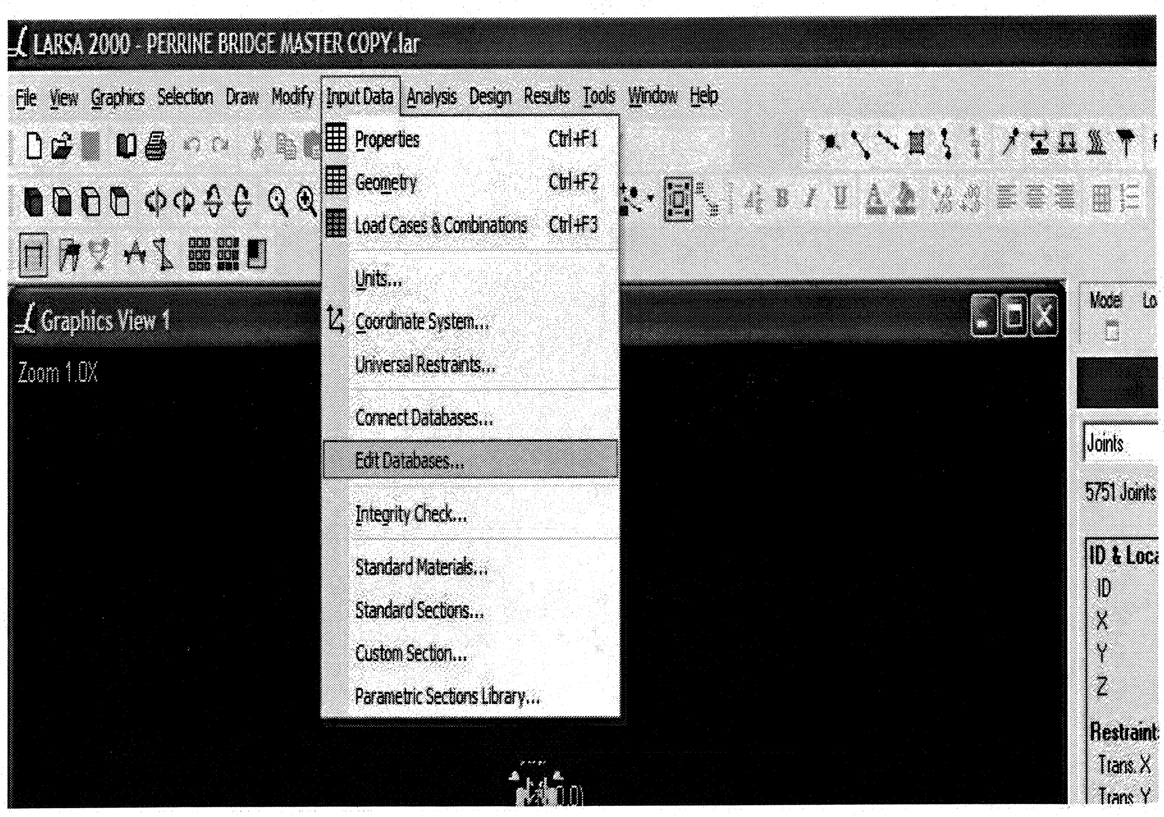


Figure E - 2: Select Edit Databases from Input Data Menu

Next, select the **(+) Button** to add a new vehicle pattern. Figure E - 3 shows where this button is located in the Database Editor screen.

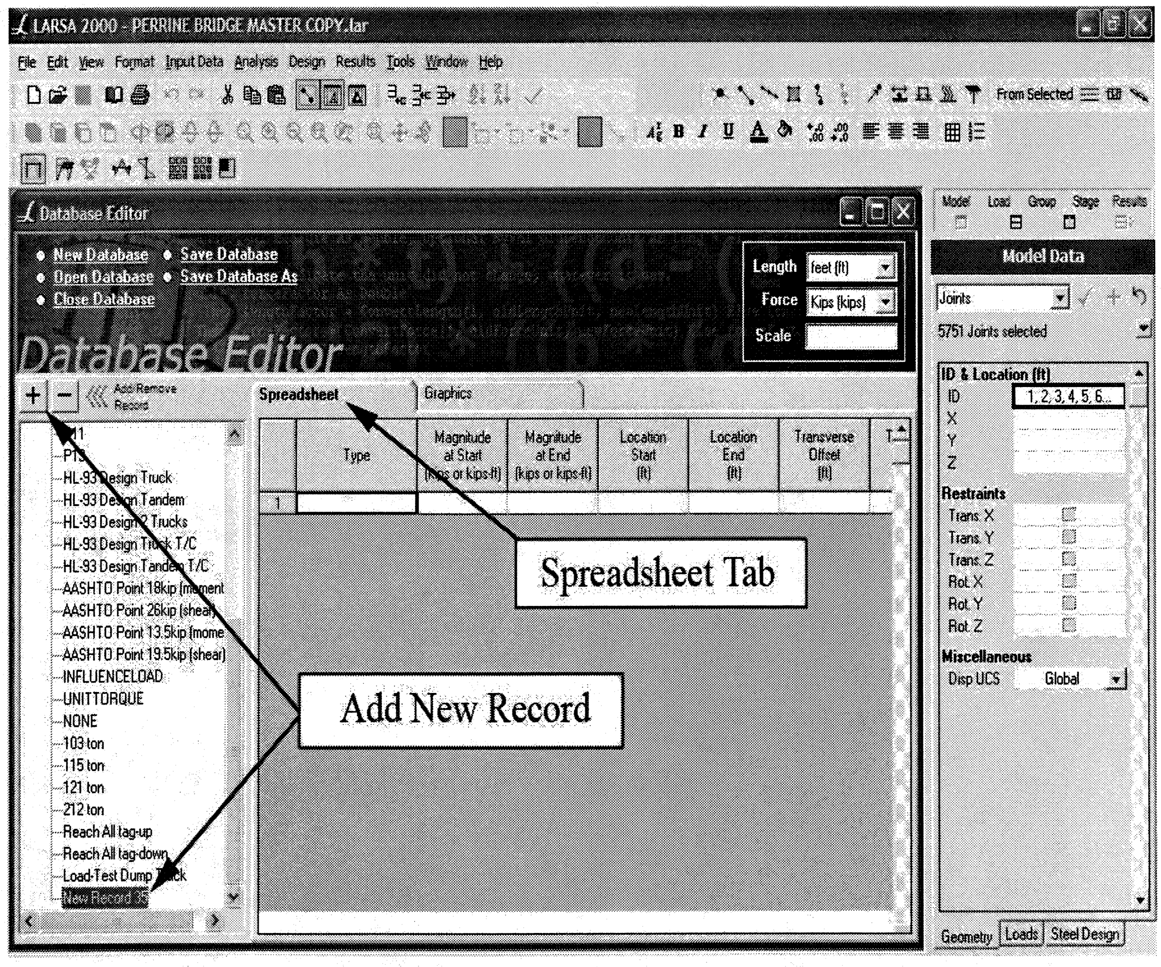


Figure E - 3: Add New Record to Database

The new load pattern is renamed to "Load-Test Dump Truck" by left-clicking the "new record 35" cell. Load information is now entered to create the new load pattern. Figure E - 4 shows the appropriate spreadsheet for the "Load-Test Dump Truck" geometry.

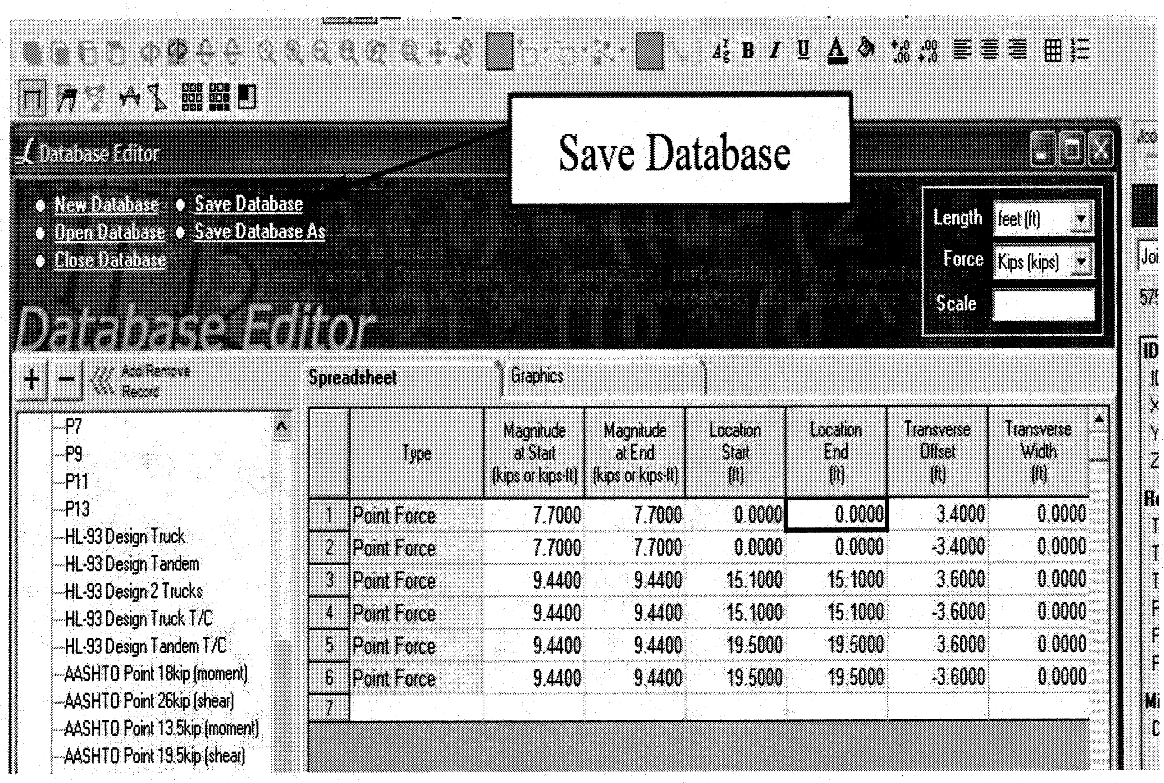


Figure E - 4: Input Moving Load Pattern

All contact points are entered as **POINT FORCE**. Note that axle weights are divided by two and entered on two rows, one row for each contact point on either side of the axle. Also note that the transverse offset is (+/-) axle width/2 for the two rows corresponding to the same axle. The load pattern can be visually checked for the appropriate geometry by selecting the graphics tab, located next to the spreadsheet tab. Figure E - 5 displays the graphics view for the "Load-Test Dump Truck." Downward arrows indicate load locations and corresponding forces.

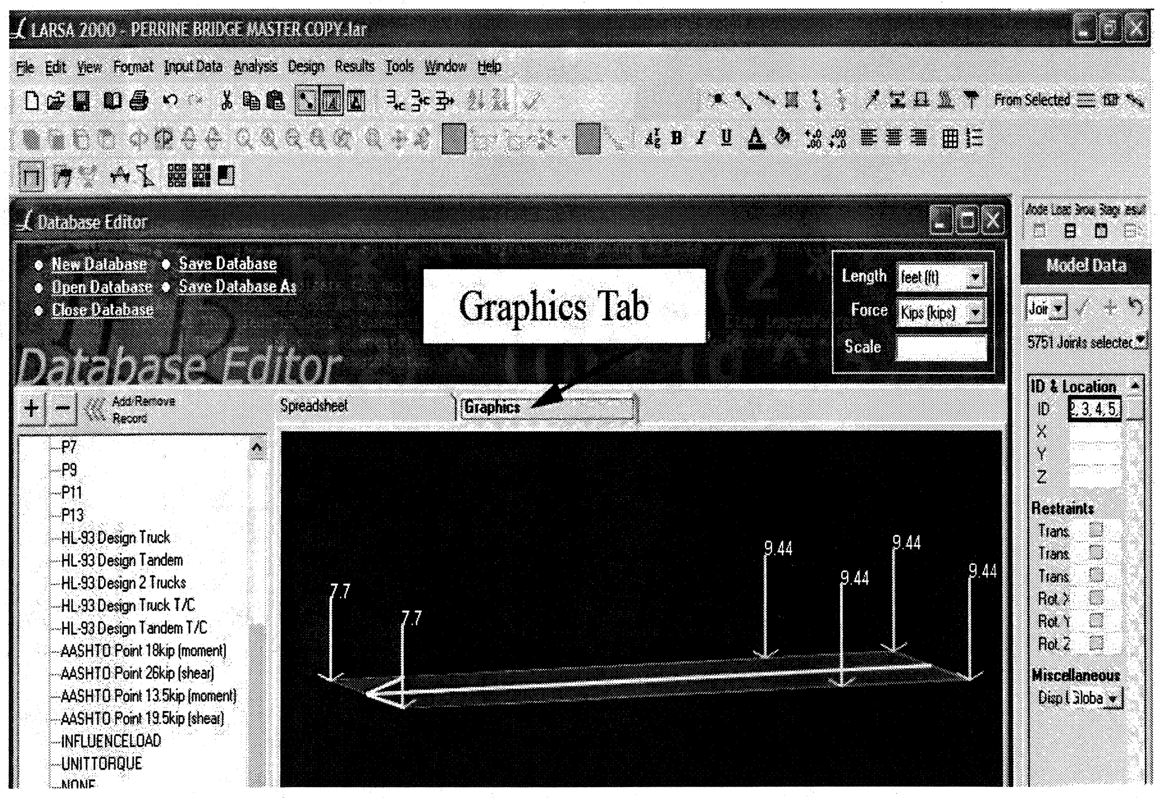


Figure E - 5: Graphics View Tab

Once the new "Load-Test Dump Truck" pattern is entered for all contact point in the configuration, click the **Save Database Button** (See Figure E - 4). At this point the new vehicle configuration is entered. Once the database is saved, follow the instructions in Appendix O for a typical LRFR analysis.

Appendix F. PRISMATIC MEMBER CAPACITY SUMMARY

Table F - 1: Stringer & Prismatic Plate Girder Capacity Summary

Sections	**Compression P_r (kip)**	**Tensile P_r (kip)**	**(+) Bend M_n (kip-ft)**	**(-) Bend M_n (kip-ft)**	**Interior Shear V_n (kip)**	**End Shear V_n (kip)**
Stringer						
W21x55 stringer	-	-	1140.0	475.84	-	214.85
W18x46(AISC)	-	-	889.9	330.87	-	175.91
W33x118(AISC)	-	-	3104.6	1168.83	-	500.51
W33x130(AISC)	-	-	3388.3	1310.54	-	527.81
South Approach Girder						
Exterior Plate Girder:						
south g. 1	-	-	8610.3	4942.6	547.88	425.38
south g. 2	-	-	14601.1	8318.0	547.88	425.38
south g. 3	-	-	14452.2	8448.7	547.88	425.38
south g. 4	-	-	19154.7	10732.8	423.60	241.32
south g. 5	-	-	19414.2	10483.9	511.61	361.57
south g. 6	-	-	15532.6	8767.3	511.61	361.57
Interior Plate Girder:						
south g. 1	-	-	8610.3	4951.3	547.88	425.38
south g. 2	-	-	14601.1	8329.0	547.88	425.38
south g. 3	-	-	14452.2	8457.2	547.88	425.38
south g. 4	-	-	19154.7	10744.6	423.60	241.32
south g. 5	-	-	19414.2	10499.9	511.61	361.57
south g. 6	-	-	15532.6	8779.0	511.61	361.57
North Approach Girder						
Exterior Plate Girder:						
N. ap. g. 1	-	-	7764.8	3767.1	500.50	407.30
N. ap. g. 2	-	-	10575.9	6018.7	460.10	339.00
N. ap. g. 3	-	-	10479.6	5835.0	460.10	339.00
Interior Plate Girder:						
N. ap. g. 1	-	-	7758.5	3778.7	500.50	407.30
N. ap. g. 2	-	-	10558.2	6030.9	460.10	339.00
N. ap. g. 3	-	-	10462.4	5850.5	460.10	339.00
Arch Girder Sections						
S str. h. arch 6	-	-	3139.5	1178.3	-	224.10
S str. h. arch' 6	-	-	2638.3	908.1	-	224.10
W33x118(AISC)	-	-	3104.6	636.8	-	500.51
W33x130(AISC)	-	-	3388.3	745.9	-	527.81
north str. hnch 6 o	-	-	3139.5	1170.2	-	224.61
north str. hnch 6 i	-	-	2638.3	900.2	-	221.60

Table F - 2: Arch Chord & Post Capacity Summary

Sections	**Compression P_r (kip)**	**Tensile P_r (kip)**	**(+) Bend M_n (kip-ft)**	**(-) Bend M_n (kip-ft)**	**Interior Shear V_n (kip)**	**End Shear V_n (kip)**
Upper Arch Chords:						
U_o-U_2	4456.10	-	-	-	-	-
U_2-U_4	5232.40	-	-	-	-	-
U_4-U_6	5752.73	-	-	-	-	-
U_6-U_8	6401.33	-	-	-	-	-
U_8-U_{10}	6920.63	-	-	-	-	-
U_{10}-U_{12}	7289.74	-	-	-	-	-
U_{12}-U_{14}	7360.62	-	-	-	-	-
U_{14}-U_{16}	7115.36	-	-	-	-	-
U_{16}-U_{18}	6853.19	-	-	-	-	-
U_{18}-U_{19}	6862.84	-	-	-	-	-
Lower Arch Chords:						
L_o-L_2	12236.98	-	-	-	-	-
L_2-L_4	11061.00	-	-	-	-	-
L_4-L_6	10187.21	-	-	-	-	-
L_6-L_8	9234.48	-	-	-	-	-
L_8-L_{10}	8533.02	-	-	-	-	-
L_{10}-L_{12}	7912.60	-	-	-	-	-
L_{12}-L_{14}	7522.74	-	-	-	-	-
L_{14}-L_{16}	7198.95	-	-	-	-	-
L_{16}-L_{18}	6763.86	-	-	-	-	-
L_{18}-L_{19}	6772.80	-	-	-	-	-
Arch Posts:						
U_o - L_o	3818.16	-	-	-	-	-
U_1-L_1	1094.35	-	-	-	-	-
U_2-L_2	1692.47	-	-	-	-	-
U_3-L_3	847.43	-	-	-	-	-
U_4-L_4	1447.06	-	-	-	-	-
U_5-L_5	606.70	-	-	-	-	-
U_6-L_6	1134.83	-	-	-	-	-
U_7-L_7	705.13	-	-	-	-	-
U_8-L_8	1225.79	-	-	-	-	-
U_9-L_9	736.62	-	-	-	-	-
U_{10}-L_{10}	831.00	-	-	-	-	-
U_{11}-L_{11}	597.67	-	-	-	-	-
U_{12}-L_{12}	895.23	-	-	-	-	-
U_{13}-L_{13}	537.09	-	-	-	-	-
U_{14}-L_{14}	943.34	-	-	-	-	-
U_{15}-L_{15}	568.53	-	-	-	-	-
U_{16}-L_{16}	974.64	-	-	-	-	-
U_{17}-L_{17}	587.27	-	-	-	-	-
U_{18}-L_{18}	707.93	-	-	-	-	-
U_{19}-L_{19}	364.79	-	-	-	-	-

Table F - 3: Arch Diagonal, Chevron Brace, & Cross Brace Capacity Summary

Sections	Compression P_r (kip)	Tensile P_r (kip)	(+) Bend M_n (kip-ft)	(-) Bend M_n (kip-ft)	Interior Shear V_n (kip)	End Shear V_n (kip)
Diagonal Sections:						
U_o-L_1	-	943.99	-	-	-	-
U_1-L_2	-	943.99	-	-	-	-
U_2-L_3	-	796.42	-	-	-	-
U_3-L_4	-	796.42	-	-	-	-
U_4-L_5	-	796.42	-	-	-	-
U_5-L_6	-	796.42	-	-	-	-
U_6-U_7	-	796.42	-	-	-	-
U_7-L_8	-	796.42	-	-	-	-
U_8-L_9	-	796.42	-	-	-	-
U_9-L_{10}	-	796.42	-	-	-	-
U_{10}-L_{11}	-	796.42	-	-	-	-
U_{11}-L_{12}	-	858.00	-	-	-	-
U_{12}-L_{13}	-	858.00	-	-	-	-
U_{13}-L_{14}	-	1022.81	-	-	-	-
U_{14}-L_{15}	-	1022.81	-	-	-	-
U_{15}-L_{16}	-	1022.81	-	-	-	-
U_{16}-U_{17}	-	1022.81	-	-	-	-
U_{17}-L_{18}	-	1022.81	-	-	-	-
U_{18}-L_{19}	-	1245.04	-	-	-	-
Arch Chevron Braces:						
U diagonal	203.93	664.56	-	-	-	-
L_1 diagonal	599.25	1318.67	-	-	-	-
L` diagonal	383.81	1001.39	-	-	-	-
Arch Cross Braces:						
2L8x4x7/16	51.17	358.45	-	-	-	-

Table F - 4: Arch Strut Capacity Summary

Sections	Compression P_r (kip)	Tensile P_r (kip)	Strong Bend M_{nxx} (kip-ft)	Weak Bend M_{nyy} (kip-ft)	Interior Shear V_n (kip)	End Shear V_n (kip)
Arch Struts:						
U Strut	340.69	612.23	445.60	60.83	-	203.91
L Strut	366.55	784.93	601.22	60.96	-	320.81

Table F - 5: Floor Beam, Column, & Bent Capacity Summary

Sections	Compression P_r (kip)	Tensile P_r (kip)	(+) Bend M_n (kip-ft)	(-) Bend M_n (kip-ft)	Interior Shear V_n (kip)	End Shear V_n (kip)
Approach Floor Beams						
W30x124(AISC)	-	-	1679.79	1357.3	-	449.57
W24x76(AISC)	-	-	825.23	825.23	-	264.77
W10x 72	-	-	373.77	373.77	-	131.39
W10x49(AISC)	-	-	247.60	247.60	-	73.95
W30x116(AISC)	-	-	1555.56	1607.1	-	434.20
W24x68(AISC)	-	-	725.79	725.79	-	249.73
W10x68(AISC)	-	-	351.41	351.41	-	102.23
W10x49(AISC)	-	-	247.60	247.60	-	73.95
Arch Floor Beams						
Arch Floor Beam 1	-	-	5225.65	2261.6	477.70	214.54
Arch Floor Beam 2	-	-	2756.64	2757.2	369.92	366.37
Arch Floor Beam 3	-	-	4430.93	4423.1	391.86	266.24
Spandrel Columns						
C_o	2609.00	-	-	-	-	-
C_2	2851.14	-	-	-	-	-
C_4	1996.42	-	-	-	-	-
C_6	1666.26	-	-	-	-	-
C_8	1918.85	-	-	-	-	-
C_{10}	2357.46	-	-	-	-	-
C_{12}	2638.66	-	-	-	-	-
C_{14}	2785.31	-	-	-	-	-
C_{16}	3376.82	-	-	-	-	-
C_{18}	3399.18	-	-	-	-	-
Spandrel Column Lateral Braces:						
HP14x73 w/ .5PL	441.73	940.25	-	-	-	-
L_o lower strut	594.26	1161.4	-	-	-	-
HP14x73(AISC) diagonal	287.86	730.59	-	-	-	-
HP14x73(AISC) strut	255.66	817.12	-	-	-	-
South Bent						
South Bent Column	5321.59	-	-	-	-	-
WT9x23(AISC)	28.20	259.18	-	-	-	-
WT12x65.5(AISC)	185.65	723.14	-	-	-	-
North Bent						
North Bent Column	3565.07	-	-	-	-	-
HP14x73(AISC)	380.28	743.70	-	-	-	-
HP14x73 w/ .5PL	1066.08	1540.9	-	-	-	-

Table F - 6: Deck Bracing Capacity Summary

Sections	Compression P_r (kip)	Tensile P_r (kip)	(+) Bend M_n (kip-ft)	(-) Bend M_n (kip-ft)	Interior Shear V_n (kip)	End Shear V_n (kip)
South Approach Braces						
Lateral Bracing:						
L6x4x5/16(AISC)	15.31	116.30	-	-	-	-
South Bent Diaphragm:						
2L7x4x3/8(AISC)	28.02	277.12	-	-	-	-
2L7x4x7/16	160.83	320.97	-	-	-	-
2L3.5x2.5x5/16	22.17	192.45	-	-	-	-
WT4x17.5(AISC)	62.71	155.35	-	-	-	-
South Abutment Diaphragm:						
MC12x35(AISC)	295.75	392.61	-	-	-	-
South Diaphragm At F_o:						
W12x53(AISC)	697.80	531.12	-	-	-	-
North Approach Braces						
North Bent Diaphragm:						
2L7x4x3/8(AISC)	28.02	277.12	-	-	-	-
2L7x4x7/16	191.77	320.97	-	-	-	-
2L3.5x2.5x5/16	22.17	192.45	-	-	-	-
WT4x17.5(AISC)	84.15	155.35	-	-	-	-
North Abutment Diaphragm:						
MC12x35(AISC)	295.75	392.61	-	-	-	-
North Diaphragm At F'_o:						
W12x53(AISC)	697.80	531.12	-	-	-	-
Arch Braces						
Lateral Bracing:						
WT12x65.5(AISC)	60.90	599.36	-	-	-	-
Floor Beam Diaphragm:						
5/16" Bent Plate	169.06	217.05	-	-	-	-
Diaphragm A:						
W12x53(AISC)	697.83	537.64	-	-	-	-
End Diaphragm:						
W12x40(AISC)	457.57	453.19	-	-	-	-
L6x3½x3/8(AISC)	71.33	114.46	-	-	-	-
L3x2½x3/8(AISC)	23.15	66.27	-	-	-	-
Traction Bracing:						
L3x2½x3/8(AISC)	45.69	66.27	-	-	-	-
L3½x3x3/8(AISC)	49.16	82.33	-	-	-	-

Appendix G. NON-PRISMATIC MEMBER CAPACITY SUMMARY

Table G - 1: North & South Approach Haunch Member Capacity Summary

Sections	Compression P_r (kip)	Tensile P_r (kip)	(+) Bend M_n (kip-ft)	(-) Bend M_n (kip-ft)	Interior Shear V_n (kip)	End Shear V_n (kip)
South Approach Haunch						
Exterior Girder						
South ap. gird 1	-	-	12652.57	8823.67	586.69	-
South ap. gird 2	-	-	13305.97	10504.96	650.62	-
South ap. gird 3	-	-	19179.28	14848.57	841.30	-
South ap. gird 4	-	-	25010.79	17939.93	1091.93	-
South ap. gird 5	-	-	29396.34	20077.45	1133.00	-
Interior Girder						
South ap. gird 1	-	-	12652.57	8823.67	586.69	-
South ap. gird 2	-	-	13305.97	10504.96	650.62	-
South ap. gird 3	-	-	19179.28	14848.57	841.30	-
South ap. gird 4	-	-	25010.79	17939.93	1091.93	-
South ap. gird 5	-	-	29396.34	20077.45	1133.00	-
North Approach Haunch						
Exterior Girder						
N. h. ap. g. 1	-	-	10012.42	5749.51	516.13	-
N. h. ap. g. 2	-	-	10351.90	9133.42	681.71	-
N. h. ap. g. 3	-	-	14498.33	10781.76	923.09	-
N. h. ap. g. 4	-	-	17031.73	9239.85	997.70	-
Interior Girder						
N. h. ap. g. 1	-	-	9993.86	5749.45	516.13	-
N. h. ap. g. 2	-	-	10348.21	9133.32	681.71	-
N. h. ap. g. 3	-	-	14493.10	10781.62	923.09	-
N. h. ap. g. 4	-	-	17025.38	9239.71	997.70	-

Table G - 2: Main Arch Span Haunch Member Capacity Summary

Sections	Compression P_r (kip)	Tensile P_r (kip)	(+) Bend M_n (kip-ft)	(-) Bend M_n (kip-ft)	Interior Shear V_n (kip)	End Shear V_n (kip)
Arch Haunch						
Arch Exterior Girder						
S str. h. arch 1	-	-	4681.23	1898.87	582.87	-
S str. h. arch 2	-	-	5389.96	1723.03	494.72	-
S str. h. arch 3	-	-	4068.88	1368.55	394.90	-
S str. h. arch 4	-	-	3364.06	1127.28	339.24	-
S. str. h. arch 5	-	-	2992.95	994.55	311.53	-
S str. h. arch 6	-	-	2936.37	973.80	305.93	-
north str. hnch 6 o	-	-	3130.11	1044.27	319.22	-
north str. hnch 5 o	-	-	3179.40	849.06	321.53	-
north str. hnch 4 o	-	-	3488.74	1170.61	343.51	-
north str. hnch 3 o	-	-	4069.77	1368.57	387.20	-
north str. hnch 2 o	-	-	5142.19	1662.26	441.26	-
north str. hnch 1 o	-	-	5955.68	1825.53	498.26	-
Arch Interior Girder						
S str. h. arch' 1	-	-	4001.35	1342.27	587.68	-
S. str. h. arch' 2	-	-	4698.20	1242.95	509.27	-
S. str. h. arch' 3	-	-	3510.53	1001.08	408.85	-
S. str. h. arch' 4	-	-	2891.01	846.69	347.43	-
S. str. h. arch' 5	-	-	2564.79	756.68	319.04	-
S str. h. arch' 6	-	-	2513.88	740.94	313.19	-
north str. hnch 6 i	-	-	2643.42	806.15	322.97	-
north str. hnch 5 i	-	-	2684.53	817.08	325.05	-
north str. hnch 4 i	-	-	2955.53	886.86	349.23	-
north str. hnch 3 i	-	-	3465.44	1007.94	379.56	-
north str. hnch 2 i	-	-	4415.72	1199.71	446.99	-
north str. hnch 1 i	-	-	5142.31	1297.51	498.26	-

Appendix H. TENSILE CAPACITY CALCULATION

Table H - 1: Tension Capacity Calculation (Part 1)

Tension Design Capacity (Article 6.8) AASHTO LRFD Bridge Design Specifications 2005 Interim Revisions

F_y A588 (ksi):	50.00
Fu A588 (ksi):	70.00

Ultimate tensile strengths were taken from Table 2.1.1, pg. 44, of Steel Structures Design and Behavior 4th ed., Charles G. Salmon and John E. Johnson

Selected Arch Diagonals:

Dimensions — 6.8.2.2

sections	Flange (in)	F thickness (in)	Web (in)	W thickness (in)	Total Area (in^2)	φ_y -	φ_u -	U -	φ_{bolt} + 1/8 in (in)	# bolts in line -	# bolt staggered -
Uo-L1	12.00	0.63	22.75	0.50	26.38	0.95	0.80	0.85	1.00	2.00	2.00
U2-L3	12.00	0.50	23.00	0.44	22.06	0.95	0.80	0.85	1.00	2.00	2.00
U12-L13	12.00	0.50	23.00	0.50	23.50	0.95	0.80	0.85	1.00	2.00	2.00
U13-L14	14.00	0.63	22.75	0.50	28.88	0.95	0.80	0.85	1.00	4.00	0.00
U18-L19	14.00	0.75	22.50	0.63	35.06	0.95	0.80	0.85	1.00	4.00	0.00

Arch Floor Beam Diaphrams: — 6.8.2.2

Name	Depth (in)	F Width (in)	F Thick (in)	Web Thick (in)	Sect. Area (in^2)	φ_y -	φ_u -	U -	φ_{bolt} + 1/8 in (in)	# bolts in line -	# bolt staggered -
5/16" Bent Plate	15.00	4.00	0.31	0.31	6.99	0.95	0.80	0.75	1.00	4.00	0.00

South Diaphram At Fo: — 6.8.2.2

Name	Depth (in)	F Width (in)	F Thick (in)	Web Thick (in)	Sect. Area (in^2)	φ_y -	φ_u -	U -	φ_{bolt} + 1/8 in (in)	# bolts in line -	# bolt staggered -
W12x53(AISC)	12.06	10.00	0.58	0.35	15.60	0.95	0.80	0.75	1.00	3.00	2.00

End Diaphram: — 6.8.2.2

Name	Depth (in)	F Width (in)	F Thick (in)	Web Thick (in)	Sect. Area (in^2)	φ_y -	φ_u -	U -	φ_{bolt} + 1/8 in (in)	# bolts in line -	# bolt staggered -
W12x40(AISC)	11.94	8.01	0.52	0.30	11.80	0.95	0.80	0.85	1.00	3.00	0.00
L6x3½x3/8(AISC)	6.00	3.50	0.38		3.42	0.95	0.80	0.85	1.00	2.00	0.00
L3x2½x3/8(AISC)	3.00	2.50	0.38		1.92	0.95	0.80	0.85	1.00	1.00	0.00

Table H - 2: Tension Capacity Calculation (Part 2)

space, s (in)	gage, g (in)	# flnge bolted -	net flnge width (in)	A_n (in^2)	A_e (in^2)	6.8.2.1-1 P_r (kip)	6.8.2.1-2 P_r (kip)	unbraced length (in)	r_{min} (in)	6.8.4 Limit Slenderness Ratio L/r_{min}	< 200	$P_{r\,tensile}$ (kip)
1.63	2.50	2.00	8.53	22.04	18.73	1252.81	943.99	388.32	2.61	148.55	200.00	1048.87
1.63	2.50	2.00	8.53	18.59	15.80	1047.97	796.42	376.68	2.56	147.36	200.00	884.91
1.63	2.50	2.00	8.53	20.03	17.02	1116.25	858.00	366.00	2.48	147.73	200.00	953.34
1.63	3.50	2.00	10.00	23.88	20.29	1371.56	1022.81	361.92	3.15	114.98	200.00	1136.45
1.63	3.50	2.00	10.00	29.06	24.70	1665.47	1245.04	395.28	3.13	126.30	200.00	1383.38

space, s (in)	gage, g (in)	# flnge bolted -	net flnge width (in)	A_n (in^2)	A_e (in^2)	6.8.2.1-1 P_r (kip)	6.8.2.1-2 P_r (kip)	unbraced length (in)	r_{min} (in)	6.8.4 Limit Slenderness Ratio L/r_{min}	< 240	$P_{r\,tensile}$ (kip)
-	-	1.00	10.38	5.74	4.31	332.13	241.17	97.00	1.12	86.31	240.00	241.17

space, s (in)	gage, g (in)	# flnge bolted -	net flnge width (in)	A_n (in^2)	A_e (in^2)	6.8.2.1-1 P_r (kip)	6.8.2.1-2 P_r (kip)	unbraced length (in)	r_{min} (in)	6.8.4 Limit Slenderness Ratio L/r_{min}	< 240	$P_{r\,tensile}$ (kip)
3.00	1.50	1.00	7.41	14.05	10.54	741.00	590.13	97.02	2.48	39.15	240.00	597.37

space, s (in)	gage, g (in)	# flnge bolted -	net flnge width (in)	A_n (in^2)	A_e (in^2)	6.8.2.1-1 P_r (kip)	6.8.2.1-2 P_r (kip)	unbraced length (in)	r_{min} (in)	6.8.4 Limit Slenderness Ratio L/r_{min}	< 240	$P_{r\,tensile}$ (kip)
-	-	1.00	7.91	10.58	8.99	560.50	503.54	97.00	1.93	50.17	240.00	503.54
-	-	1.00	3.63	2.67	2.27	162.45	127.18	97.00	0.99	98.15	240.00	127.18
-	-	1.00	1.63	1.55	1.31	91.20	73.63	106.32	0.74	144.46	240.00	73.63

Appendix I. COMPRESSION CAPACITY CALCULATION

Table I - 1: Compression Capacity Calculation (Part 1)

Compression Design Capacity (Article 6.9) AASHTO LRFD Bridge Design Specifications 2005 Interim Revisions

F_y A514 (ksi):	100	F_y A588 (ksi):	50

Selected Arch Upper Chords:

sections	Dimensions: Base (in)	B thickness (in)	Hieght (in)	H thickness (in)	Total Area (in^2)	φ_c -	unbraced L (in)	r_x (in)	r_y (in)	C.4.6.2.5-1 K -
U_0-U_2*	18.00	0.75	23.00	0.88	67.25	0.90	388.20	9.11	7.40	0.88
U_2-U_4*	18.00	0.88	23.00	1.00	77.50	0.90	373.20	9.17	7.34	0.88
U_4-U_6*	18.00	0.88	23.00	1.13	83.25	0.90	359.88	9.02	7.38	0.88
U_6-U_8*	18.00	1.13	23.00	1.13	92.25	0.90	348.12	9.42	7.20	0.88

Selected Arch Posts:

sections	Dimensions: Flange (in)	F thickness (in)	Web (in)	W thickness (in)	Total Area (in^2)	φ_c -	unbraced L (in)	r_x (in)	r_y (in)	C.4.6.2.5-1 K -
U_1-L_1	18.00	0.75	22.50	0.75	43.88	0.90	456.00	9.97	4.08	0.88
U_2-L_2	18.00	1.13	21.75	0.88	59.53	0.90	432.00	10.08	4.29	0.88
U_3-L_3	16.00	0.75	22.50	0.63	38.06	0.90	410.76	10.04	3.67	0.88
U_4-L_4	18.00	0.88	22.25	0.63	45.41	0.90	389.52	10.27	4.33	0.88
U_5-L_5	14.00	0.63	22.75	0.63	31.72	0.90	370.92	9.73	3.00	0.88
U_6-L_6	17.00	0.75	22.50	0.63	39.56	0.90	352.32	10.11	3.94	0.88

Selected Spandrel Columns:

Shape	Dimensions: Base (in)	B thickness (in)	Hieght (in)	H thickness (in)	Total Area (in^2)	φ_c -	unbraced L (in)	r_x (in)	r_y (in)	C.4.6.2.5-1 K_x -
C_2*	38.25	0.88	38.75	0.88	134.75	0.90	1917.58	16.06	15.37	0.80
C_4*	30.50	0.75	31.00	0.75	92.25	0.90	1511.30	12.86	12.25	0.80
C_6*	24.75	0.63	25.25	0.63	62.50	0.90	1155.67	10.47	9.94	0.80
C_8*	22.75	0.63	23.25	0.63	57.50	0.90	850.54	9.66	9.12	0.80

* hieght of box excludes the width of the base on the top and bottom of the box

Table I - 2: Compression Capacity Calculation (Part 2)

6.9.3 Limit Slenderness Ratio $(Kl_x)/r_x$	< 120	6.9.3 Limit Slenderness Ratio $(Kl_y)/r_y$	< 120	6.9.4.1-3 λ_x	6.9.4.1-3 λ_y	6.9.4.1 $\lambda < 2.25$ yes	no	Axis of Control	6.9.4.1-1 P_n (kip)	6.9.4.1-2 P_n (kip)	6.9.4.2-1 k plates
				-	-			-			
37.28	120.00	45.90	120.00	0.49	0.74	0.74	2.25	y	4951.23	-	1.49
35.61	120.00	44.48	120.00	0.44	0.69	0.69	2.25	y	5813.78	-	1.49
34.91	120.00	42.64	120.00	0.43	0.64	0.64	2.25	y	6391.92	-	1.49
32.35	120.00	42.30	120.00	0.37	0.63	0.63	2.25	y	7112.58	-	1.49

6.9.3 Limit Slenderness Ratio $(Kl_x)/r_x$	< 120	6.9.3 Limit Slenderness Ratio $(Kl_y)/r_y$	< 120	6.9.4.1-3 λ_x	6.9.4.1-3 λ_y	6.9.4.1 $\lambda < 2.25$ yes	no	Axis of Control	6.9.4.1-1 P_n (kip)	6.9.4.1-2 P_n (kip)	6.9.4.2-1 k Web
				-	-			-			
40.02	120.00	97.83	120.00	0.28	1.67	1.67	2.25	y	1094.35	-	1.49
37.49	120.00	88.15	120.00	0.25	1.36	1.36	2.25	y	1692.47	-	1.49
35.79	120.00	97.95	120.00	0.22	1.68	1.68	2.25	y	947.75	-	1.49
33.19	120.00	78.73	120.00	0.19	1.08	1.08	2.25	y	1447.06	-	1.49
33.35	120.00	108.03	120.00	0.19	2.04	2.04	2.25	y	679.24	-	1.49
30.50	120.00	78.22	120.00	0.16	1.07	1.07	2.25	y	1268.24	-	1.49

C.4.6.2.5-1 K_y	6.9.3 Limit Slenderness Ratio $(Kl_x)/r_x$	< 120	6.9.3 Limit Slenderness Ratio $(Kl_y)/r_y$	< 120	6.9.4.1-3 λ_x	6.9.4.1-3 λ_y	6.9.4.1 $\lambda < 2.25$ yes	no	Axis of Control	6.9.4.1-1 P_n (kip)	6.9.4.1-2 P_n (kip)
-					-	-			-		
0.80	95.51	120.00	99.79	120.00	1.60	1.74	1.74	2.25	y	3268.01	-
0.80	94.02	120.00	98.72	120.00	1.55	1.70	1.70	2.25	y	2272.04	-
0.80	88.27	120.00	93.03	120.00	1.36	1.51	1.51	2.25	y	1666.26	-
0.80	70.46	120.00	74.60	120.00	0.87	0.97	0.97	2.25	y	1918.85	-

Table I - 3: Compression Capacity Calculation (Part 3)

Height Side 6.9.4.2-6 width/ thick ratio $b/t < 1.7 (E/F_y)^{0.5}$		Base Side 6.9.4.2-6 width/ thick ratio $b/t < 1.7 (E/F_y)^{0.5}$		AISC App. B5-(d) width/ thick ratio $1.40 (E/f)^{0.5}$	AISC App. B5-11 b_e (in)	effective area A_e (in^2)	AISC App. B5-14 Q_a -	AISC App. B5-7 Q_s -
24.00	28.95	26.29	28.95	-	-	-	-	-
20.57	28.95	23.00	28.95	-	-	-	-	-
20.57	28.95	20.44	28.95	-	-	-	-	-
16.00	28.95	20.44	28.95	-	-	-	-	-

Web 6.9.4.2 width/ thick ratio $b/t < k (E/F_y)^{0.5}$		Flanges 6.9.4.2-4 k_c -	6.9.4.2 width/ thick ratio $b/t < 0.64 (k_c * E/F_y)^{0.5}$		AISC 16.1-90 width/ thick ratio $1.17(k_c * E/F_y)^{0.5}$	AISC App. B5-(d) width/ thick ratio $1.49 (E/f)^{0.5}$	AISC App. B5-11 b_e (in)	effective area A_e (in^2)
30.00	35.88	0.73	12.00	13.17	24.08	-	-	-
24.86	35.88	0.80	8.00	13.81	25.24	-	-	-
36.00	35.88	0.67	10.67	12.58	23.01	35.88	22.21	37.88
35.60	35.88	0.67	10.29	12.62	23.07	-	-	-
36.40	35.88	0.66	11.20	12.55	22.94	35.88	22.28	31.43
36.00	35.88	0.67	11.33	12.58	23.01	35.88	22.21	39.38

6.9.4.2-1 k -	Height Side 6.9.4.2-6 width/ thick ratio $b/t < 1.7 (E/F_y)^{0.5}$		Base Side 6.9.4.2-6 width/ thick ratio $b/t < 1.7 (E/F_y)^{0.5}$		Slender Column AISC App. B5-(d) width/ thick ratio $1.40 (E/f)^{0.5}$	AISC App. B5-11 b_e base (in)	AISC App. B5-11 b_e height (in)	effective area A_e (in^2)
1.49	44.29	40.94	43.71	40.94	33.72	-	31.93	123.69
1.49	41.33	40.94	40.67	40.94	33.72	-	26.86	86.79
1.49	40.40	40.94	39.60	40.94	-	-	-	-
1.49	37.20	40.94	36.40	40.94	-	-	-	-

Table I - 4: Compression Capacity Calculation (Part 4)

Q $Q = Q_a * Q_s$	λ_c $\lambda_c = (Kl/r_{min}\,\pi)*(F_y/E)^{0.5}$	AISC App. B5.3d axial compression loading $\lambda_c(Q^{0.5})$ < 1.5		AISC App. 5-15 F_{cr} (ksi)	6.9.2.1-1 P_{rc} (kip)
-	-	-	-	-	4456.10
-	-	-	-	-	5232.40
-	-	-	-	-	5752.73
-	-	-	-	-	6401.33

AISC 16.1-91 Q_a -	AISC App. B5-7 Q_s -	Q $Q = Q_a * Q_s$	λ_c $\lambda_c = (Kl/r_{min},\pi)*(F_y/E)^{0.5}$	AISC App. B5.3d axial compression loading $\lambda_c(Q^{0.5})$ < 1.5		AISC App. 5-15 F_{cr} (ksi)	6.9.2.1-1 P_{rc} (kip)
-	-	-	-	-	-	-	1094.35
-	-	-	-	-	-	-	1692.47
1.00	1.00	1.00	1.30	1.29	1.50	24.74	847.43
-	-	-	-	-	-	-	1447.06
0.99	1.00	0.99	1.43	1.42	1.50	21.25	606.70
1.00	1.00	1.00	1.03	1.03	1.50	31.87	1134.83

AISC 16.1-93 Q_a -	AISC App. B5-3 Q_s -	Q $Q = Q_a * Q_s$	λ_c $\lambda_c = (Kl/r_{min}\,\pi)*(F_y/E)^{0.5}$	AISC App. B5.3d axial compression loading $\lambda_c(Q^{0.5})$ < 1.5		AISC App. 5-15 F_{cr} (ksi)	6.9.2.1-1 P_{rc} (kip)
0.92	1.00	0.92	1.32	1.26	1.50	23.51	2851.14
0.94	1.00	0.94	1.31	1.27	1.50	24.05	1996.42
-	-	-	-	-	-	-	1666.26
-	-	-	-	-	-	-	1918.85

Appendix J. POSITIVE COMPOSITE BENDING MOMENT CAPACITY CALCULATION

Table J - 1: Composite Positive Bending Moment Design Capacity Calculation (Part 1)

Positive Flexure Design Capacity (Article 6.10.7) AASHTO LRFD Bridge Design Specifications 2005 Interim Revision

Note: Deck reinforcement is neglected in calculating positive flexural moment capacity.

Note: Haunch pad is the distance from the top of the top flange to the bottom surface of the slab.

f'_c (ksi): 4.0 **F_y rebar (ksi):** 60.0

F_y A588 (ksi): 50.0 **Top of slab to top flange (in):** 10.0

Plate Girder Sections:

Prismatic Plate Girder Sections

Sections	*I Section Dimensions:* Top Flange Width (in)	Top Flange Thickness (in)	Bottom Flange Width (in)	Bottom Flange Thickness (in)	Web Height (in)	Web Thickness (in)	*Slab Dimensions:* Slab Thickness (in)	Effective Slab Width (in)	*Haunch:* Bottom Flange θ (deg)	Haunch Pad (in)
South Approach Interior Girder										
south g. 1	16.00	0.875	20.00	1.125	60.00	0.375	7.50	98.00		2.50
south g. 2	16.00	1.500	20.00	2.000	60.00	0.375	7.50	98.00		2.50
North Approach Interior Girder										
N. ap. g. 1	12.00	0.750	14.00	1.500	54.00	0.375	7.50	96.00		2.50
N. ap. g. 2	12.00	1.500	14.00	2.500	54.00	0.375	7.50	96.00		2.50
N. ap. g. 3	12.00	1.000	14.00	2.500	54.00	0.375	7.50	96.00		2.50
Arch Exterior Girder (Haunched)										
S str. h. arch 1	11.50	1.000	11.50	1.000	61.50	0.375	7.50	96.37	0.00	2.50
S str. h. arch 2	11.50	1.000	11.50	1.000	53.59	0.375	7.50	96.37	5.00	2.50
S str. h. arch 3	11.50	1.000	11.50	1.000	42.09	0.375	7.50	96.37	2.00	2.50
North Approach Interior Girder (Haunched)										
N. h. ap. g. 1	12.00	2.500	14.00	2.125	53.13	0.375	7.50	96.00	1.00	2.50
N. h. ap. g. 2	12.00	2.500	14.00	2.125	66.19	0.375	7.50	96.00	7.00	2.50

Rolled I-Shape Sections:

Arch Stringers

W33x118(AISC)	11.48	0.740	11.48	0.740	31.38	0.550	7.50	97.00	2.50
W33x130(AISC)	11.51	0.855	11.51	0.855	31.38	0.580	7.50	96.37	2.50

Table J - 2: Composite Positive Bending Moment Design Capacity Calculation (Part 2)

Calculate PNA and Plastic Moment Capacity

Section Areas:				*Section Forces:*						*Case I*			*Case II*		
A_{tf} (in^2)	A_{web} (in^2)	A_{bf} (in^2)	A_{slab} (in^2)	P_{slab} (kip)	P_{tf} (kip)	P_{web} (kip)	P_{bf} (kip)	P_c (kip)	P_t (kip)	$P_t + P_w$ > (kip)	$P_c + P_s$ (kip)	Is Case I True?	$P_c + P_w + P_t$ > (kip)	P_{slab} (kip)	Is Case II True?
14.0	22.5	22.5	735.0	2940.0	700.0	1125.0	1125.0	700.0	1125.0	2250.0	3640.0	No	2950.0	2940.0	Yes
24.0	22.5	40.0	735.0	2940.0	1200.0	1125.0	2000.0	1200.0	2000.0	3125.0	4140.0	No	4325.0	2940.0	Yes
9.0	20.3	21.0	720.0	2880.0	450.0	1012.5	1050.0	450.0	1050.0	2062.5	3330.0	No	2512.5	2880.0	No
18.0	20.3	35.0	720.0	2880.0	900.0	1012.5	1750.0	900.0	1750.0	2762.5	3780.0	No	3662.5	2880.0	Yes
12.0	20.3	35.0	720.0	2880.0	600.0	1012.5	1750.0	600.0	1750.0	2762.5	3480.0	No	3362.5	2880.0	Yes
11.5	23.1	11.5	722.8	2891.1	575.0	1153.1	575.0	575.0	575.0	1728.1	3466.1	No	2303.1	2891.1	No
11.5	20.1	11.5	722.8	2891.1	575.0	1004.9	572.8	575.0	572.8	1577.7	3466.1	No	2152.7	2891.1	No
11.5	15.8	11.5	722.8	2891.1	575.0	789.3	574.7	575.0	574.7	1363.9	3466.1	No	1938.9	2891.1	No
30.0	19.9	29.8	720.0	2880.0	1500.0	996.1	1487.3	1500.0	1487.3	2483.4	4380.0	No	3983.4	2880.0	Yes
30.0	24.8	29.8	720.0	2880.0	1500.0	1241.0	1476.4	1500.0	1476.4	2717.4	4380.0	No	4217.4	2880.0	Yes
8.5	17.3	8.5	727.5	2910.0	424.8	863.0	424.8	424.8	424.8	1287.7	3334.8	No	1712.5	2910.0	No
9.8	18.2	9.8	722.8	2891.1	492.1	910.0	492.1	492.1	492.1	1402.1	3383.2	No	1894.1	2891.1	No

Table J - 3: Composite Positive Bending Moment Design Capacity Calculation (Part 3)

Case III			*Case I: PNA in Web*						*Case II: PNA in Top Flange*					
$P_c + P_w + P_t$ <	P_{slab}	Is Case III True?	Y_{bar} From Top of Web	d_{slab}	d_w	d_t	d_c	M_p	Y_{bar} From Top of Flange	d_{slab}	d_w	d_t	d_c	M_p
(kip)	(kip)		(in)	(in)	(in)	(in)	(in)	(kip-in)	(in)	(in)	(in)	(in)	(in)	(kip-in)
Omit		n/a	Omit						0.006	6.3	30.9	61.4	n/a	122532.8
Omit		n/a	Omit						0.866	7.1	30.6	61.6	n/a	179113.1
2512.5	2880.0	Yes	Omit						Omit					
Omit		n/a	Omit						0.652	6.9	27.8	56.1	n/a	146588.6
Omit		n/a	Omit						0.402	6.7	27.6	55.8	n/a	144990.5
2303.1	2891.1	Yes	Omit						Omit					
2152.7	2891.1	Yes	Omit						Omit					
1938.9	2891.1	Yes	Omit						Omit					
Omit		n/a	Omit						0.919	7.2	28.1	55.8	n/a	132626.5
Omit		n/a	Omit						1.114	7.4	34.5	68.6	n/a	166275.8
1712.5	2910.0	Yes	Omit						Omit					
1894.1	2891.1	Yes	Omit						Omit					

Table J - 4: Composite Positive Bending Moment Design Capacity Calculation (Part 4)

Case III: PNA in Slab							Check if section is compact (6.10.6.2.2)						
							Yield Strength		Web Requirement		Web Slenderness		Is Flange Compact?
Y_{bar} From Top of Slab (in)	d_{slab} (in)	d_w (in)	d_t (in)	d_c (in)	M_p (kip-in)	M_p (kip-in)	F_{yf} < (ksi)	70.00 (ksi)	D/t_w < -	150.00 -	$2D_{cp}/t_w$ < -	$3.76(E/F_{yc})^{0.5}$ -	
Omit						122532.8	50.0	70.0	160.0	150.0	0.0	90.6	No
Omit						179113.1	50.0	70.0	160.0	150.0	0.0	90.6	No
6.5	n/a	31.2	59.0	3.8	103446.0	103446.0	50.0	70.0	144.0	150.0	0.0	90.6	Yes
Omit						146588.6	50.0	70.0	144.0	150.0	0.0	90.6	Yes
Omit						144990.5	50.0	70.0	144.0	150.0	0.0	90.6	Yes
6.0	n/a	35.8	67.0	4.5	89275.2	89275.2	50.0	70.0	164.0	150.0	0.0	90.6	No
5.6	n/a	32.2	59.5	4.9	75293.5	75293.5	50.0	70.0	142.9	150.0	0.0	90.6	Yes
5.0	n/a	27.0	48.6	5.5	57252.2	57252.2	50.0	70.0	112.3	150.0	0.0	90.6	Yes
Omit						132626.5	50.0	70.0	141.7	150.0	0.0	90.6	Yes
Omit						166275.8	50.0	70.0	176.5	150.0	0.0	90.6	No
4.4	n/a	22.0	38.1	6.0	41481.5	41481.5	50.0	70.0	57.1	150.0	0.0	90.6	Yes
4.9	n/a	21.6	37.7	5.5	45626.0	45626.0	50.0	70.0	54.1	150.0	0.0	90.6	Yes

Table J - 5: Composite Positive Bending Moment Design Capacity Calculation (Part 5)

Apply for Non-compact Sections							
Hybrid Factor	Determine Load Shedding Factor; R_b, 6.10.1.10.2					Flange Local Buckling Stress	
6.10.1.10.1 R_h -	D_c (in)	6.10.1.10.2-4 $\lambda_{rw} = 5.7(E/F_{yc})^{0.5}$ -	Is $2D_c/t_w < \lambda_{rw}$?	6.10.1.10.2-5 $a_{wc} = 2D_c t_w / b_{fc} t_{fc}$ -	6.10.1.10.2-3 $R_b < 1.0$ -	6.10.8.2.2-1 $F_{nc}(FLB) = R_b R_h F_{yc}$ (ksi)	φ_f
1.0	0.0	137.3	Yes	n/a	1.00	50.0	0.90
1.0	0.0	137.3	Yes	n/a	1.00	50.0	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90
1.0	0.0	137.3	Yes	n/a	1.00	50.0	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90
1.0	0.0	137.3	Yes	n/a	1.00	50.0	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90
1.0	0.0	137.3	Yes	n/a	1.00	Omit	0.90

Table J - 6: Composite Positive Bending Moment Design Capacity Calculation (Part 6)

Note: f_l is the lateral bending stress within the compression flange resulting from an applied lateral force such as wind. Since the compression flagne is laterally braced by the deck (+ moment), no lateral bending stresses are assumed to occur.

Apply for Non-compact Sections					
Allowable Compression Flange Stress	Allowable Tension Flange Stress				
6.10.7.2.1-1	6.10.7.2.2-2	6.10.7.2.1-2		6.10.1.3-1	
$f_{bu} < \varphi_f(F_{nc})$ (ksi)	$F_{nt} = R_h(F_{yt})$ (ksi)	$f_{bu} + 1/3f_l <$ (ksi)	$\varphi_f F_{nt}$ (ksi)	$f_l <$ (ksi)	$0.6F_{yt}$ (ksi)
45.0	50.0	45.00	45.00	0.00	30.00
45.0	50.0	45.00	45.00	0.00	30.00
Omit	Omit	'Omit			
Omit	Omit	Omit			
Omit	Omit	Omit			
45.0	50.0	45.00	45.00	0.00	30.00
Omit	Omit	Omit			
Omit	Omit	Omit			
Omit	Omit	Omit			
45.0	50.0	45.00	45.00	0.00	30.00
Omit	Omit	Omit			
Omit	Omit	Omit			

Table J - 7: Composite Positive Bending Moment Design Capacity Calculation (Part 7)

Apply for Compact Sections							
Nominal Flexural Resistance					Article B.6		
			6.10.7.1.2-1	6.10.7.1.2-2	B.6.2.2		
D_p (in)	$0.10D_t$ (in)	Is $D_p < 0.10D_t$?	$M_n = M_p$ (kip - in)	$M_n = M_p(1.07 - 0.7(D_p/D_t))$ (kip - in)	$b_{fc}/2t_{fc}$ -	$0.38(E/F_{yc})^{0.5}$ -	Is compression flange compact?
Omit							
Omit							
6.5	6.6	Yes	103446.0	n/a	8.0	9.2	Yes
10.7	6.8	No	n/a	140775.8	4.0	9.2	Yes
10.4	6.8	No	n/a	139499.2	6.0	9.2	Yes
Omit							
5.6	6.6	Yes	75293.5	n/a	5.8	9.2	Yes
5.0	5.4	Yes	57252.2	n/a	5.8	9.2	Yes
10.9	6.8	No	n/a	126947.3	2.4	9.2	Yes
Omit							
4.4	4.3	No	n/a	41395.1	7.8	9.2	Yes
4.9	4.3	No	n/a	45177.8	6.7	9.2	Yes

Table J - 8: Composite Positive Bending Moment Design Capacity Calculation (Part 8)

Note: Moment capacity is calculated using the limiting stress f_{ub} = 45 ksi for both the tension & compression flanges and web stress.

Apply for Compact Sections										
Article B.6							If No: $M_n = \min(M_n, 1.3R_hM_y)$			
B.6.2.2			B6.6.2			Article B.6 Satisfied?	I_{xx}	c	$M_y = F_y(I_{xx})/c$	M_n
b_{fc} (in)	D/4.25 (in)	*Adequate aspect ratio?*	moment rotation (rad) θ_{rl}	> 0.009	Satisfies B6.6.2?		(in^4)	(in)	(kip-in)	(kip-in)
Omit							116685.4	50.8	103323.6	
Omit							184353.2	47.3	175213.4	
12.0	15.6	No	0.027	0.009	Yes	No	98771.8	46.8	105524.6	103446.0
12.0	16.0	No	0.036	0.009	Yes	No	134976.8	44.4	151837.4	140775.8
12.0	15.9	No	0.034	0.009	Yes	No	133443.1	43.6	152911.4	139499.2
Omit							89459.9	55.9	72017.4	
11.5	15.4	No	0.023	0.009	Yes	No	67314.8	50.0	60640.5	75293.5
11.5	12.7	No	0.034	0.009	Yes	No	41828.9	41.1	45766.5	57252.2
12.0	15.9	No	0.038	0.009	Yes	No	117506.0	46.5	126463.4	126947.3
Omit							183441.9	55.6	148352.0	
11.5	10.1	Yes	0.028	0.009	Yes	Yes	22815.1	32.4	35251.7	
11.5	10.1	Yes	0.035	0.009	Yes	Yes	24818.5	32.0	38738.0	

Table J - 9: Composite Positive Bending Moment Design Capacity Calculation (Part 9)

Note: For sections that are **NOT** compact, plastic stress distribution does **NOT** apply, use linear stress distribution to determine (+) moment capacity.

Check ductility requirement (6.10.7.3-1)

D_p (in)	0.42D (in)	Is ductility requirement satified?	M_n (kip-in)	φM_n (kip-ft)
10.01	30.2	Yes	103323.6	8610.3
10.87	30.9	Yes	175213.4	14601.1
6.54	27.8	Yes	103446.0	7758.5
10.65	28.6	Yes	140775.8	10558.2
10.40	28.4	Yes	139499.2	10462.4
5.97	30.9	Yes	72017.4	6001.5
5.58	27.5	Yes	75293.5	6274.5
5.03	22.7	Yes	57252.2	4771.0
10.92	28.5	Yes	126947.3	9521.0
11.11	33.9	Yes	148352.0	12362.7
4.41	18.0	Yes	41395.1	3104.6
4.91	18.1	Yes	45177.8	3388.3

Appendix K. NON-COMPOSITE/ NEGATIVE COMPOSITE BENDING MOMENT CAPACITY CALCULATION

Table K - 1: Non-Composite/ Negative Composite Bending Moment Capacity (Part 1)

Nominal Negative Moment Capacity, M_n

f'_c (ksi):	4	F_y rebar (ksi):	60	
F_y A588 (ksi):	50	Top of slab to top flange (in):		10.0

Plate Girder Sections

Sections	I Section Dimensions: Top Flange Width (in)	Top Flange Thickness (in)	Bottom Flange Width (in)	Bottom Flange Thickness (in)	Web Height (in)	Web Thickness (in)	Slab Dimensions: Slab Thickness (in)	Effective Slab Width (in)	Haunch: Bottom Flange θ (deg)
South Approach Interior Girder									
south g. 1	16.00	0.875	20.00	1.125	60.00	0.375	7.50	98.00	
south g. 2	16.00	1.500	20.00	2.000	60.00	0.375	7.50	98.00	
North Approach Interior Girder									
N. ap. g. 1	12.00	0.750	14.00	1.500	54.00	0.375	7.50	96.00	
N. ap. g. 2	12.00	1.500	14.00	2.500	54.00	0.375	7.50	96.00	
N. ap. g. 3	12.00	1.000	14.00	2.500	54.00	0.375	7.50	96.00	
Arch Exterior Girder (Haunched)									
S str. h. arch 1	11.50	1.000	11.50	1.000	61.50	0.375	7.50	96.37	0.00
S str. h. arch 2	11.50	1.000	11.50	1.000	53.59	0.375	7.50	96.37	5.00
S str. h. arch 3	11.50	1.000	11.50	1.000	42.09	0.375	7.50	96.37	2.00
North Approach Interior Girder (Haunched)									
N. h. ap. g. 1	12.00	2.500	14.00	2.125	53.13	0.375	7.50	96.00	1.00
N. h. ap. g. 2	12.00	2.500	14.00	2.125	66.19	0.375	7.50	96.00	7.00
N. h. ap. g. 3	12.00	2.500	14.00	2.125	89.50	0.375	7.50	96.00	13.00
N. h. ap. g. 4	12.00	2.500	14.00	2.125	102.69	0.375	7.50	96.00	0.00

Note: θ is the inclination of the bottom flange measured from horizontal

Rolled I Sections

Sections	I Section Dimensions: Top Flange Width (in)	Top Flange Thickness (in)	Bottom Flange Width (in)	Bottom Flange Thickness (in)	Web Height (in)	Web Thickness (in)	Slab Dimensions: Slab Thickness (in)	Effective Slab Width (in)
W33x118(AISC)	11.48	0.740	11.48	0.740	31.38	0.550	7.50	97.00
W33x130(AISC)	11.51	0.855	11.51	0.855	31.38	0.580	7.50	96.37

Table K - 2: Non-Composite/ Negative Composite Bending Moment Capacity (Part 2)

South Approach Interior Girder

Top Reinforcement:				*Bottom Reinforcement:*				*Minimum Clear Distance:*		*Section Areas:*		
Diam. # 4 Bar (in)	Area # 4 Bar (in^2)	Number of Bars Located Within Effective Width	A_{rt} (in^2)	Diam. # 6 Bar (in)	Area # 6 Bar (in^2)	Number of Bars Located Within Effective Width	A_{rb} (in^2)	Top Reinforcment (in)	Bottom Reinforcement (in)	A_{tf} (in^2)	A_{web} (in^2)	A_{bf} (in^2)
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	14.00	22.50	22.50
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	24.00	22.50	40.00
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	9.00	20.25	21.00
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	18.00	20.25	35.00
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	12.00	20.25	35.00
0.50	0.20	8	1.57	0.75	0.44	9	3.97	2.50	1.00	11.50	23.06	11.50
0.50	0.20	8	1.57	0.75	0.44	9	3.97	2.50	1.00	11.50	20.10	11.50
0.50	0.20	8	1.57	0.75	0.44	9	3.97	2.50	1.00	11.50	15.79	11.50
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	30.00	19.92	29.75
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	30.00	24.82	29.75
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	30.00	33.56	29.75
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	30.00	38.51	29.75

South Approach Interior Girder

Top Reinforcement:				*Bottom Reinforcement:*				*Minimum Clear Distance:*		*Section Areas:*		
Diam. # 4 Bar (in)	Area # 4 Bar (in^2)	Number of Bars Located Within Effective Width	A_{rt} (in^2)	Diam. # 6 Bar (in)	Area # 6 Bar (in^2)	Number of Bars Located Within Effective Width	A_{rb} (in^2)	Top Reinforcment (in)	Bottom Reinforcement (in)	A_{tf} (in^2)	A_{web} (in^2)	A_{bf} (in^2)
0.50	0.20	9	1.77	0.75	0.44	9	3.97	2.50	1.00	8.50	17.26	8.50
0.50	0.20	8	1.57	0.75	0.44	9	3.97	2.50	1.00	9.84	18.20	9.84

Table K - 3: Non-Composite/ Negative Composite Bending Moment Capacity (Part 3)

Determine Composite Negative Moment Capacity

Calculate PNA and Plastic Moment Capacity

Section Forces:								Case I			Case II		
P_{rt} (kip)	P_{rb} (kip)	P_{slab} (kip)	P_{tf} (kip)	P_{web} (kip)	P_{bf} (kip)	P_c (kip)	P_t (kip)	$P_c + P_w >$ (kip)	$P_t + P_{rb} + P_{rt}$ (kip)	Is Case I True?	$P_c + P_w + P_t >$ (kip)	$P_{rb} + P_{rt}$ (kip)	Is Case II True?
106	238	2499	700	1125	1125	1125	700	2250	1044	Yes	Omit when Case I is true.		omit
106	238	2499	1200	1125	2000	2000	1200	3125	1544	Yes			
106	238	2448	450	1013	1050	1050	450	2063	794	Yes	Omit when Case I is true.		omit
106	238	2448	900	1013	1750	1750	900	2763	1244	Yes			
106	238	2448	600	1013	1750	1750	600	2763	944	Yes			
94	238	2457	575	1153	575	575	575	1728	908	Yes	Omit when Case I is true.		
94	238	2457	575	1005	573	573	575	1578	908	Yes			
94	238	2457	575	789	575	575	575	1364	908	Yes			
106	238	2448	1500	996	1487	1487	1500	2483	1844	Yes	Omit when Case I is true.		
106	238	2448	1500	1241	1476	1476	1500	2717	1844	Yes			
106	238	2448	1500	1678	1449	1449	1500	3127	1844	Yes			
106	238	2448	1500	1925	1488	1488	1500	3413	1844	Yes			

Calculate PNA and Plastic Moment Capacity

Section Forces:								Case I			Case II		
P_{rt} (kip)	P_{rb} (kip)	P_{slab} (kip)	P_{tf} (kip)	P_{web} (kip)	P_{bf} (kip)	P_c (kip)	P_t (kip)	$P_c + P_w >$ (kip)	$P_t + P_{rb} + P_{rt}$ (kip)	Is Case I True?	$P_c + P_w + P_t >$ (kip)	$P_{rb} + P_{rt}$ (kip)	Is Case II True?
106	238	2474	425	863	425	425	425	1288	769	Yes			
94	238	2457	492	910	492	492	492	1402	825	Yes			

Table K - 4: Non-Composite/ Negative Composite Bending Moment Capacity (Part 4)

Case I						*Case II*						*Compression Flange Slenderness*		
Y_{bar} From Top of Web (in)	d_{rt} (in)	d_{rb} (in)	d_t (in)	d_c (in)	M_p (kip-in)	Y_{bar} From Top of Flange (in)	d_{rt} (in)	d_{rb} (in)	d_w (in)	d_c (in)	M_p (kip-in)	6.10.8.5.5-3 $\lambda_f = b_{fc}/2t_{fc}$ flange	6.10.8.2.2-4 $\lambda_{pf} = 0.38(E/F_{yc})^{0.5}$ compact limit	Compact Flange? $\lambda < \lambda_{pf}$ -
32.15	40.27	36.90	32.59	28.41	84804	Omit when Case I is true.						8.89	9.15	**Yes**
42.15	50.90	47.52	42.90	18.85	125549							5.00	9.15	**Yes**
33.82	41.82	38.44	34.19	20.93	65504	Omit when Case I is true.						4.67	9.15	**Yes**
40.48	49.23	45.86	41.23	14.77	96181							2.80	9.15	**Yes**
48.48	56.73	53.36	48.98	6.77	82289							2.80	9.15	**Yes**
21.88	30.13	26.75	22.38	40.12	64360	Omit when Case I is true.						5.75	9.15	**Yes**
17.87	26.12	22.74	18.37	36.23	54153							5.75	9.15	**Yes**
12.17	20.42	17.04	12.67	30.43	40539							5.75	9.15	**Yes**
17.04	26.79	23.41	18.29	37.15	106035	Omit when Case I is true.						3.29	9.15	**Yes**
23.28	33.03	29.65	24.53	43.97	134620							3.29	9.15	**Yes**
34.21	43.96	40.59	35.46	56.34	188823							3.29	9.15	**Yes**
41.82	51.57	48.20	43.07	61.92	224805							3.29	9.15	**Yes**

Case I						*Case II*						*Compression Flange Slenderness*		
Y_{bar} From Top of Web (in)	d_{rt} (in)	d_{rb} (in)	d_t (in)	d_c (in)	M_p (kip-in)	Y_{bar} From Top of Flange (in)	d_{rt} (in)	d_{rb} (in)	d_w (in)	d_c (in)	M_p (kip-in)	6.10.8.5.5-3 $\lambda_f = b_{fc}/2t_{fc}$ flange	6.10.8.2.2-4 $\lambda_{pf} = 0.38(E/F_{yc})^{0.5}$ compact limit	Compact Flange? $\lambda < \lambda_{pf}$ -
9.43	17.42	14.04	9.80	22.32	26686							7.76	9.15	**Yes**
9.95	18.06	14.68	10.38	21.85	29157							6.73	9.15	**Yes**

Table K - 5: Non-Composite/ Negative Composite Bending Moment Capacity (Part 5)

Note: D_c is depth of web in compression in the elastic range (in). For composite sections in negative flexure only the steel section and longitudinal reinforcement need be considered.

Note: Load Shedding Factor must be less than or equal to one.

Determine allowable flange local buckling stress in compression flange								
Hybrid Factor	Elastic Neutral Axis	Determine Load Shedding Factor; R_b, 6.10.1.10.2						Flange Local Buckling Stress
6.10.1.10.1 R_h -	Measured from bottom of section (in)	D_c (in)	$2D_c/t_w$ -	6.10.1.10.2-4 $\lambda_{rw} = 5.7(E/F_{yc})^{0.5}$ -	Is $2D_c/t_w < \lambda_{rw}$?	6.10.1.10.2-5 $a_{wc} = 2D_c t_w / b_{fc} t_{fc}$ -	6.10.1.10.2-3 $R_b < 1.0$ -	6.10.8.2.2-1 $F_{nc}(FLB) = R_b R_h F_{yc}$ (ksi)
1.00	30.26	29.13	155.38	137.27	No	0.97	0.99	49.41
1.00	28.82	26.82	143.06	137.27	No	0.50	1.00	49.89
1.00	25.84	24.34	129.82	137.27	Yes	n/a	1.00	50.00
1.00	25.73	23.23	123.92	137.27	Yes	n/a	1.00	50.00
1.00	23.06	20.56	109.67	137.27	Yes	n/a	1.00	50.00
1.00	35.68	34.68	184.96	137.27	No	2.26	0.94	47.13
1.00	31.52	30.52	162.75	137.27	No	1.99	0.97	48.59
1.00	25.41	24.41	130.18	137.27	Yes	n/a	1.00	50.00
1.00	31.12	28.99	154.63	137.27	No	0.73	0.99	49.55
1.00	37.95	35.82	191.06	137.27	No	0.90	0.97	48.35
1.00	50.07	47.94	255.69	137.27	No	1.21	0.91	45.42
1.00	56.89	54.77	292.08	137.27	No	1.38	0.87	43.38

Determine allowable flange local buckling stress in compression flange								
Hybrid Factor	Elastic Neutral Axis	Determine Load Shedding Factor; R_b, 6.10.1.10.2						Flange Local Buckling Stress
6.10.1.10.1 R_h -	Measured from bottom of section (in)	D_c (in)	$2D_c/t_w$ -	6.10.1.10.2-4 $\lambda_{rw} = 5.7(E/F_{yc})^{0.5}$ -	Is $2D_c/t_w < \lambda_{rw}$?	6.10.1.10.2-5 $a_{wc} = 2D_c t_w / b_{fc} t_{fc}$ -	6.10.1.10.2-3 $R_b < 1.0$ -	6.10.8.2.2-1 $F_{nc}(LFB) = R_b R_h F_{yc}$ (ksi)
1.00	19.49	18.75	68.20	137.27	Yes	n/a	1.00	50.00
1.00	19.27	18.42	63.51	137.27	Yes	n/a	1.00	50.00

Table K - 6: Non-Composite/ Negative Composite Bending Moment Capacity (Part 6)

Note: If $L_b < L_p$ then unbraced length is compact

Note: If $L_b < L_r$ then unbrace length is noncompact

Determine allowable lateral torsional buckling stress in compression flange

Radius Gyration (Compression Sect.)	Compact Unbraced Length	Unbraced Length		Noncompact Unbraced Length		
6.10.8.2.3-9	6.10.8.2.3-4			6.10.8.2.3-5		
$r_t = b_f/(12(1+(1/3)(D_c t_w/b_{fc} t_{fc})))^{0.5}$ (in)	$L_p = 1.0r_t(E/F_{yc})^{0.5}$ (in)	L_b (in)	$L_b < L_p$?	$F_{yr} = 0.7F_{yc}$ (ksi)	$L_r = \pi r_t(E/F_{yr})^{0.5}$ (in)	$L_b < L_r$?
5.36	129.00	96.25	No	35.00	484.13	Yes
5.55	133.56	96.25	No	35.00	501.25	Yes
3.78	90.96	84.67	No	35.00	341.39	Yes
3.88	93.53	84.67	No	35.00	351.01	Yes
3.90	93.94	84.67	No	35.00	352.57	Yes
2.83	68.13	180.83	No	35.00	255.71	Yes
2.88	69.28	180.83	No	35.00	260.02	Yes
2.95	71.08	180.83	No	35.00	266.75	Yes
3.82	91.89	84.72	No	35.00	344.88	Yes
3.77	90.74	84.72	No	35.00	340.55	Yes
3.69	88.80	84.72	No	35.00	333.26	Yes
3.64	87.76	84.72	No	35.00	329.35	Yes

Determine allowable lateral torsional buckling stress in compression flange

Radius Gyration (Compression Sect.)	Compact Unbraced Length	Unbraced Length		Noncompact Unbraced Length		
6.10.8.2.3-9	6.10.8.2.3-4			6.10.8.2.3-5		
$r_t = b_f/(12(1+(1/3)(D_c t_w/b_{fc} t_{fc})))^{0.5}$ (in)	$L_p = 1.0r_t(E/F_{yc})^{0.5}$ (in)	L_b (in)	$L_b < L_p$?	$F_{yr} = 0.7F_{yc}$ (ksi)	$L_r = \pi r_t(E/F_{yr})^{0.5}$ (in)	$L_b < L_r$?
2.80	67.34	209.00	No	35.00	252.73	Yes
2.85	68.57	209.00	No	35.00	257.34	Yes

Table K - 7: Non-Composite/ Negative Composite Bending Moment Capacity (Part 7)

Note: In lieu of calculating a C_b value, it may conservatively be assumed equal to 1.0, C6.10.8.2.3

Allowable Lateral Torsional Buckling stress		Allowable Compression Flange Stress
C_b	6.10.8.2.3-2 F_{nc} (LTB) = $C_b[1 - (1 - (F_{yr}/R_hF_{yc}))((L_b - L_p)/(L_r - L_p))]R_bR_hF_{yc}$	F_{nc} = min(F_{nc} (LFB), F_{nc} (LTB))
-	(ksi)	(ksi)
1.00	50.78	49.41
1.00	51.41	49.89

1.00	50.38	50.00
1.00	50.52	50.00
1.00	50.54	50.00

If No (6.10.8.2.3-8): F_{nc} (LTB) = $C_bR_b\pi^2E/(L_b/r_t)^2$

1.00	38.63	38.63
1.00	40.06	40.06
1.00	41.59	41.59

1.00	49.97	49.55
1.00	48.70	48.35
1.00	45.65	45.42
1.00	43.54	43.38

If No (6.10.8.2.3-8): F_{nc} (LTB) = $C_bR_b\pi^2E/(L_b/r_t)^2$

Allowable Lateral Torsional Buckling stress		Allowable Compression Flange Stress
C_b	6.10.8.2.3-2 F_{nc} (LTB) = $C_b[1 - (1 - (F_{yr}/R_hF_{yc}))((L_b - L_p)/(L_r - L_p))]R_bR_hF_{yc}$	F_{nc} = min(F_{nc} (LFB), F_{nc} (LTB))
-	(ksi)	(ksi)
1.00	38.54	38.54
1.00	38.84	38.84

Table K - 8: Non-Composite/ Negative Composite Bending Moment Capacity (Part 8)

Note: Table 3.8.1.1-1 allows that V_{30} by be taken as V_B = 100 mph in absence of better criterion.

Determine lateral bending stress (wind load) on compression flange
Open Country

Table 3.8.1.1-1		3.8.1.1			3.8.1.1-1	Table 3.8.1.2.2-1 Beam	*3.8.1.2.1-1*	*3.8.1.2.1-1*	*3.4.1-1*	*1.3.3*	*1.3.4*	*1.3.5*	*1.3.2.1*
V_o	Z_o	V_B	V_{30}	Z	V_{DZ}	P_B	P_D	P_D	γ_{WS}	η_D	η_R	η_I	η_i
(mph)	*(ft)*	*(mph)*	*(mph)*	*(ft)*	*(mph)*	*(ksf)*	*(ksf)*	*(ksi)*	-	-	-	-	-
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00

Determine lateral bending stress (wind load) on compression flange
Open Country

Table 3.8.1.1-1		3.8.1.1			3.8.1.1-1	Table 3.8.1.2.2-1 Beam	*3.8.1.2.1-1*	*3.8.1.2.1-1*	*3.4.1-1*	*1.3.3*	*1.3.4*	*1.3.5*	*1.3.2.1*
V_o	Z_o	V_B	V_{30}	Z	V_{DZ}	P_B	P_D	P_D	γ_{WS}	η_D	η_R	η_I	η_i
(mph)	*(ft)*	*(mph)*	*(mph)*	*(ft)*	*(mph)*	*(ksf)*	*(ksf)*	*(ksi)*	-	-	-	-	-
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00
8.20	0.23	100.00	100.00	500.00	157.53	0.05	0.12	0.0009	-	1.00	1.00	1.00	1.00

Table K - 9: Non-Composite/ Negative Composite Bending Moment Capacity (Part 9)

Note: f_{bu} used to calculate f_l is the largest stress calculated by enveloping dead and live loads in Larsa. f_{bu} = 20 ksi for approach girders, 6 ksi for Arch sections.

Wind load on compression flange continued.

Distributed Wind Load	Moment Resulting From Wind	Wind Bending Stress	Elastic Lateral Torsion Buckling Stress	
C4.6.2.7.1-1	*C4.6.2.7.1-2*		*6.10.8.2.3-8*	*6.10.1.6-4*
W	M_w	f_{l1}	F_{cr}	$f_l = f_{l1}(0.85/(1 - f_{bu}/F_{cr}))$
(kip/in)	*(kip-in)*	*(ksi)*	*(ksi)*	*(ksi)*
0.03	24.74	0.33	875.05	0.29
0.03	25.34	0.19	947.19	0.17
0.02	17.37	0.35	569.04	0.31
0.02	17.91	0.22	601.57	0.19
0.02	17.76	0.22	606.92	0.19
0.02	53.68	2.44	65.97	2.97
0.01	46.99	2.13	70.32	2.53
0.01	37.27	1.69	76.16	1.95
0.01	10.72	0.15	574.78	0.14
0.02	13.14	0.19	546.82	0.17
0.02	17.46	0.25	491.93	0.22
0.03	19.91	0.29	458.88	0.25

Wind load on compression flange continued.

Distributed Wind Load	Moment Resulting From Wind	Wind Bending Stress	Elastic Lateral Torsion Buckling Stress	
C4.6.2.7.1-1	*C4.6.2.7.1-2*		*6.10.8.2.3-8*	*6.10.1.6-4*
W	M_w	f_{l1}	F_{cr}	$f_l = f_{l1}(0.85/(1 - f_{bu}/F_{cr}))$
(kip/in)	*(kip-in)*	*(ksi)*	*(ksi)*	*(ksi)*
0.01	37.10	2.28	51.18	2.20
0.01	37.36	1.98	53.06	1.90

Table K - 10: Non-Composite/ Negative Composite Bending Moment Capacity (Part 10)

Note: moment capacity is calculated using the limiting tension stress f_{bu} = 45 ksi and limiting compression stress $f_{bu} = \varphi_f F_{nc} - (1/3) f_l$.

Note: Since section is **NOT** compact, plastic stress distribution does **NOT** apply, use linear stress distribution to determine (-) moment capacity.

Note: Neutral axis is measured from bottom surface of section, limiting compression stress controls.

φ_f	6.10.8.1.1-1		6.10.1.6-1		6.10.8.1.3-1		Elastic Moment Capacity		(-) Bending	(-) Bending
	$f_{bu} = \varphi_f F_{nc} - (1/3) f_l$ (ksi)	$\varphi_f F_{nc}$ (ksi)	f_l < (ksi)	$0.6F_{yc}$ (ksi)	f_{ub} < (ksi)	$\varphi_f R_h F_{yt}$ (ksi)	I_{eff} (in^4)	$M_y = (\sigma_{bu} I_{eff})/y_{bar}$ (kip-in)	M_n (-) (kip-in)	M_n (-) (kip-ft)
0.90	44.37	44.47	0.29	30.00	44.37	45.00	48057.76	70476.30	70476.30	5873.03
0.90	44.85	44.90	0.17	30.00	44.85	45.00	74596.24	116067.34	116067.34	9672.28
0.90	44.90	45.00	0.31	30.00	44.90	45.00	33583.58	58347.84	58347.84	4862.32
0.90	44.94	45.00	0.19	30.00	44.94	45.00	52038.58	90866.50	90866.50	7572.21
0.90	44.94	45.00	0.19	30.00	44.94	45.00	45155.71	87981.02	87981.02	7331.75
0.90	33.78	34.77	2.97	30.00	33.78	45.00	36366.70	34432.03	34432.03	2869.34
0.90	35.21	36.06	2.53	30.00	35.21	45.00	27191.73	30383.40	30383.40	2531.95
0.90	36.78	37.43	1.95	30.00	36.78	45.00	16528.08	23924.33	23924.33	1993.69
0.90	44.55	44.60	0.14	30.00	44.55	45.00	56757.87	81262.89	81262.89	6771.91
0.90	43.46	43.51	0.17	30.00	43.46	45.00	87932.99	100698.81	100698.81	8391.57
0.90	40.80	40.88	0.22	30.00	40.80	45.00	162952.77	132802.16	132802.16	11066.85
0.90	38.96	39.04	0.25	30.00	38.96	45.00	217155.83	148700.10	148700.10	12391.67

φ_f	6.10.8.1.1-1		6.10.1.6-1		6.10.8.1.3-1		Elastic Moment Capacity		(-) Bending	(-) Bending
	$f_{bu} = \varphi_f F_{nc} - (1/3) f_l$ (ksi)	$\varphi_f F_{nc}$ (ksi)	f_l < (ksi)	$0.6F_{yc}$ (ksi)	f_{ub} < (ksi)	$\varphi_f R_h F_{yt}$ (ksi)	I_{eff} (in^4)	$M_y = (\sigma_{bu} I_{eff})/y_{bar}$ (kip-in)	M_n (-) (kip-in)	M_n (-) (kip-ft)
0.90	33.95	34.68	2.20	30.00	33.95	45.00	8053.00	14025.92	14025.92	1168.83
0.90	34.33	34.96	1.90	30.00	34.33	45.00	8830.44	15726.46	15726.46	1310.54

Appendix L. PRISMATIC SHEAR CAPACITY CALCULATION

Table L - 1: Prismatic Shear Capacity Calculation (Part 1)

Shear Resistance Design Capacity (Article 6.10.9) AASHTO LRFD Bridge Design Specifications 2005 Interim Revision

f'_c (ksi): 4.0 — F_y rebar (ksi): 60.0

F_y A588 (ksi): 50.0 — Top of slab to top flange (in): 10.0

Note: Haunch pad is the distance from the top of the top flange to the bottom surface of the slab.

Plate Girder Sections:

Prismatic Plate Girder Sections

Sections	*I Section Dimensions:*						*Slab Dimensions:*		*Haunch:*	
	Top Flange Width (in)	Top Flange Thickness (in)	Bottom Flange Width (in)	Bottom Flange Thickness (in)	Web Height (in)	Web Thickness (in)	Slab Thickness (in)	Effective Slab Width (in)	Bottom Flange θ (deg)	Haunch Pad (in)
South Approach Interior Girder										
south g. 1	16.00	0.875	20.00	1.125	60.00	0.375	7.50	98.00		2.50
south g. 2	16.00	1.500	20.00	2.000	60.00	0.375	7.50	98.00		2.50
south g. 3	16.00	2.000	20.00	2.000	60.00	0.375	7.50	98.00		2.50
south g. 4	16.00	2.000	20.00	2.625	60.00	0.375	7.50	98.00		2.50
North Approach Interior Girder										
N. ap. g. 1	12.00	0.750	14.00	1.500	54.00	0.375	7.50	96.00		2.50
N. ap. g. 2	12.00	1.500	14.00	2.500	54.00	0.375	7.50	96.00		2.50
N. ap. g. 3	12.00	1.000	14.00	2.500	54.00	0.375	7.50	96.00		2.50
Arch Girder Sections										
S str. h. arch 6	11.50	1.000	11.50	1.000	31.00	0.375	7.50	96.37		2.50
S str. h. arch' 6	11.50	0.750	11.50	0.750	31.00	0.375	7.50	95.74		2.50
north str. hnch 6 o	11.50	1.000	11.50	1.000	31.00	0.375	7.50	96.37		2.50
north str. hnch 6 i	11.50	0.750	11.50	0.750	31.00	0.375	7.50	95.74		2.50

Rolled I-Shape Sections:

Sections	Top Flange Width (in)	Top Flange Thickness (in)	Bottom Flange Width (in)	Bottom Flange Thickness (in)	Web Height (in)	Web Thickness (in)	Slab Thickness (in)	Effective Slab Width (in)	Bottom Flange θ (deg)	Haunch Pad (in)
Arch Stringers										
W33x118(AISC)	11.48	0.740	11.48	0.740	31.38	0.550	7.50	97.00		2.50
W33x130(AISC)	11.51	0.855	11.51	0.855	31.38	0.580	7.50	96.37		2.50

Table L - 2: Prismatic Shear Capacity Calculation (Part 2)

Interior Panels

Note: All plate girders are stiffened, Article 6.10.9.3 applies

6.10.9.3.2-1		Tension Field Action 6.10.9.3.2-3		Stiffener Spacing	*Shear-Buckling Coef.* 6.10.9.3.2-7	*Limit*	*Limit*
$2Dt_w/(b_{fc}t_{fc} + b_{ft}t_{ft})$	< 2.5	$V_p = 0.58F_{yw}Dt_w$	D/t_w	d_o	k	$1.12(Ek/f_{yw})^{0.5}$	$1.40(Ek/f_{yw})^{0.5}$
-	-	*(kip)*	-	*(in)*	-	-	-
1.23	2.5	52.2	160	36.75	18.3	408.3	510.3
0.70	2.5	52.2	160	36.75	18.3	408.3	510.3
0.63	2.5	52.2	160	36.75	18.3	408.3	510.3
0.53	2.5	52.2	160	57.75	10.4	307.5	384.4
1.35	2.5	47.0	144	36.75	15.8	379.0	473.8
0.76	2.5	47.0	144	42.31	13.1	345.7	432.2
0.86	2.5	47.0	144	42.31	13.1	345.7	432.2
*Unstiffened sections **DO NOT** have interior shear capacity*					**6.10.9.2**		
1.01	2.5	27.0	82.667	**Omit**	5.0	213.2	266.6
1.35	2.5	27.0	82.667	**Omit**	5.0	213.2	266.6
1.01	2.5	27.0	82.667	**Omit**	5.0	213.2	266.6
1.35	2.5	27.0	82.667	**Omit**	5.0	213.2	266.6
*Unstiffened sections **DO NOT** have interior shear capacity*							
2.03	2.5	40.0	57.055	**Omit**	5.0	213.2	266.6
1.85	2.5	42.2	54.103	**Omit**	5.0	213.2	266.6

Table L - 3: Prismatic Shear Capacity Calculation (Part 3)

Is $D/t_w < 1.12(Ek/f_{yw})^{0.5}$?	Is $1.12(Ek/f_{yw})^{0.5} < D/t_w < 1.40(Ek/f_{yw})^{0.5}$?	Is $1.40(Ek/f_{yw})^{0.5} < D/t_w$?	6.10.9.3.2-2 C -	6.10.9.3.2-2 $V_n = V_p(C + 0.87(1-C)/(1+(d_o/D)^2)^{0.5}$ (kip)	V_n Interior (kip)
No	No	Yes	8.15	224.0	224.0
No	No	Yes	8.15	224.0	224.0
No	No	Yes	8.15	224.0	224.0
No	No	Yes	4.62	157.5	157.5
No	No	Yes	8.67	220.8	220.8
No	No	Yes	7.21	196.5	196.5
No	No	Yes	7.21	196.5	196.5
No	No	Yes	8.33	n/a	n/a
No	No	Yes	8.33	n/a	n/a
No	No	Yes	8.33	n/a	n/a
No	No	Yes	8.33	n/a	n/a
			6.10.9.3.2-4		
Yes	n/a	n/a	1.0	n/a	n/a
Yes	n/a	n/a	1.0	n/a	n/a

Table L - 4: Prismatic Shear Capacity Calculation (Part 4)

End Panels

Note: Shear in end panels adjacent to simple suports is limited to either the shear-yielding or shear buckling resistance in order to provide an anchor for the tension field in adjacent interior panels.

6.10.9.3.2-2 C -	*6.10.9.3.3-1* $V_{cr} = CV_p$ *(kip)*	V_n *End* *(kip)*
8.15	425.4	425.4
8.15	425.4	425.4
8.15	425.4	425.4
4.62	241.3	241.3
8.67	407.3	407.3
7.21	339.0	339.0
7.21	339.0	339.0
8.33	224.6	224.6
8.33	224.6	224.6
8.33	224.6	224.6
8.33	224.6	224.6
1.0	40.0	40.0
1.0	42.2	42.2

Appendix M. HAUNCHED SECTION PLASTIC SHEAR CAPACITY CALCULATION

The following Mathcad worksheet calculates the plastic shear capacity for haunched girder sections based on the yield strength of steel, slope of bottom flange, radius of curvature of bottom flange, dimensions of I-section, and applied moment. The worksheet was created by Richard J. Nielsen based on the structural mechanics presented by Blodgett (1966).

Solving for the plastic shear capacity as a function of the section geometry and applied moment – Richard J. Nielsen

Given

web steel yield stress

$\sigma_{yield} := 50000psi$

slope of bottom flange θ:

$\theta := 0.5 \cdot deg$

Radius of curvature of bottom flange

$r \equiv 1000 ft$

Web thickness

$t_w := \frac{3}{8} \cdot in$

Applied moment

$M \equiv 900 kip \cdot ft$

Assume the bottom flange is in compression, and the haunch is parabolic.

Section properties calculated in Excel file "MEMBER CAPACITY"

$I_x := 6852.3 in^4$

$t_f := 0.75 \cdot in$

$h_w := 35.5 \cdot in$

$w_f := 11.5 \cdot in$

$A_w := t_w \cdot h_w$

$A_f := t_f \cdot w_f$

$c := 17.5 \cdot in$

$y_{max} := 18.25 \cdot in$

Calculating the Huber-von Mises stress

$$\sigma_{cr} = \sigma_{yield} = \sqrt{\sigma_x^2 - \sigma_x \cdot \sigma_y + \sigma_y^2 + 3 \cdot \tau_{xy}^2}$$

Substituting the net normal stress in the y-direction

$$\sigma_y = \sigma_{y\_M} + \sigma_{y\_f}$$

$$\sigma_{yield} = \sqrt{\sigma_x^2 - \sigma_x \cdot \left(\sigma_{y\_M} + \sigma_{y\_f}\right) + \left(\sigma_{y\_M} + \sigma_{y\_f}\right)^2 + 3 \cdot \tau_{xy}^2}$$

Substituting the compressive stress in the web due to bottom flange curvature

$$\sigma_{y\_f} = -\frac{F_x}{t_w \cdot r}$$

$$\sigma_{yield} = \sqrt{\sigma_x^2 - \sigma_x \cdot \left(\sigma_{y\_M} - \frac{F_x}{t_w \cdot r}\right) + \left(\sigma_{y\_M} - \frac{F_x}{t_w \cdot r}\right)^2 + 3 \cdot \tau_{xy}^2}$$

Square both sides for convenience.

$$\sigma_{yield}^2 = \sigma_x^2 - \sigma_x \cdot \left(\sigma_{y\_M} - \frac{F_x}{t_w \cdot r}\right) + \left(\sigma_{y\_M} - \frac{F_x}{t_w \cdot r}\right)^2 + 3 \cdot \tau_{xy}^2$$

Substitute the horizontal stresses for the x & y direction stresses

$$\sigma_x = \frac{\sigma_h + \sigma_v}{2} + \frac{\sigma_h - \sigma_v}{2} \cdot \cos(-2 \cdot \theta) + \tau_w \cdot \sin(-2 \cdot \theta)$$

$$\tau_{xy} = -\frac{\sigma_h - \sigma_v}{2} \cdot \sin(-2 \cdot \theta) + \tau_w \cdot \cos(-2 \cdot \theta)$$

$$\sigma_{y\_M} = \frac{\sigma_h + \sigma_v}{2} + \frac{\sigma_h - \sigma_v}{2} \cdot \cos(-2 \cdot \theta + 180 \deg) + \tau_w \cdot \sin(-2 \cdot \theta + 180 \deg)$$

$$A = \left(\frac{\sigma_h + \sigma_v}{2} + \frac{\sigma_h - \sigma_v}{2} \cdot \cos(-2 \cdot \theta) + \tau_w \cdot \sin(-2 \cdot \theta)\right)^2$$

$$B = \left(\frac{\sigma_h + \sigma_v}{2} + \frac{\sigma_h - \sigma_v}{2} \cdot \cos(-2 \cdot \theta) + \tau_w \cdot \sin(-2 \cdot \theta)\right)$$

$$C = \frac{\sigma_h + \sigma_v}{2} + \frac{\sigma_h - \sigma_v}{2} \cdot \cos(-2 \cdot \theta + 180 \deg) + \tau_w \cdot \sin(-2 \cdot \theta + 180 \deg) - \frac{F_h}{t_w \cdot r \cdot \cos(\theta)}$$

$$D = \left(\frac{\sigma_h + \sigma_v}{2} + \frac{\sigma_h - \sigma_v}{2} \cdot \cos(-2 \cdot \theta + 180 \deg) + \tau_w \cdot \sin(-2 \cdot \theta + 180 \deg) - \frac{F_h}{t_w \cdot r \cdot \cos(\theta)}\right)^2$$

$$E = 3 \cdot \left(-\frac{\sigma_h - \sigma_v}{2} \cdot \sin(-2 \cdot \theta) + \tau_w \cdot \cos(-2 \cdot \theta)\right)^2$$

$$\sigma_{yield}^2 = A - B \cdot C + D + E$$

Since the vertical compressive stress σ_v is always zero, this simplifies slightly

$$A = \left(\frac{\sigma_h}{2} + \frac{\sigma_h}{2}\cdot\cos(-2\cdot\theta) + \tau_w\cdot\sin(-2\cdot\theta)\right)^2$$

$$B = \left(\frac{\sigma_h}{2} + \frac{\sigma_h}{2}\cdot\cos(-2\cdot\theta) + \tau_w\cdot\sin(-2\cdot\theta)\right)$$

$$C = \left(\frac{\sigma_h}{2} + \frac{\sigma_h}{2}\cdot\cos(-2\cdot\theta + 180\cdot\deg) + \tau_w\cdot\sin(-2\cdot\theta + 180\cdot\deg) - \frac{F_h}{t_w\cdot r\cdot\cos(\theta)}\right)$$

$$D = \left(\frac{\sigma_h}{2} + \frac{\sigma_h}{2}\cdot\cos(-2\cdot\theta + 180\cdot\deg) + \tau_w\cdot\sin(-2\cdot\theta + 180\cdot\deg) - \frac{F_h}{t_w\cdot r\cdot\cos(\theta)}\right)^2$$

$$E = 3\cdot\left(-\frac{\sigma_h}{2}\cdot\sin(-2\cdot\theta) + \tau_w\cdot\cos(-2\cdot\theta)\right)^2$$

$$\sigma_{yield}^2 = A - B\cdot C + D + E$$

Substituting the following

Shear stress in the web

$$\tau_w = \frac{V - F_v}{A_w}$$

Compressive stress due to bending in the web

$$\sigma_h = -\frac{M\cdot h_w}{2\cdot I_x}$$

Horizontal force due to bending stress in lower flange

$$F_h = \frac{M\cdot c\cdot A_f}{I_x}$$

$$A = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta) + \frac{V - F_v}{A_w} \cdot \sin(-2 \cdot \theta) \right)^2$$

$$B = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta) + \frac{V - F_v}{A_w} \cdot \sin(-2 \cdot \theta) \right)$$

$$C = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta + \pi) + \frac{V - F_v}{A_w} \cdot \sin(-2 \cdot \theta + \pi) - \frac{M \cdot c \cdot A_f}{I_x \cdot t_w \cdot r \cdot \cos(\theta)} \right)$$

$$D = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta + \pi) + \frac{V - F_v}{A_w} \cdot \sin(-2 \cdot \theta + \pi) - \frac{M \cdot c \cdot A_f}{I_x \cdot t_w \cdot r \cdot \cos(\theta)} \right)^2$$

$$E = 3 \cdot \left(\frac{M \cdot h_w}{4 \cdot I_x} \cdot \sin(-2 \cdot \theta) + \frac{V - F_v}{A_w} \cdot \cos(-2 \cdot \theta) \right)^2$$

$$\sigma_{yield}^{\ 2} = A - B \cdot C + D + E$$

Since

$$F_v = F_h \cdot \tan(\theta) = \frac{M \cdot c \cdot A_f}{I_x} \cdot \tan(\theta)$$

$$A = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta) + \frac{V - \frac{M \cdot c \cdot A_f}{I_x} \cdot \tan(\theta)}{A_w} \cdot \sin(-2 \cdot \theta) \right)^2$$

$$B = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta) + \frac{V - \frac{M \cdot c \cdot A_f}{I_x} \cdot \tan(\theta)}{A_w} \cdot \sin(-2 \cdot \theta) \right)$$

$$C = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta + \pi) + \frac{V - \frac{M \cdot c \cdot A_f}{I_x} \cdot \tan(\theta)}{A_w} \cdot \sin(-2 \cdot \theta + \pi) - \frac{M \cdot c \cdot A_f}{I_x \cdot t_w \cdot r \cdot \cos(\theta)} \right)$$

$$D = \left(-\frac{M \cdot h_w}{4 \cdot I_x} - \frac{M \cdot h_w}{4 \cdot I_x} \cdot \cos(-2 \cdot \theta + \pi) + \frac{V - \frac{M \cdot c \cdot A_f}{I_x} \cdot \tan(\theta)}{A_w} \cdot \sin(-2 \cdot \theta + \pi) - \frac{M \cdot c \cdot A_f}{I_x \cdot t_w \cdot r \cdot \cos(\theta)} \right)^2$$

$$E = 3 \cdot \left(\frac{M \cdot h_w}{4 \cdot I_x} \cdot \sin(-2 \cdot \theta) + \frac{V - \frac{M \cdot c \cdot A_f}{I_x} \cdot \tan(\theta)}{A_w} \cdot \cos(-2 \cdot \theta) \right)^2$$

$$\sigma_{yield}^{\ 2} = A - B \cdot C + D + E$$

This equation was verified in another Mathcad sheet. (shear curved bottom flange RJN.xmcd)

Solve for V

$$A = \frac{1}{2 \cdot \left[(-12) + 12 \cdot \sin(\theta)^2 \right]} \qquad B = (-24) \cdot t_w \cdot r \cdot \cos(\theta) \cdot M \cdot c \cdot A_f \sin(\theta) \qquad C = 24 \cdot \sin(\theta)^3 \cdot A_w \cdot M \cdot c \cdot A_f$$

$$D = 24 \cdot \sin(\theta) \cdot A_w \cdot M \cdot c \cdot A_f \qquad E = 36 \cdot t_w^{\ 2} \cdot r^2 \cdot \cos(\theta)^2 \cdot M^2 \cdot c^2 \cdot A_f^{\ 2} \cdot \sin(\theta)^2$$

$$F = 36 \cdot \sin(\theta)^6 \cdot A_w^{\ 2} \cdot M^2 \cdot c^2 \cdot A_f^{\ 2} \qquad G = 72 \cdot \sin(\theta)^4 \cdot A_w^{\ 2} \cdot M^2 \cdot c^2 \cdot A_f^{\ 2} + 48 \cdot \sin(\theta)^2 \cdot A_w^{\ 2} \cdot M^2 \cdot c^2 \cdot A_f^{\ 2}$$

$$H = 24 \cdot M^2 \cdot h_w \cdot A_w^{\ 2} \cdot t_w \cdot r \cdot \cos(\theta) \cdot c \cdot A_f \sin(\theta)^2 - 36 \cdot t_w^{\ 2} \cdot r^2 \cdot M^2 \cdot c^2 \cdot A_f^{\ 2} \cdot \sin(\theta)^2 - 3 \cdot t_w^{\ 2} \cdot r^2 \cdot M^2 \cdot h_w^{\ 2} \cdot A_w^{\ 2}$$

$$I = 6 \cdot \sin(\theta)^2 \cdot t_w^{\ 2} \cdot r^2 \cdot M^2 \cdot h_w^{\ 2} \cdot A_w^{\ 2} + 12 \cdot \sigma_{yield}^{\ 2} \cdot I_x^{\ 2} \cdot A_w^{\ 2} \cdot t_w^{\ 2} \cdot r^2 - 24 \cdot \sigma_{yield}^{\ 2} \cdot I_x^{\ 2} \cdot A_w^{\ 2} \cdot t_w^{\ 2} \cdot r^2 \cdot \sin(\theta)^2$$

$$J = 12 \cdot M^2 \cdot c^2 \cdot A_f^{\ 2} \cdot A_w^{\ 2} + 6 \cdot t_w \cdot r \cdot \cos(\theta) \cdot M^2 \cdot h_w \cdot A_w^{\ 2} \cdot c \cdot A_f + 18 \cdot t_w \cdot r \cdot \cos(\theta) \cdot M^2 \cdot h_w \cdot A_w^{\ 2} \cdot \sin(\theta)^4 \cdot c \cdot A_f$$

$$K = 18 \cdot t_w \cdot r \cdot \cos(\theta) \cdot M^2 \cdot h_w \cdot A_w^{\ 2} \cdot \sin(\theta)^4 \cdot c \cdot A_f + 36 \cdot t_w^{\ 2} \cdot r^2 \cdot M^2 \cdot c^2 \cdot A_f^{\ 2} \cdot \sin(\theta)^4$$

$$L = 3 \cdot t_w^{\ 2} \cdot r^2 \cdot M^2 \cdot h_w^{\ 2} \cdot A_w^{\ 2} \cdot \sin(\theta)^4 + 12 \cdot \sin(\theta)^4 \cdot \sigma_{yield}^{\ 2} \cdot I_x^{\ 2} \cdot A_w^{\ 2} \cdot t_w^{\ 2} \cdot r^2$$

$$M = r \cdot t_w \cdot I_x$$

$$V = \begin{bmatrix} A \cdot \left[\dfrac{B + C - D + 4 \cdot (E + F - G - H + I - J + K - L)^{\frac{1}{2}}}{M} \right] \\ A \cdot \left[\dfrac{B + C - D + 4 \cdot (E + F - G - H + I - J + K - L)^{\frac{1}{2}}}{M} \right] \end{bmatrix}$$

$$V = \begin{pmatrix} -317 \\ 321 \end{pmatrix} kip$$

Use the positive root

$V := V_1$ $\qquad\qquad\qquad$ $V = 321 kip$

Appendix N. HAUNCHED SECTION PLASTIC SHEAR CAPACITY RANGE

The following sections present the plastic shear capacity ranges for “stepped” elements modeling haunched girders. Consistent with the figures presented in Section 3, negative moments are reported as positive values. Conversely, positive moments are reported as negative values.

6.1 N.1 Haunched South Approach Girder Sections

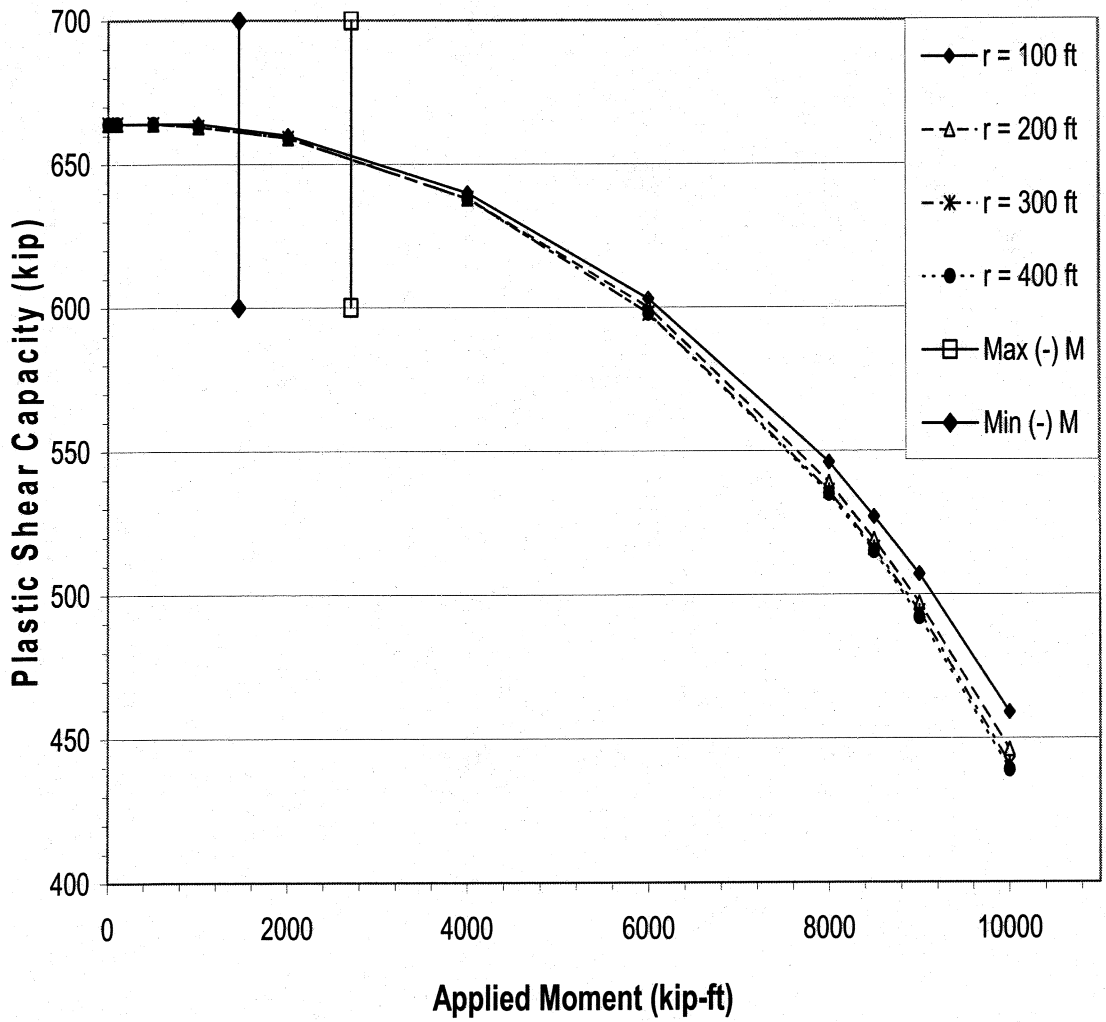


Figure N - 1: Plastic Shear Capacity Range South Approach Girder 1

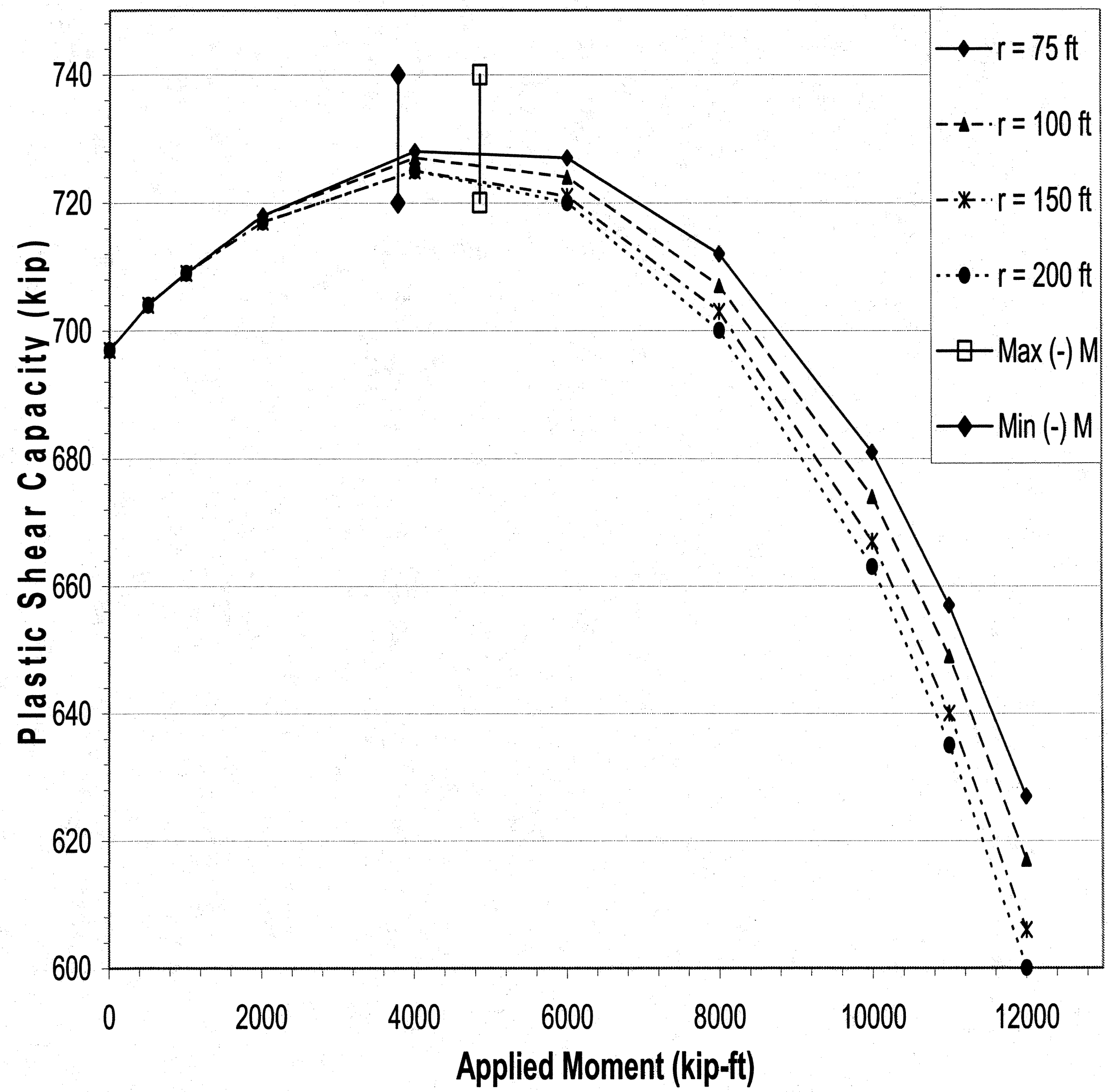


Figure N - 2: Plastic Shear Capacity Range South Approach Girder 2

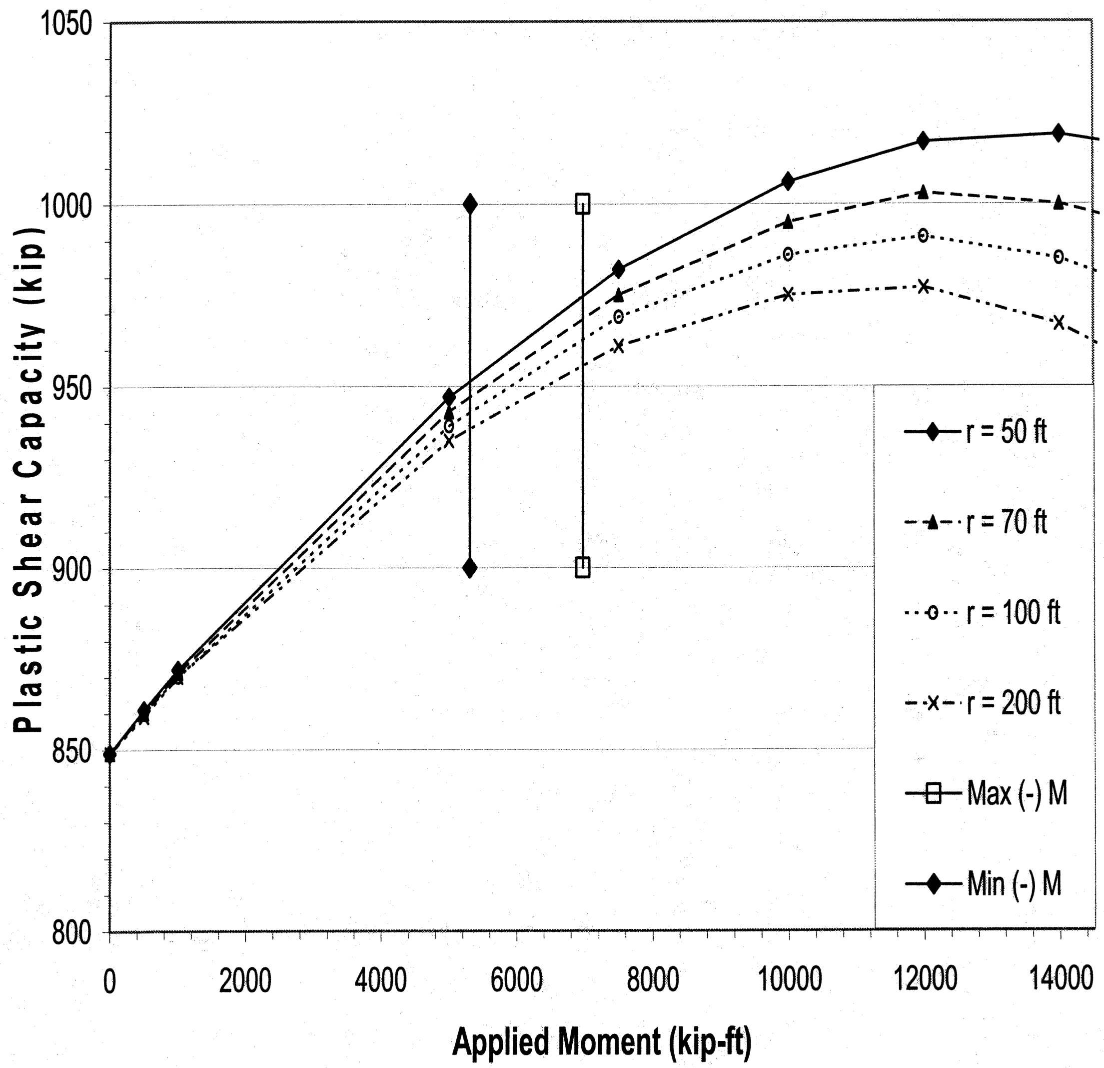


Figure N - 3: Plastic Shear Capacity Range South Approach Girder 3

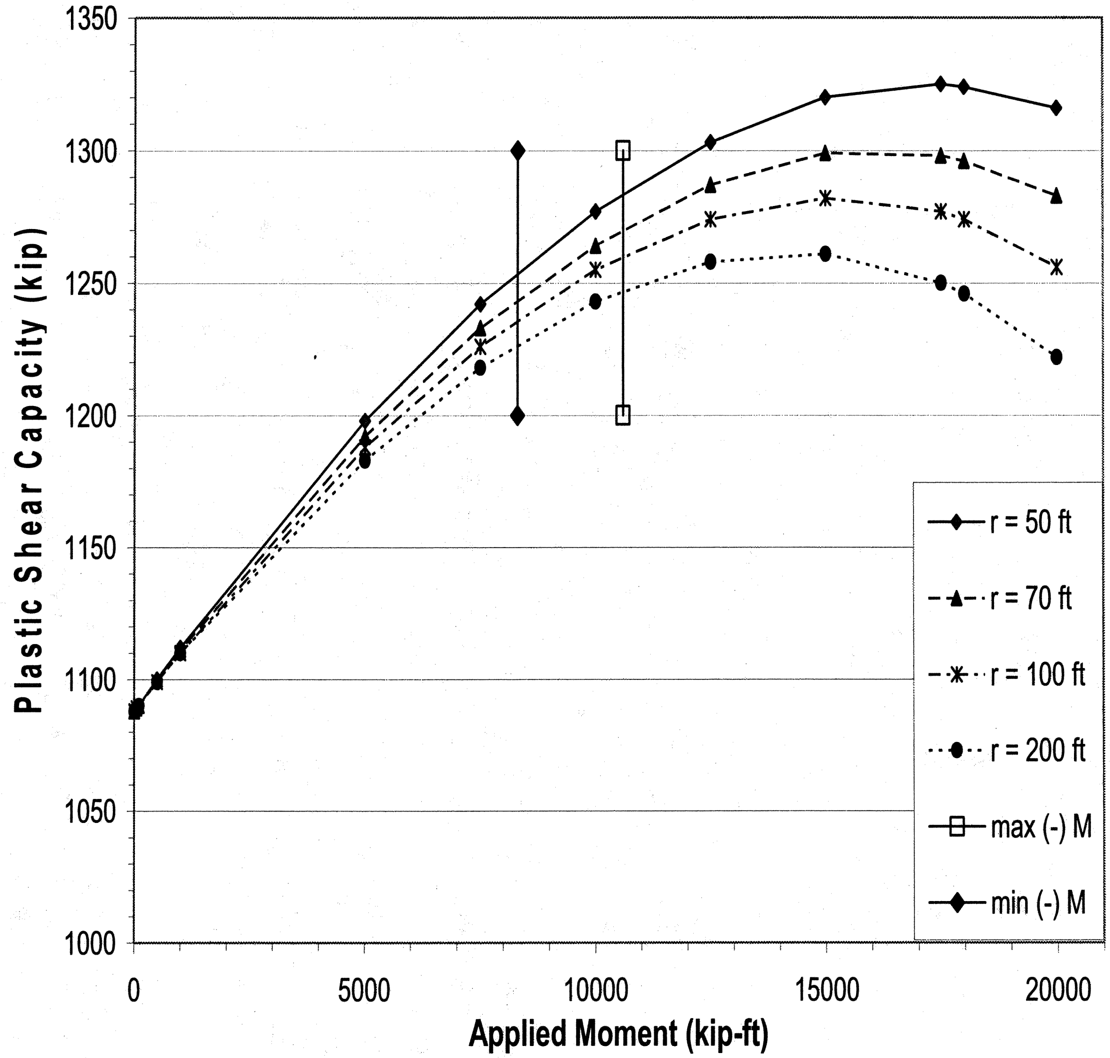


Figure N - 4: Plastic Shear Capacity Range South Approach Girder 4

6.2 N.2 Haunched North Approach Girder Sections

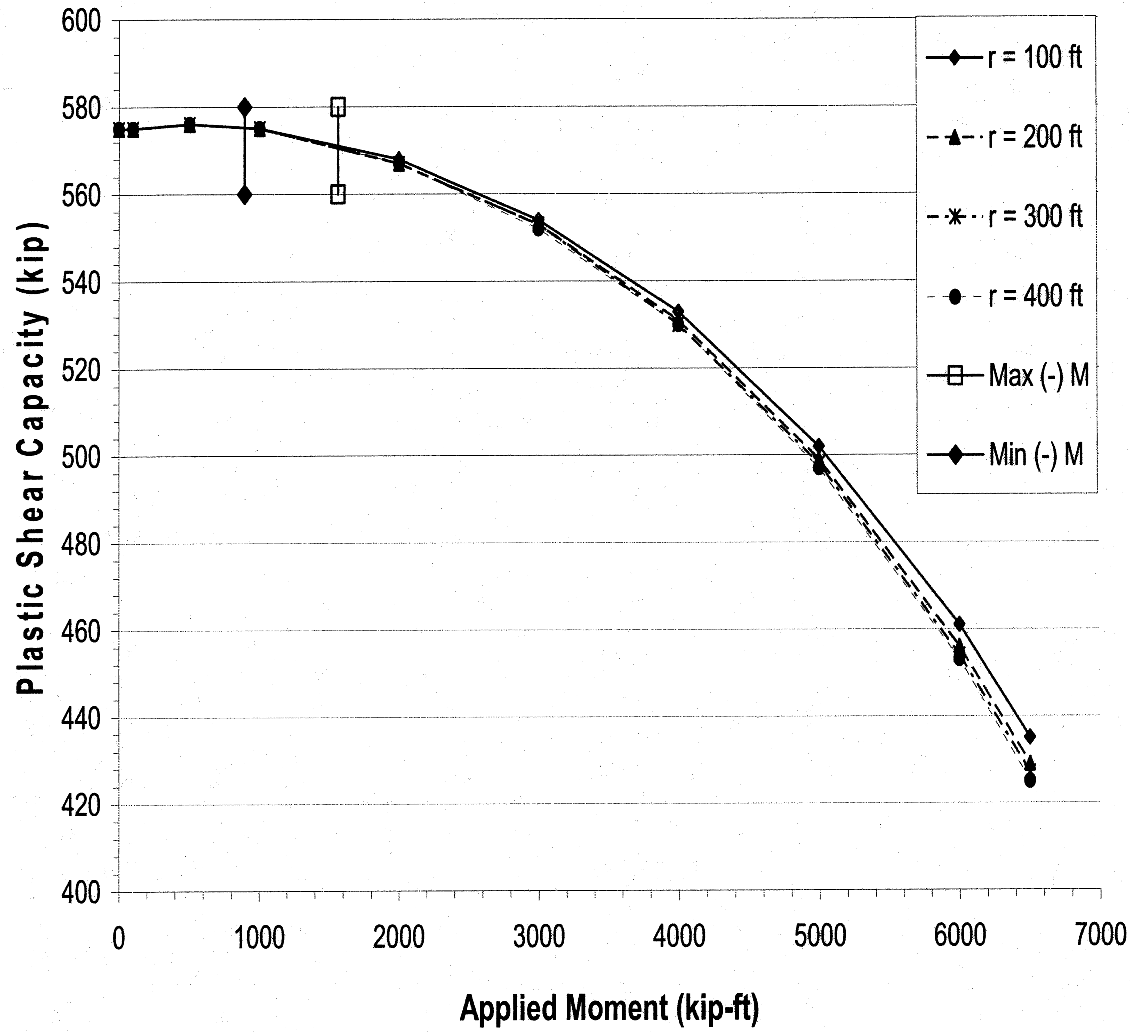


Figure N - 5: Plastic Shear Capacity Range North Approach Girder 1

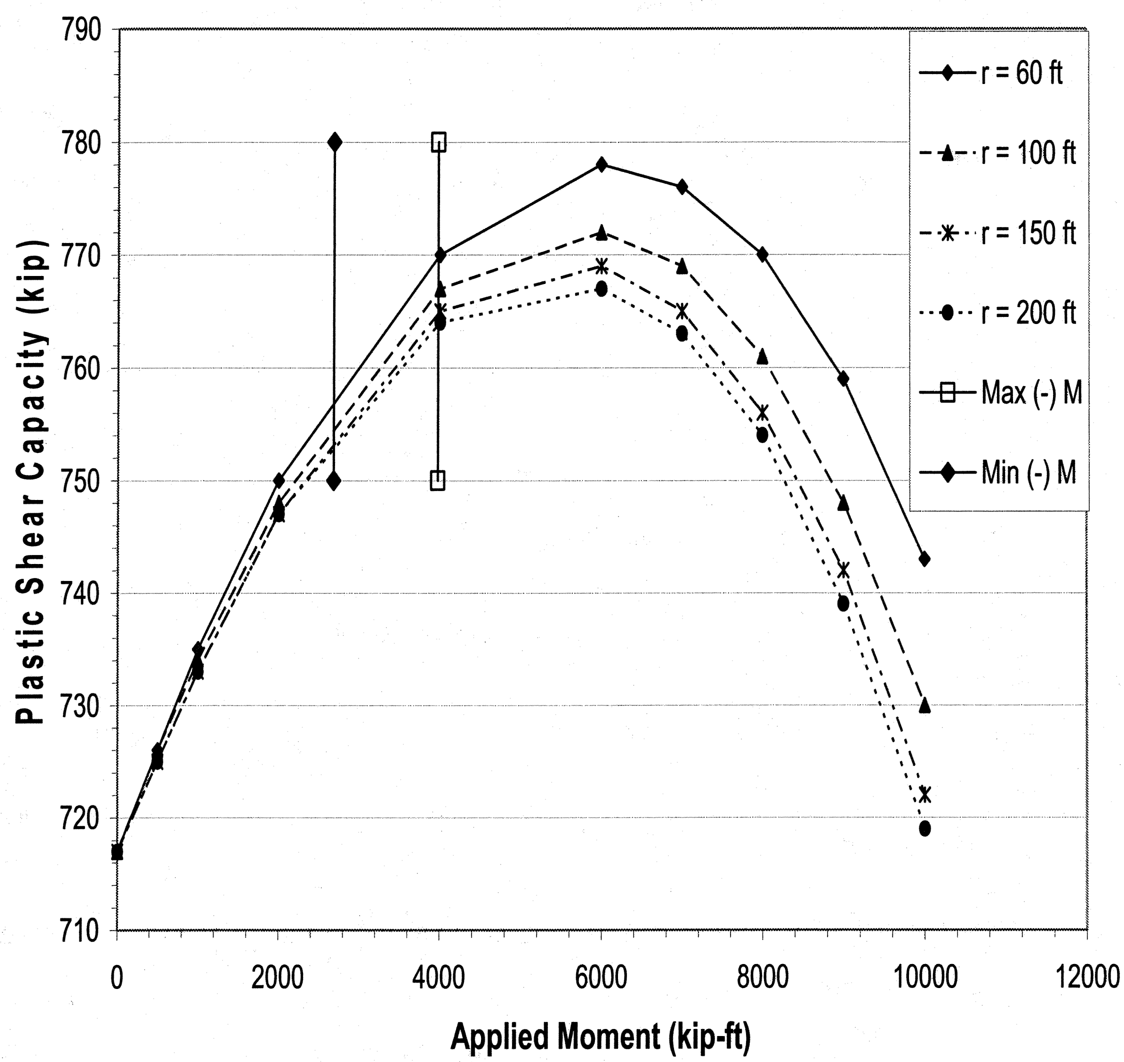


Figure N - 6: Plastic Shear Capacity Range North Approach Girder 2

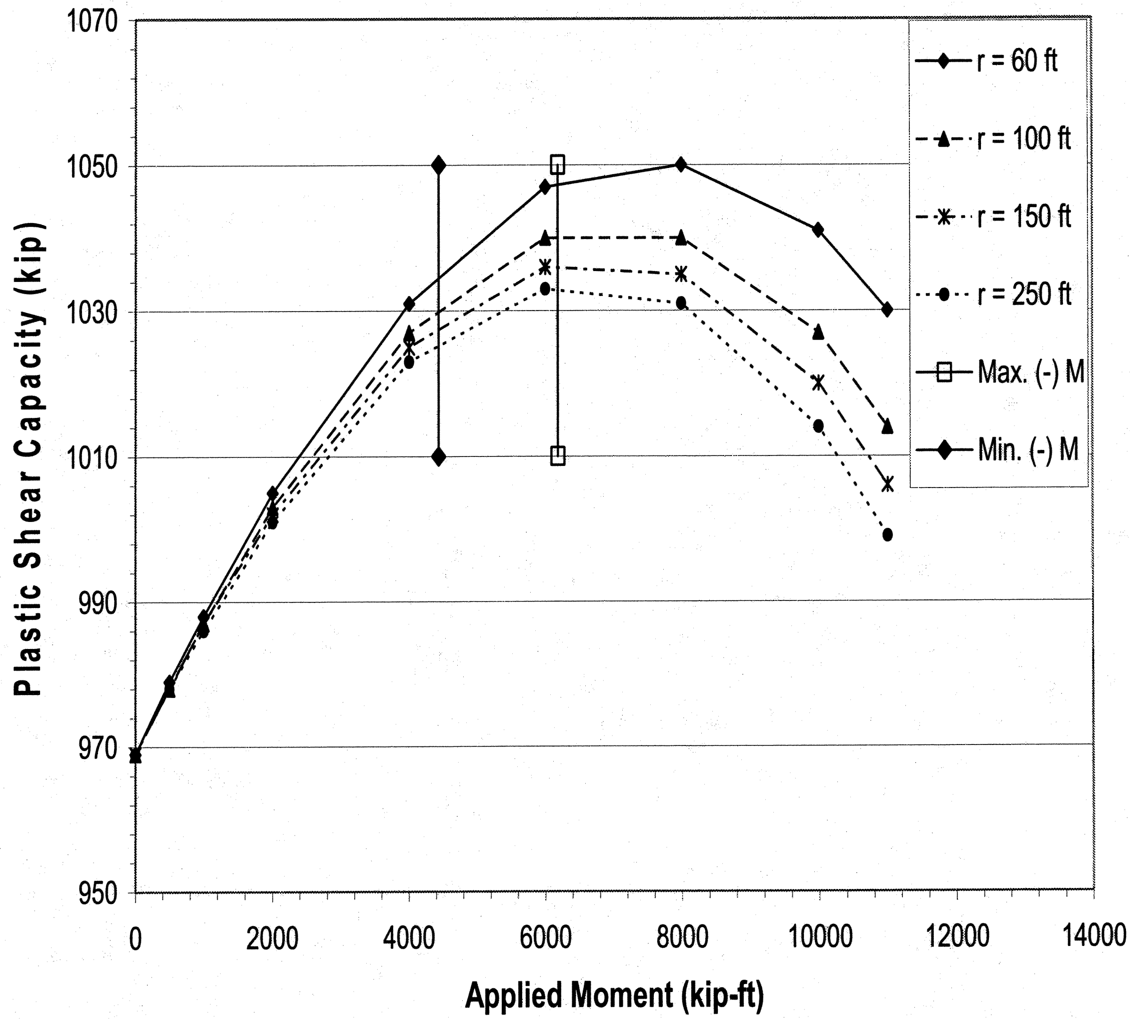


Figure N - 7: Plastic Shear Capacity Range North Approach Girder 3

6.3 N.3 South Expansion Joint Girder Sections

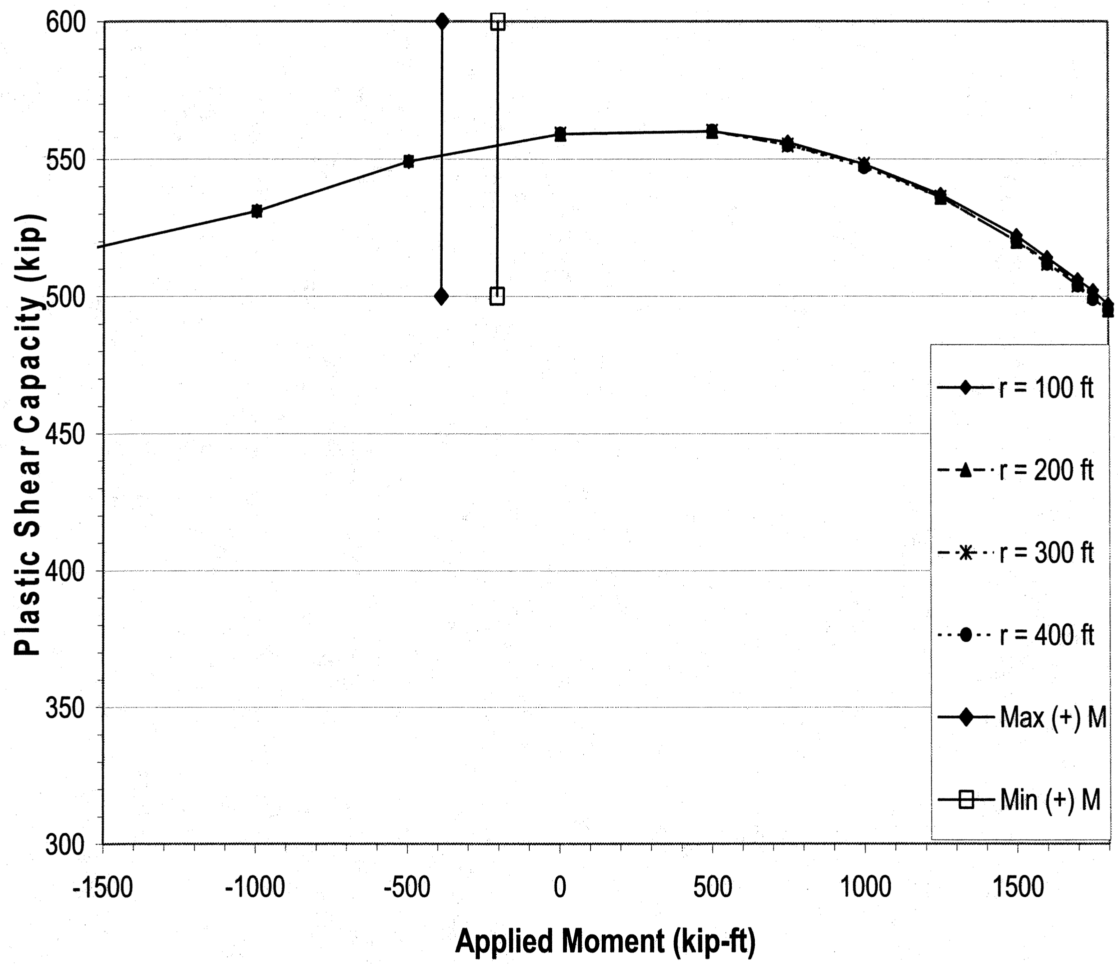


Figure N - 8: Plastic Shear Capacity Range Ext. South Expansion Joint Girder 2

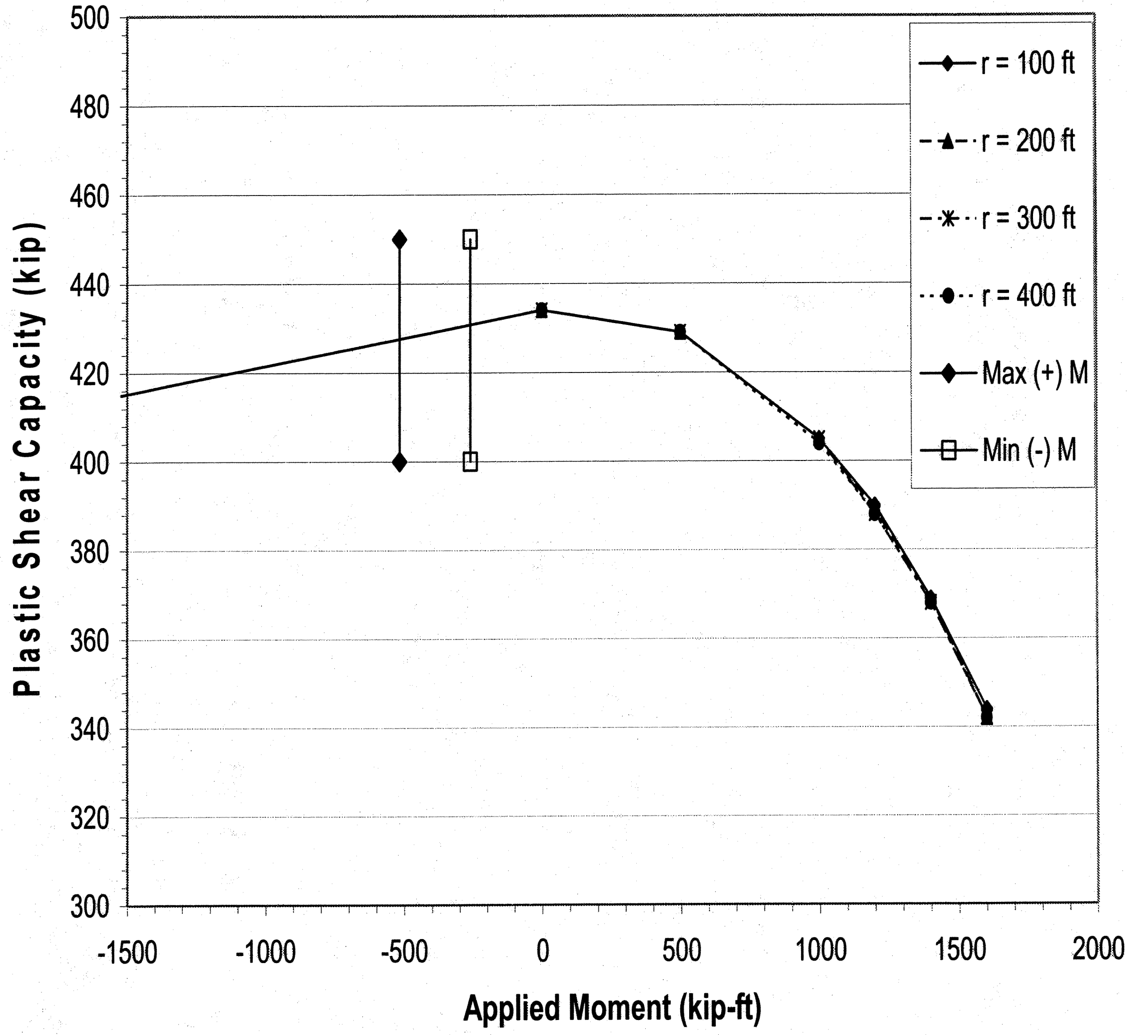


Figure N - 9: Plastic Shear Capacity Range Ext. South Expansion Joint Girder 3

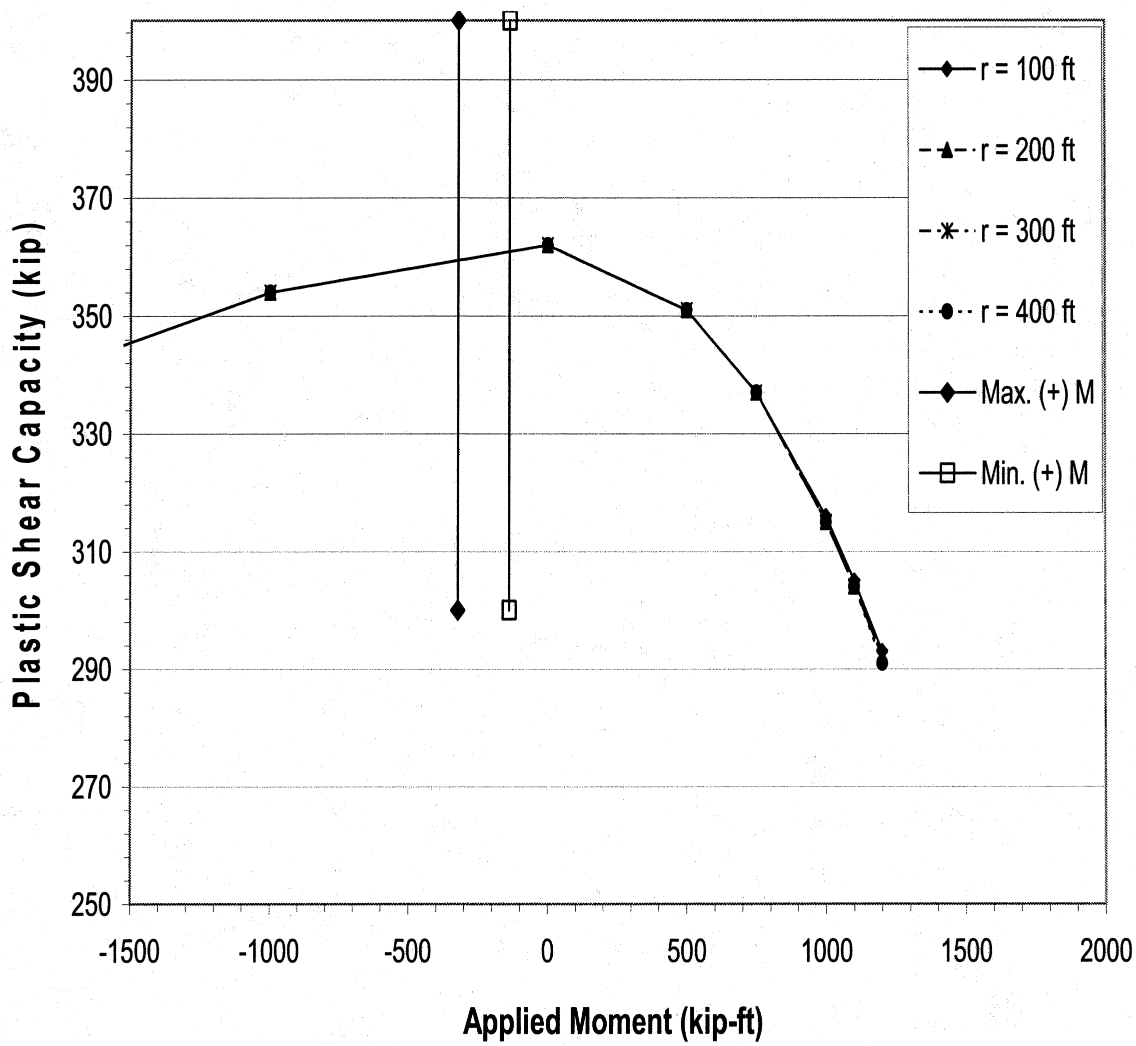


Figure N - 10: Plastic Shear Capacity Range Ext. South Expansion Joint Girder 4

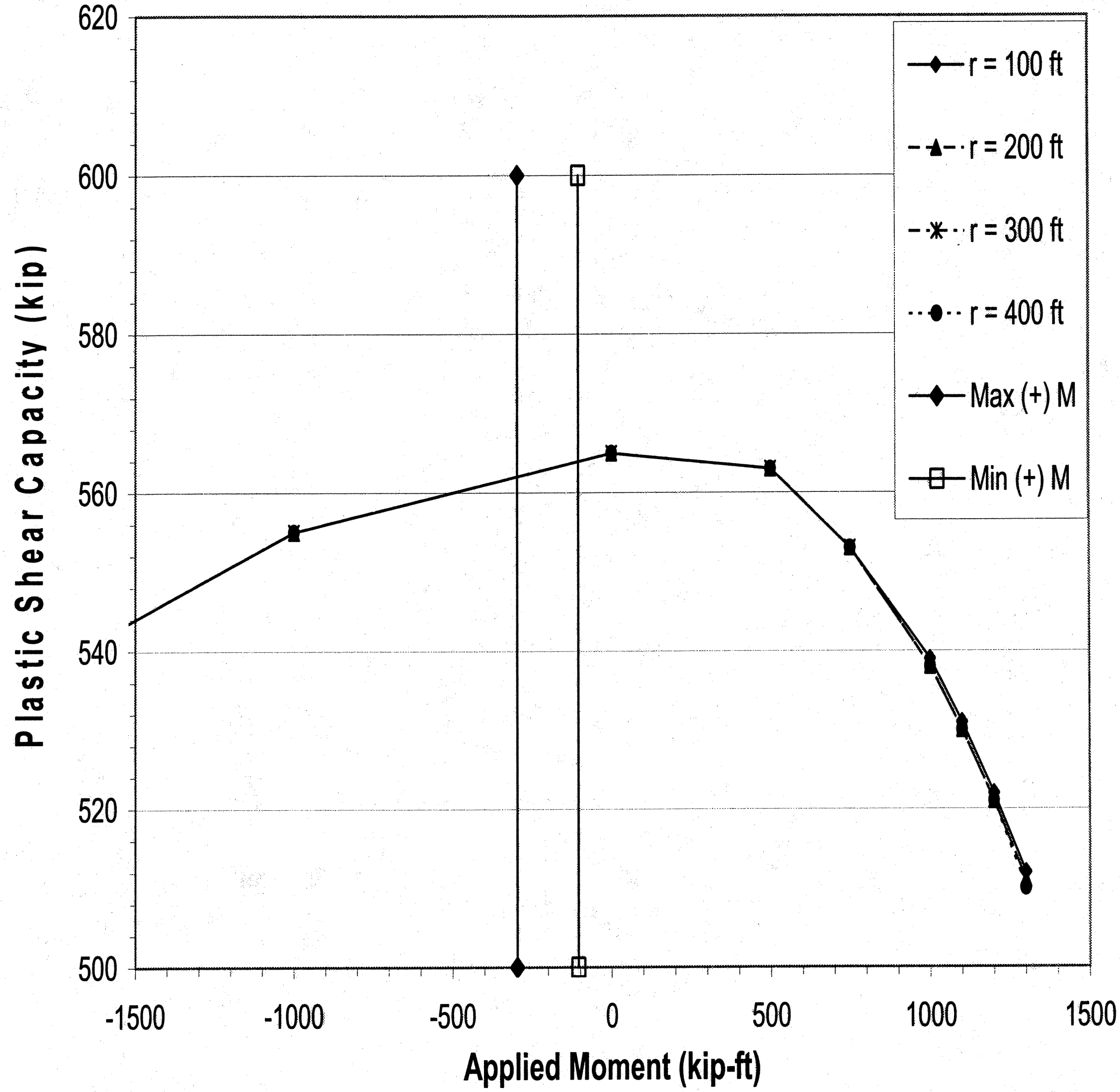


Figure N - 11: Plastic Shear Capacity Range Int. South Expansion Joint Girder 2

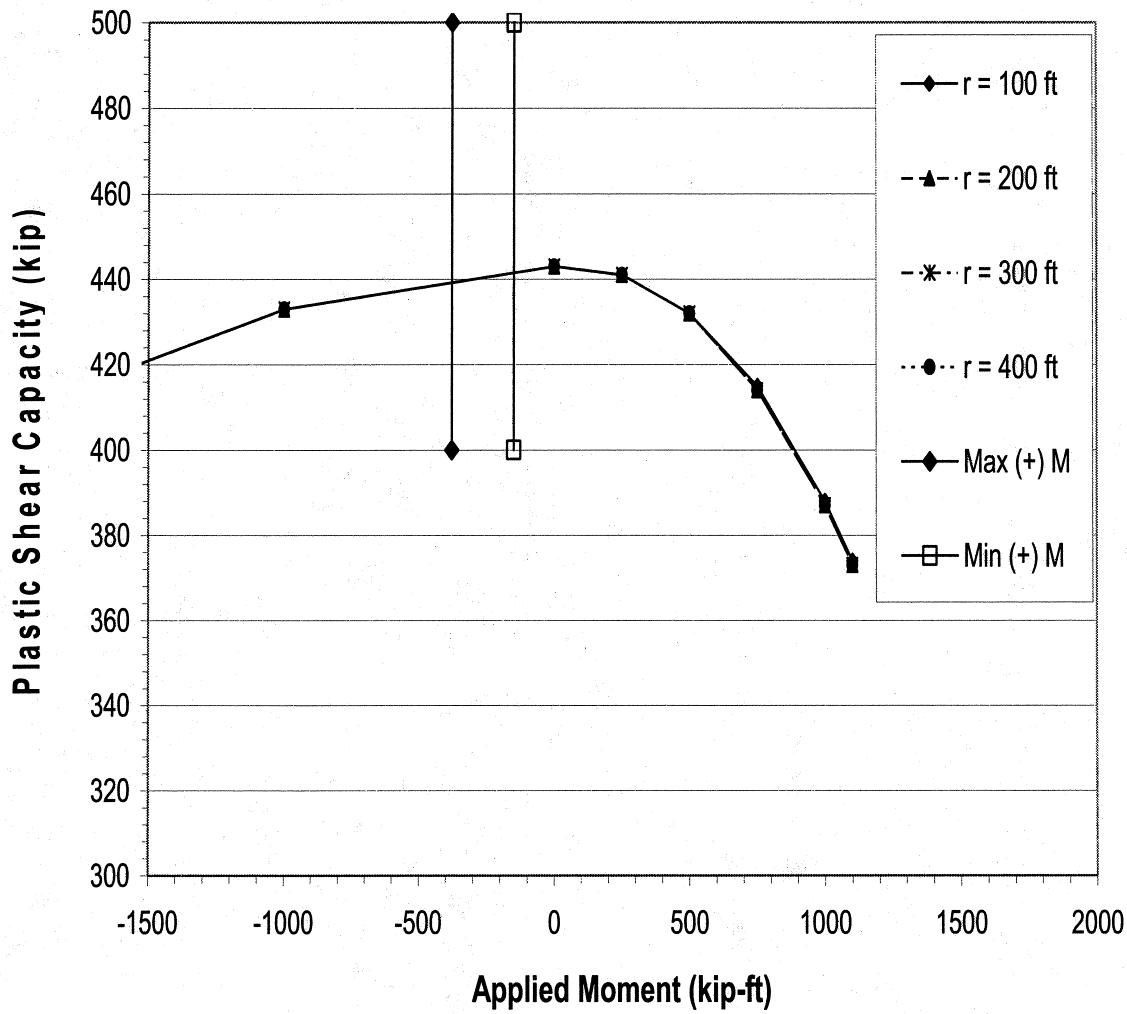


Figure N - 12: Plastic Shear Capacity Range Int. South Expansion Joint Girder 3

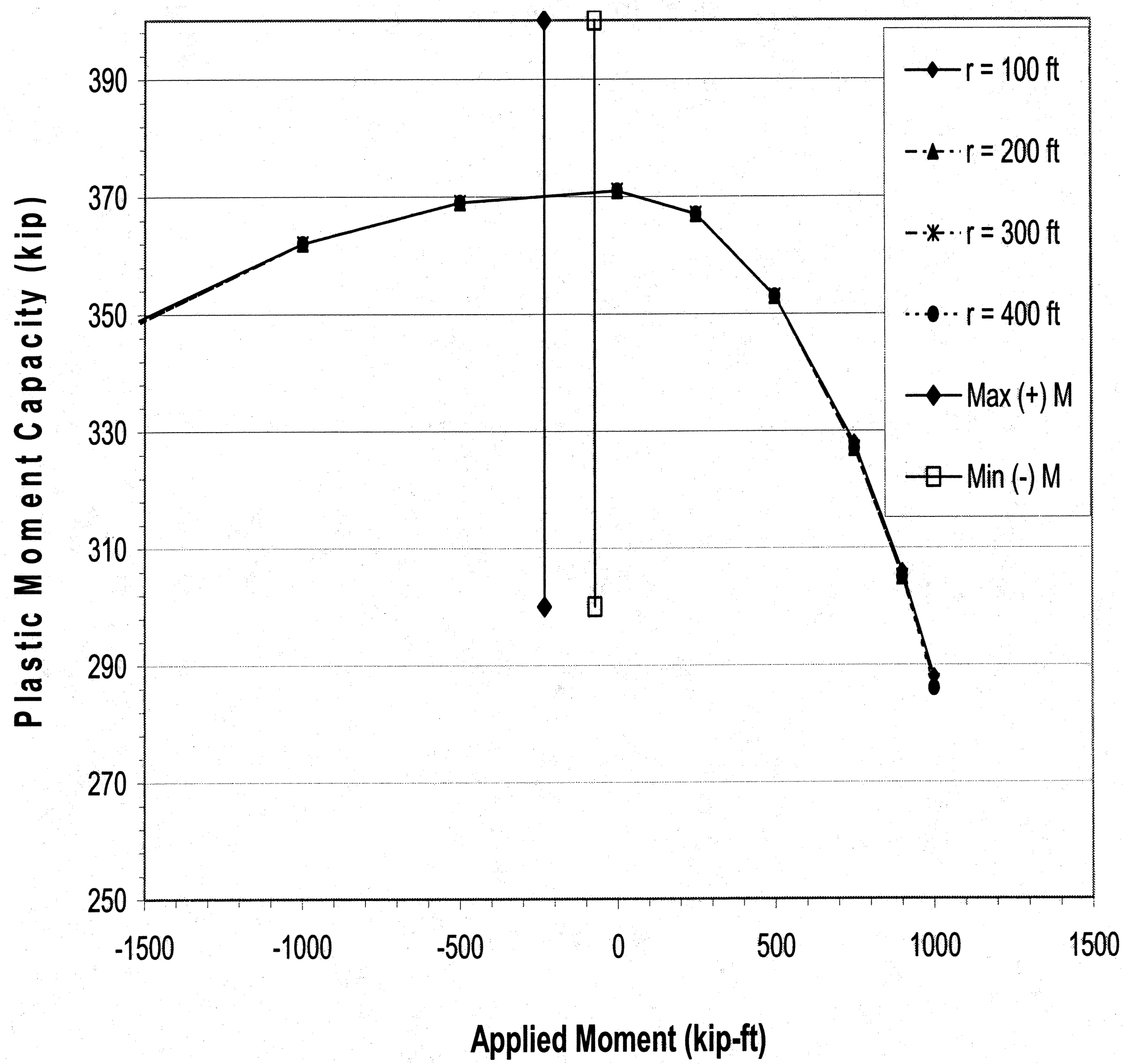


Figure N - 13: Plastic Shear Capacity Range Int. South Expansion Joint Girder 4

6.4 N.4 North Expansion Joint Girder Sections

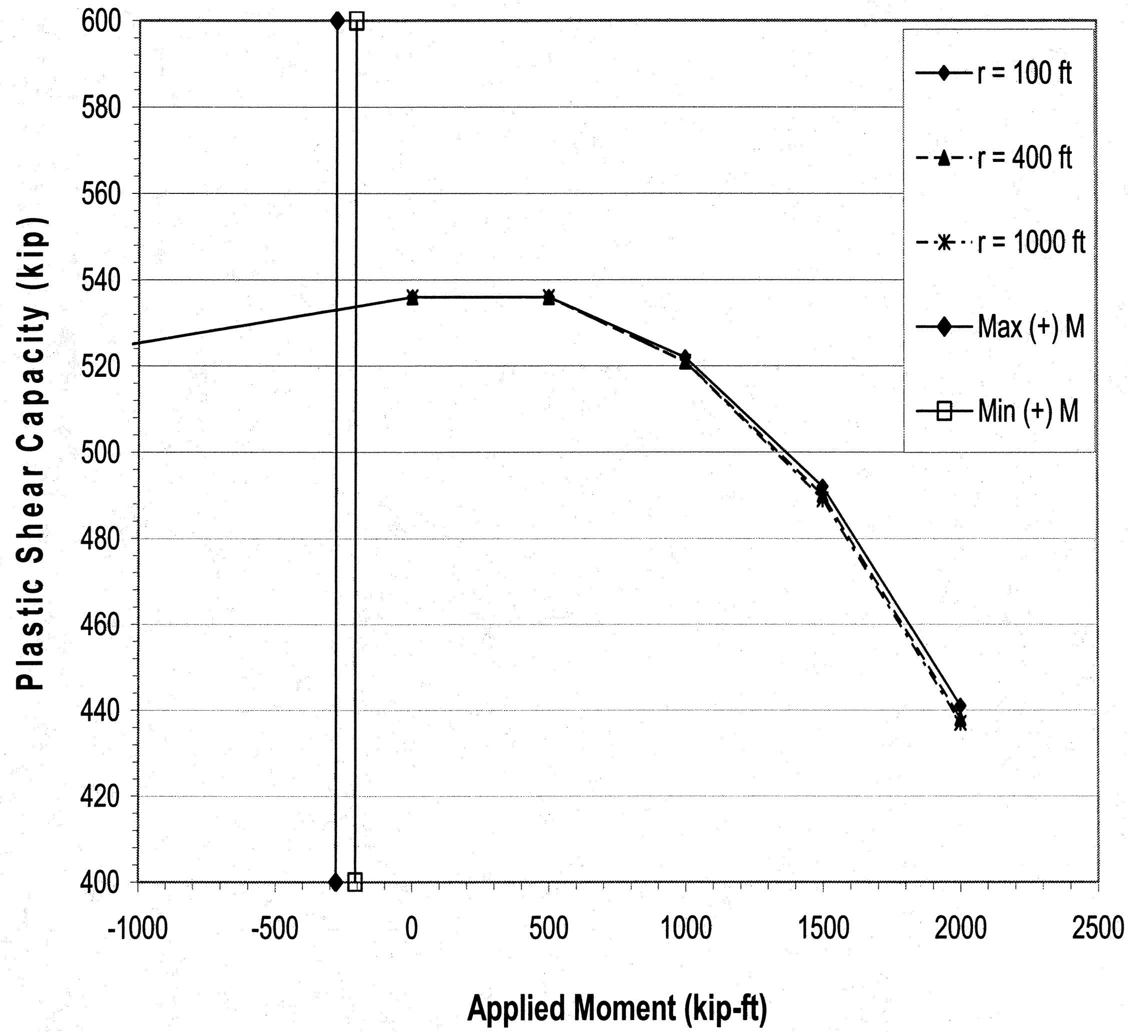


Figure N - 14: Plastic Shear Capacity Range Ext. North Expansion Joint Girder 2

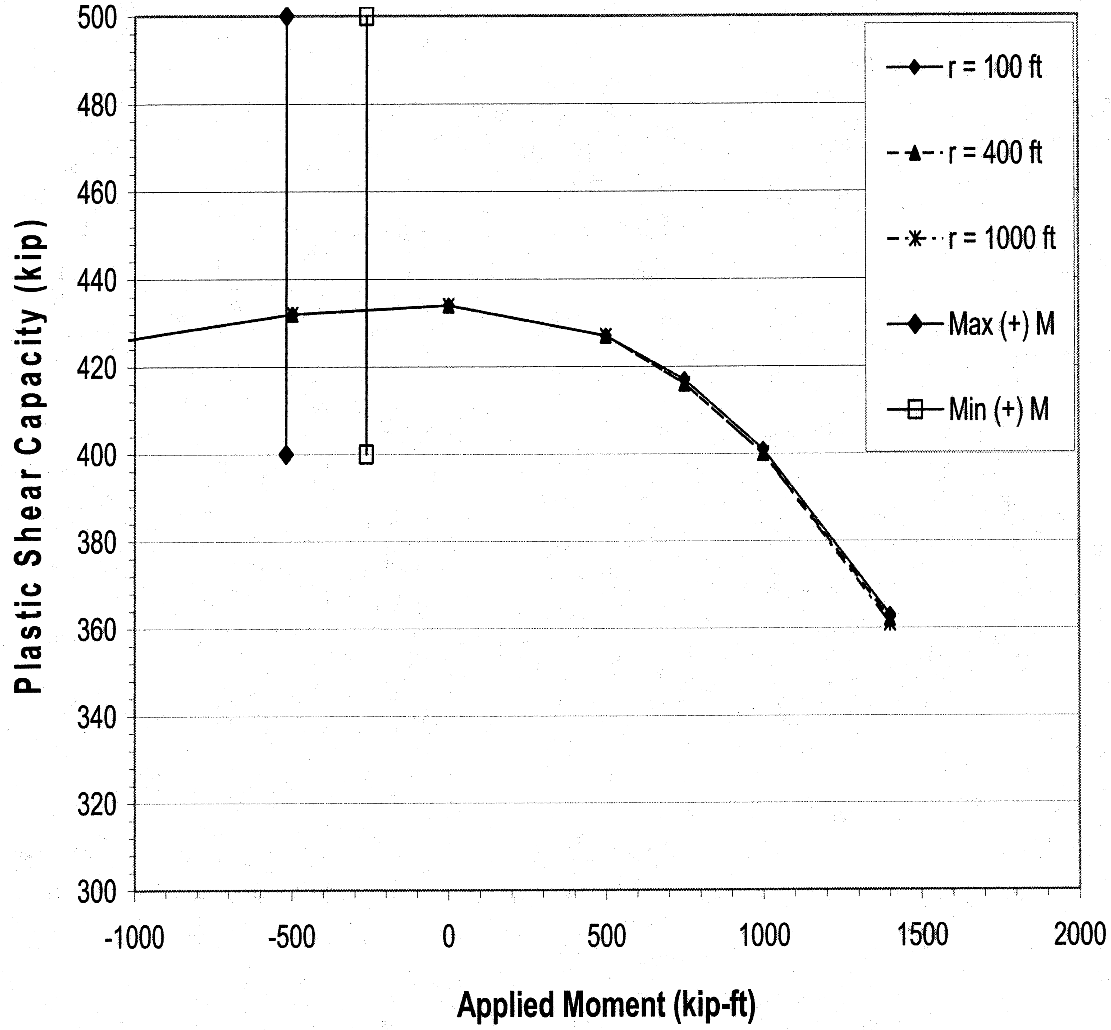


Figure N - 15: Plastic Shear Capacity Range Ext. North Expansion Joint Girder 3

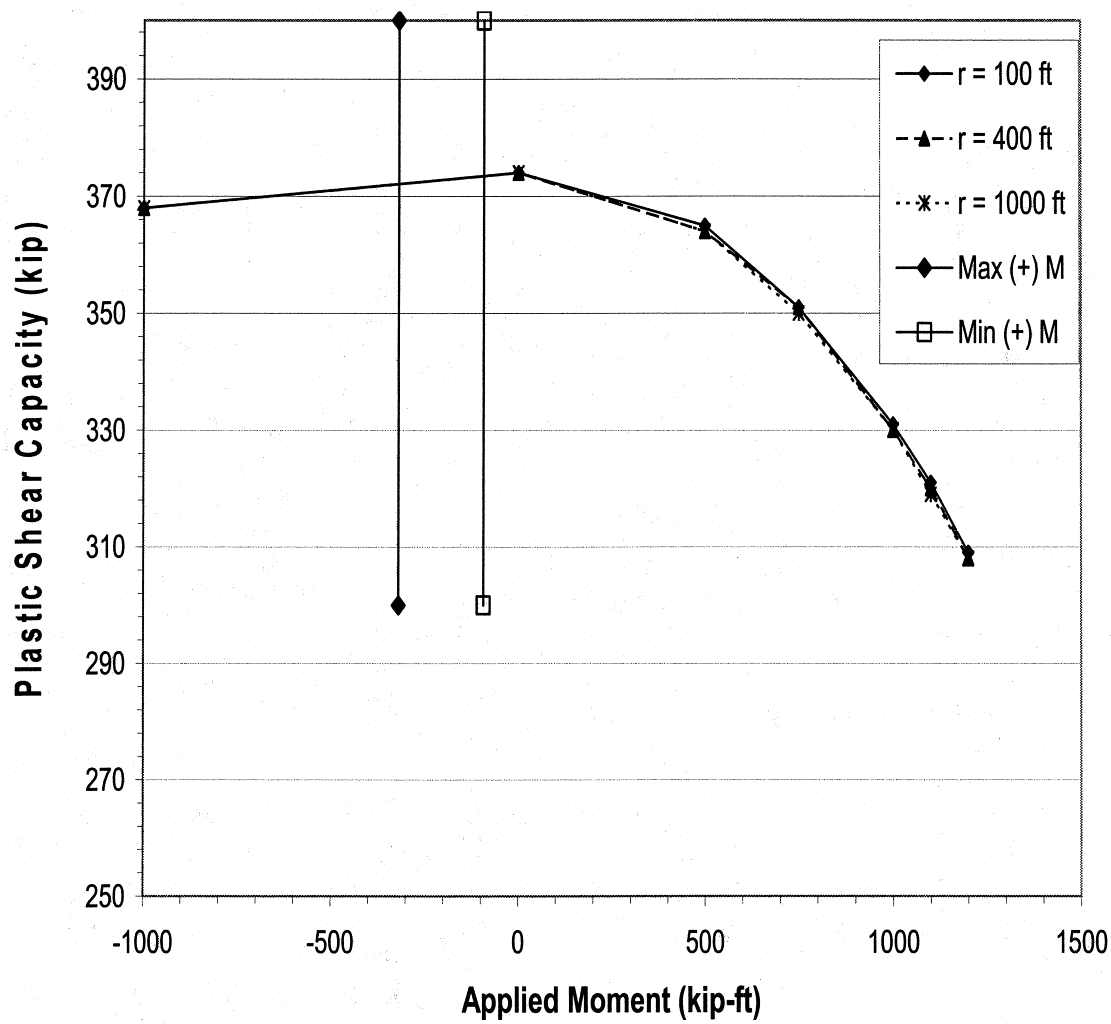


Figure N - 16: Plastic Shear Capacity Range Ext. North Expansion Joint Girder 4

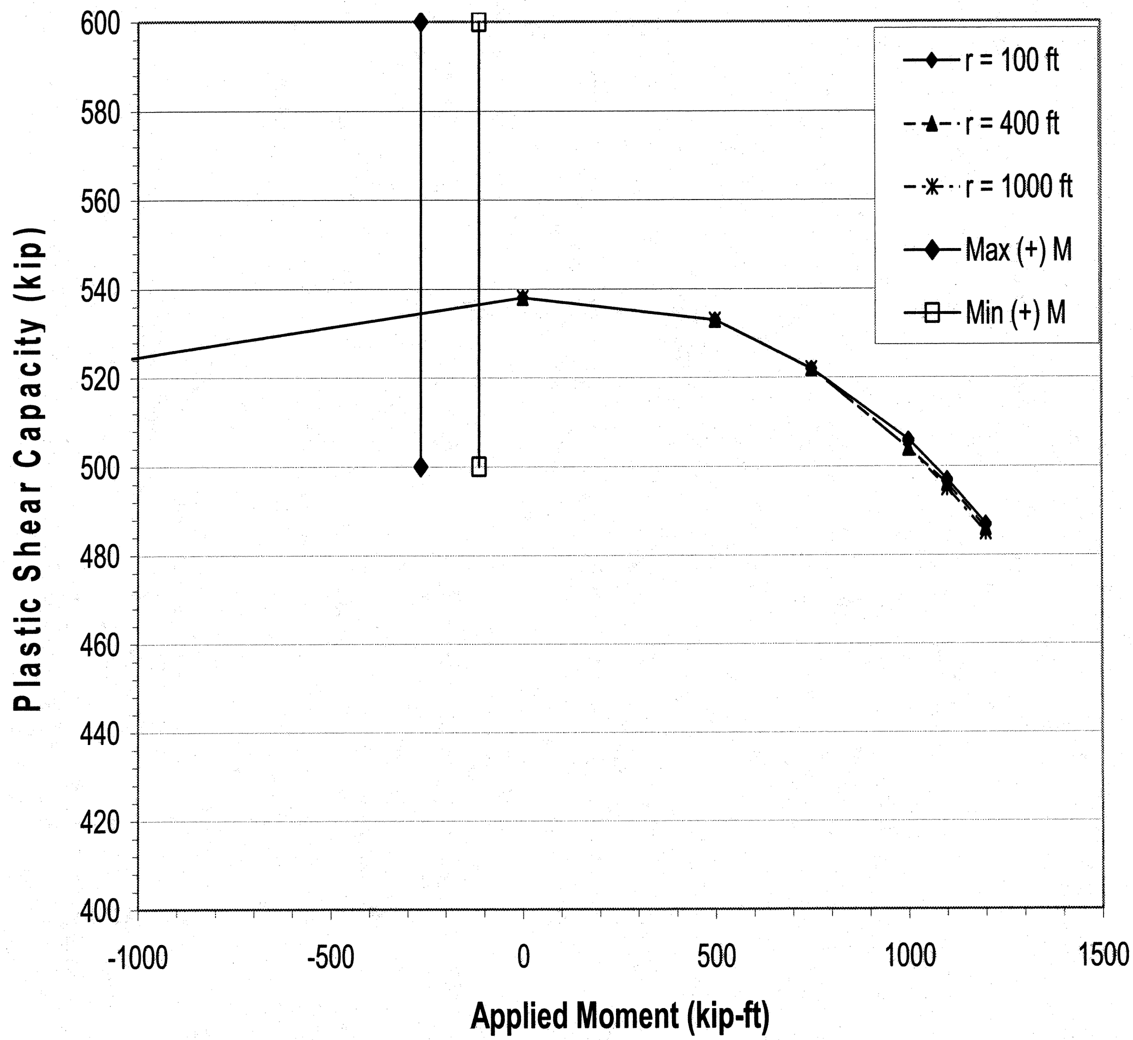


Figure N - 17: Plastic Shear Capacity Range Int. North Expansion Joint Girder 2

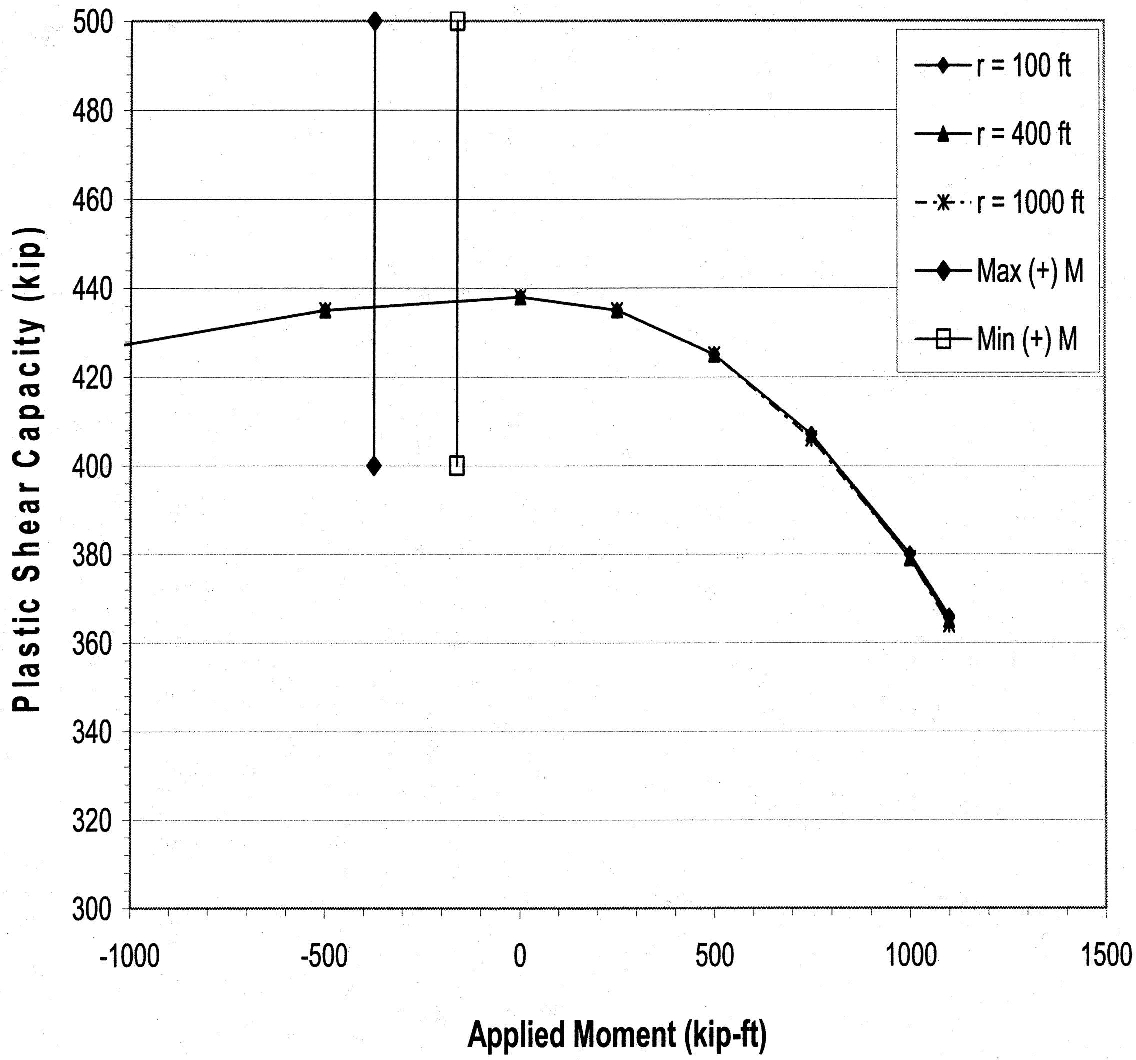


Figure N - 18: Plastic Shear Capacity Range Int. North Expansion Joint Girder 3

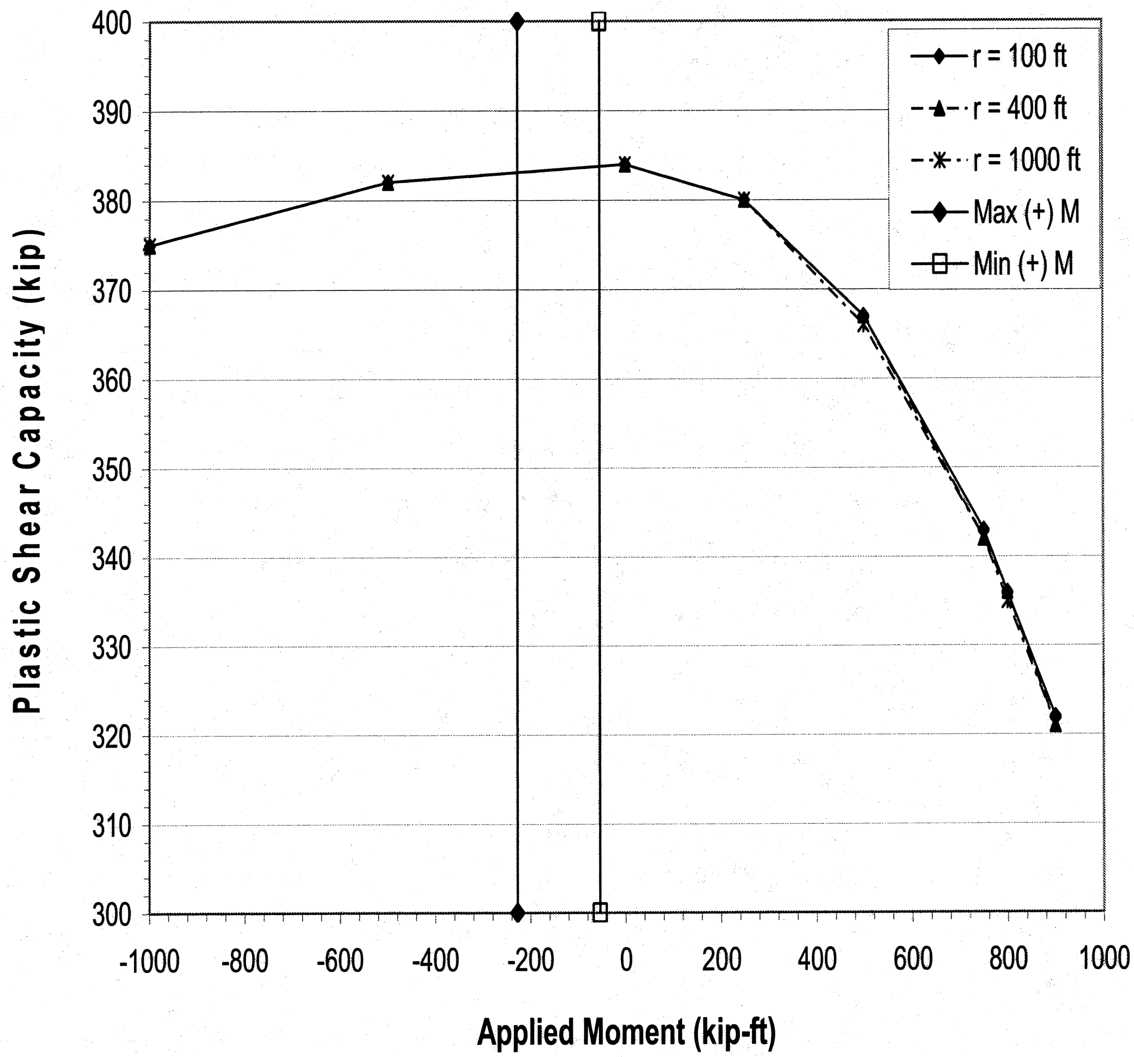


Figure N - 19: Plastic Shear Capacity Range Int. North Expansion Joint Girder 4

Appendix O. PROCEDURE FOR BASIC LRFR ANALYSIS

This appendix describes how to perform a basic load rating analysis. Performing a basic load rating analysis involves: (1) performing a moving load analysis using LARSA 2000 Plus v6.08.65; (2) running the Excel LRFR load rating spreadsheet; and (3) interpreting the results of the LRFR load rating analysis. This is a basic LRFR analysis for predefined load patterns and LRFR rating.

The basic LRFR analysis considers a single trip permit vehicle traveling in mixed traffic at posted speed limits. All structural components are inspected and satisfy good condition status. If the vehicle load pattern for a permit vehicle is not already included in the moving load database, it will need to be entered. Entering vehicle load patterns is described in Appendix E. If the vehicle permit is not a single trip traveling in mixed traffic; if the vehicle travels less than ten miles an hour; or if structural components are in a condition other than satisfactory, the LRFR rating parameters must be modified. Adjusting LRFR rating parameters is described in Section 3.2.

O.1 Performing a Moving Analysis

LARSA's moving load analysis estimates live load demands for all structural members. The results from the moving load analysis are used by the Excel spreadsheet to calculate LRFR rating factors for all bridge members. The I.B. Perrine Bridge Model is created in LARSA 2000 Plus, version 06.08.65. Sections O.1.1 and O.1.2 describe how to run a moving load analysis and provide an example of a moving load analysis, respectively.

O.1.1 Instructions: Moving Load Analysis

1. Open LARSA 2000 Plus.
2. Open "PERRINE BRIDGE MASTER COPY" file.
    a. File → Open → Local Disk (D:) → Perrine Bride Project folder → "PERRINE BRIDGE MASTER COPY"
3. Under the Explorer Menu, located on the right side of the screen, select the **Load Explorer**.

4. In the Load Explorer, under Load Cases (not Load Combinations), double-click **vehicle lane load** and in the screen that appears, select the **Moving Loads** Tab.

Figure O - 1 shows the options for running a moving load analysis.

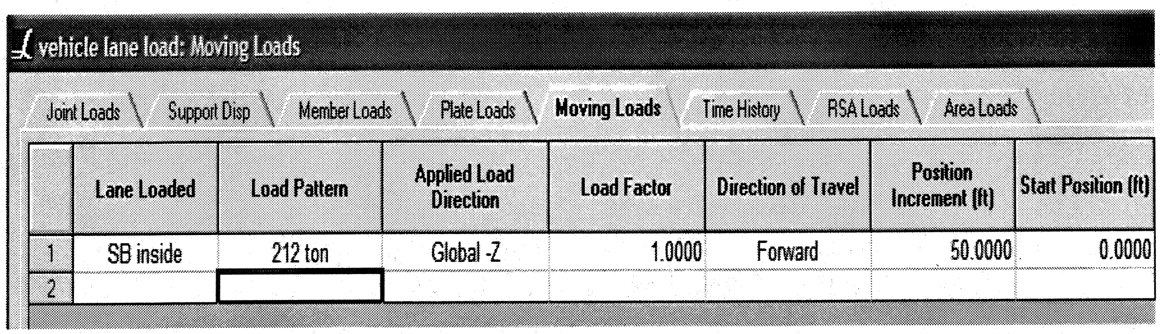


Figure O - 1: Options to Run a Moving Load Analysis

5. Under Lane Loaded column, select the lane that the vehicle will travel on the bridge. The permit should specify that the vehicle shall travel in the lane for which the moving load analysis is performed, or the rating should be performed for both lanes in the desired travel direction.

   a. There are five lanes for northbound (NB) travel and five lanes for south bound (SB) travel.

   b. Lanes labeled "outside" and "inside" correspond to the striped lanes on either side of the bridge deck. Permit vehicles will travel in striped lanes unless otherwise required to increase LRFR rating factors. ***For an initial analysis, choose a striped lane in the appropriate direction of travel, either north or southbound.***

   c. The lane labeled "middle" straddles the two striped lanes and can be used for extremely wide loads that will not fit in the striped lanes. Also, if an LRFR analysis is completed for travel in a striped lane and the LRFR factors are slightly less than adequate, the "middle" lane can be loaded to see if rating factors are adequate when the load proceeds along this travel path.

   d. dNB and dSB correspond to design lanes on either side of the bridge. These lanes are used to create worst case loading scenarios for model verification and will **NOT** be used for permit load rating. Design lanes follow typical AASHTO design requirements.

6. Under the Load Pattern column, select the vehicle load pattern that corresponds to the rating vehicle. If a vehicle load pattern needs to be added, see Appendix E.
7. Under the Position Increment column, select the interval at which moving loads will be generated.
   a. The default increment is set at 10.00 feet.
8. In the Analysis Menu, located at the top of the screen, select **Moving Load Analysis**.
   a. Click the Analyze button. This performs the moving load analysis.
   b. A prompt will appear that asks the user to save before running an analysis; select **Yes**. If No is selected the analysis will not run.

For a moving load analysis the user must select only the three options mentioned above to run a moving load analysis: loaded lane, load pattern, and position increment. All other options should remain unchanged from one moving analysis to another. The options that remain unchanged are the applied load direction (Global –Z), load factor (1.00), direction of travel (Forward), and start position (0.00).

O.1.2 Example: 212-Ton Vehicle Moving Load Analysis

The following example will demonstrate how a moving load analysis is performed for a 212-ton vehicle configuration that will be traveling southbound across the Perrine Bridge. The 212-ton vehicle configuration has already been created and is included in the load pattern database. This is the first moving load analysis for this vehicle configuration.

Once the “PERRINE BRIDGE MASTER COPY” file is open, select the **Load Explorer**. Figure O - 2 shows where the Load Explorer is located.

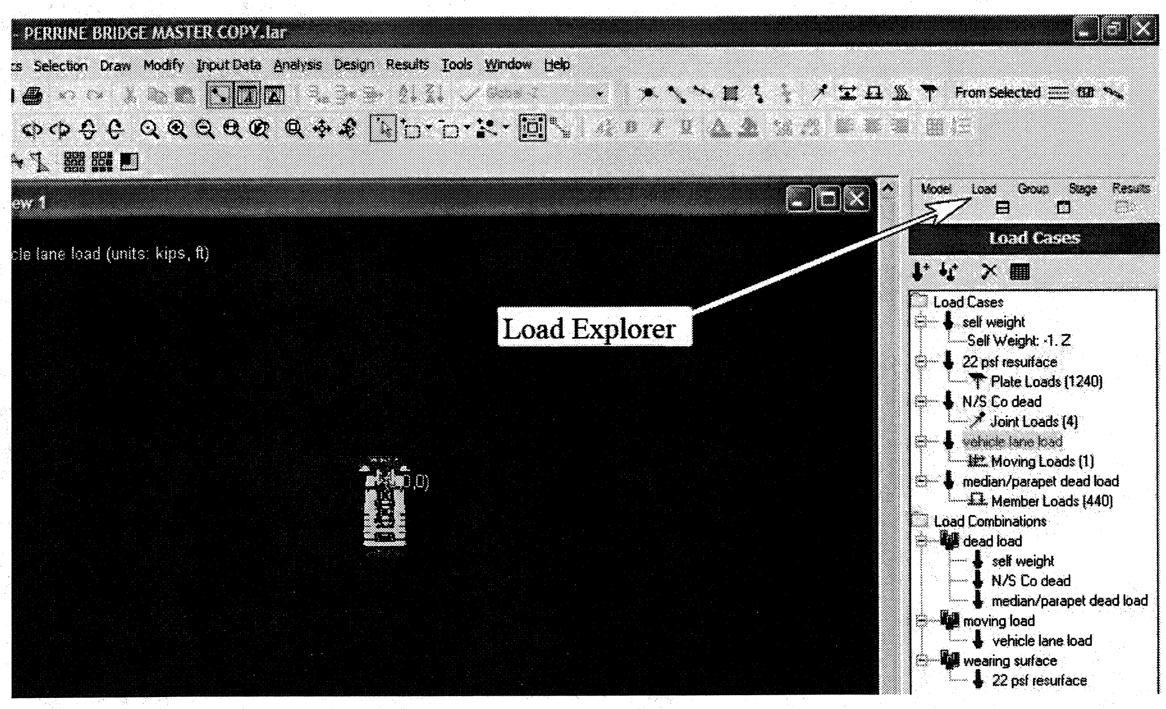


Figure O - 2: Load Explorer

Next, double-click **vehicle lane load** and in the screen that appears, select the **Moving Loads** Tab. Figure O - 3 shows the vehicle lane load icon, as well as the moving loads tab.

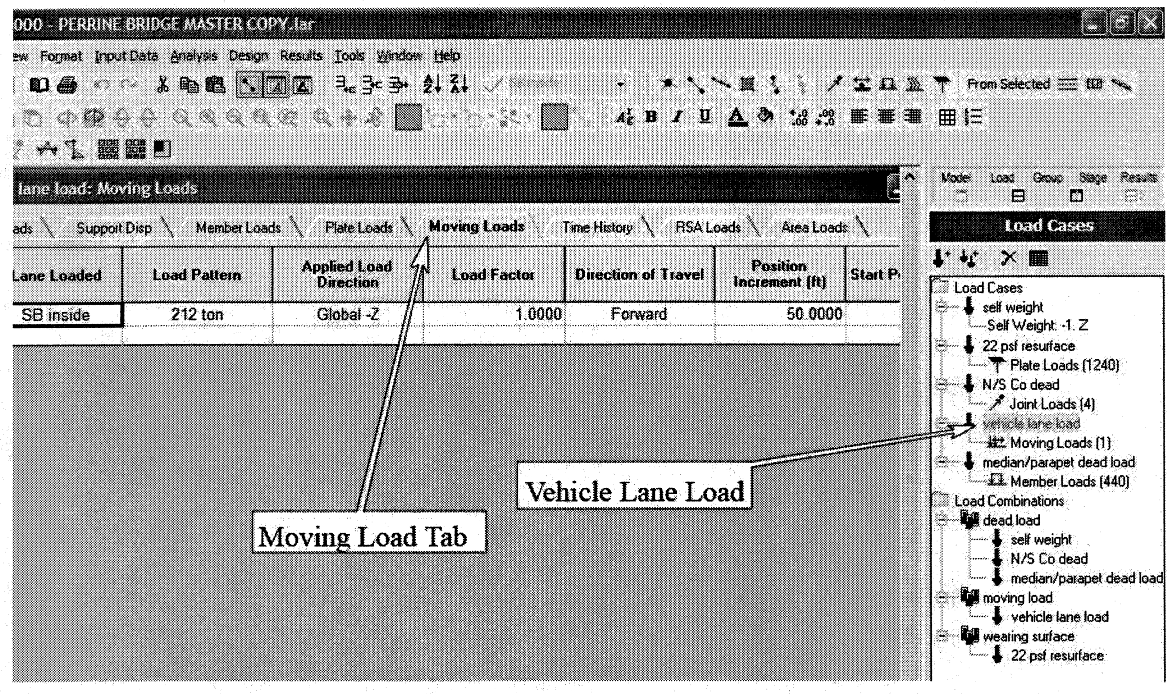


Figure O - 3: Moving Loads Tab

Once the moving load screen appears, select a lane for the moving load analysis. Since the vehicle is traveling southbound and this is the first LRFR rating analysis, either the SB inside or SB outside lanes would be acceptable lane choices. The SB inside lane is chosen from the list of lanes available in the Loaded Lane pull down menu. A striped lane should be selected for the initial moving load analysis and subsequent LRFR analysis because the vehicle will likely be traveling in a striped lane when it crosses the bridge. Further LRFR analyses can determine if the vehicle will be required to travel along another travel path.

Next, the load pattern for the moving load analysis must be selected. The load pattern menu is located adjacent to the loaded lane column. Figure O - 4 illustrates superimposed pull-down menu screens for both the loaded lane and load pattern options.

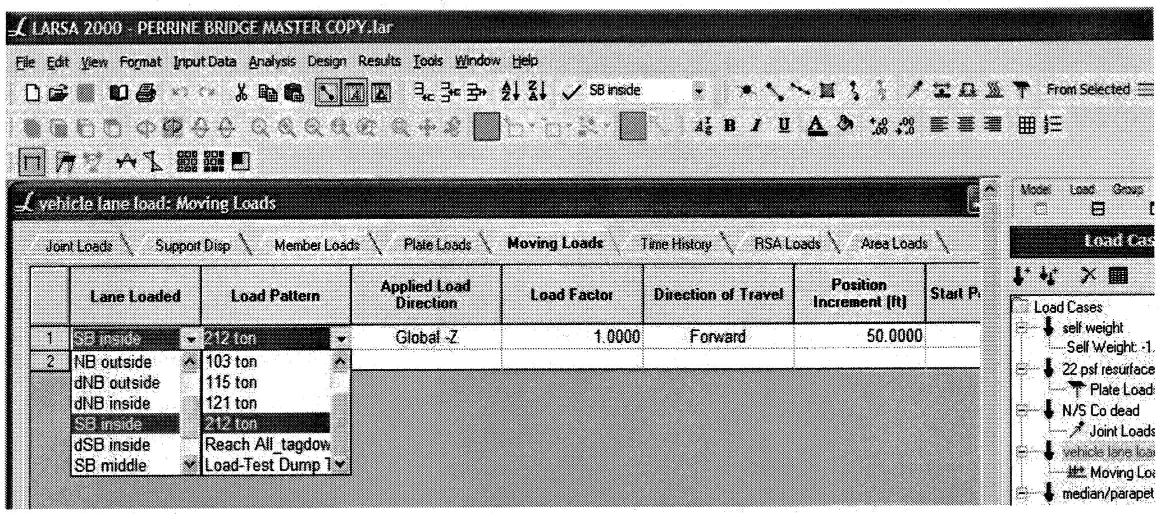


Figure O - 4: Select Loaded Lane and Load Pattern

The 212-ton load pattern is selected from the available load patterns that are defined in a load pattern database. In this same screen, under the Position Increment column, enter the interval at which load cases will be generated for the moving load analysis. The default setting is ten feet.

Finally, in the Analysis Menu, located at the top of the screen, select **Moving Load Analysis**. This is illustrated in Figure O - 5.

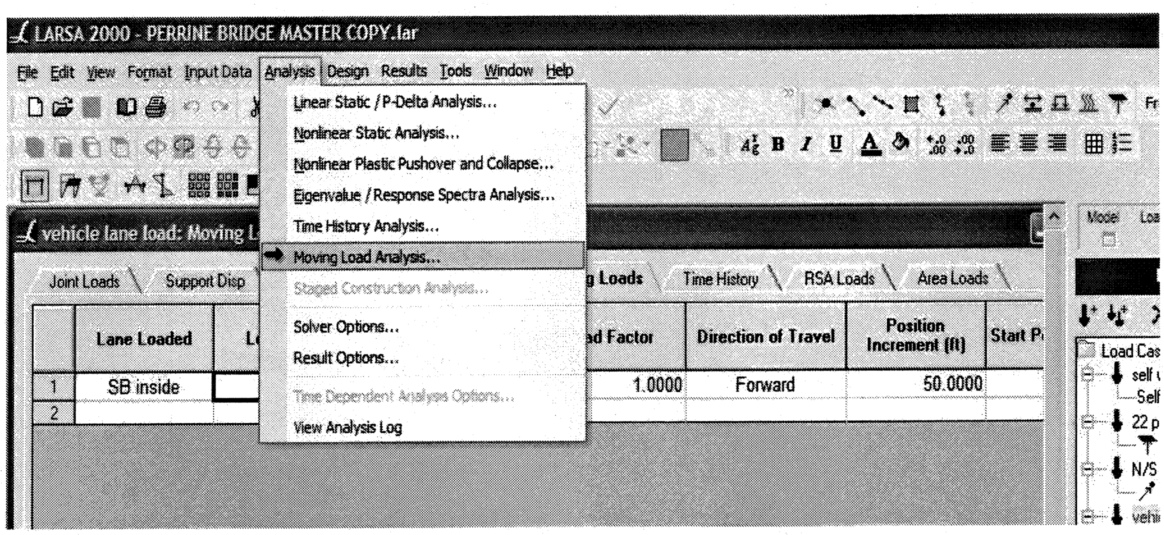


Figure O - 5: Select Moving Load Analysis

Once the moving load analysis screen appears, select the Analyze button. This performs the moving load analysis. A prompt will appear that asks the user to save before running an analysis, click **Yes**. If No is selected the analysis will not run. See Figure O - 6.

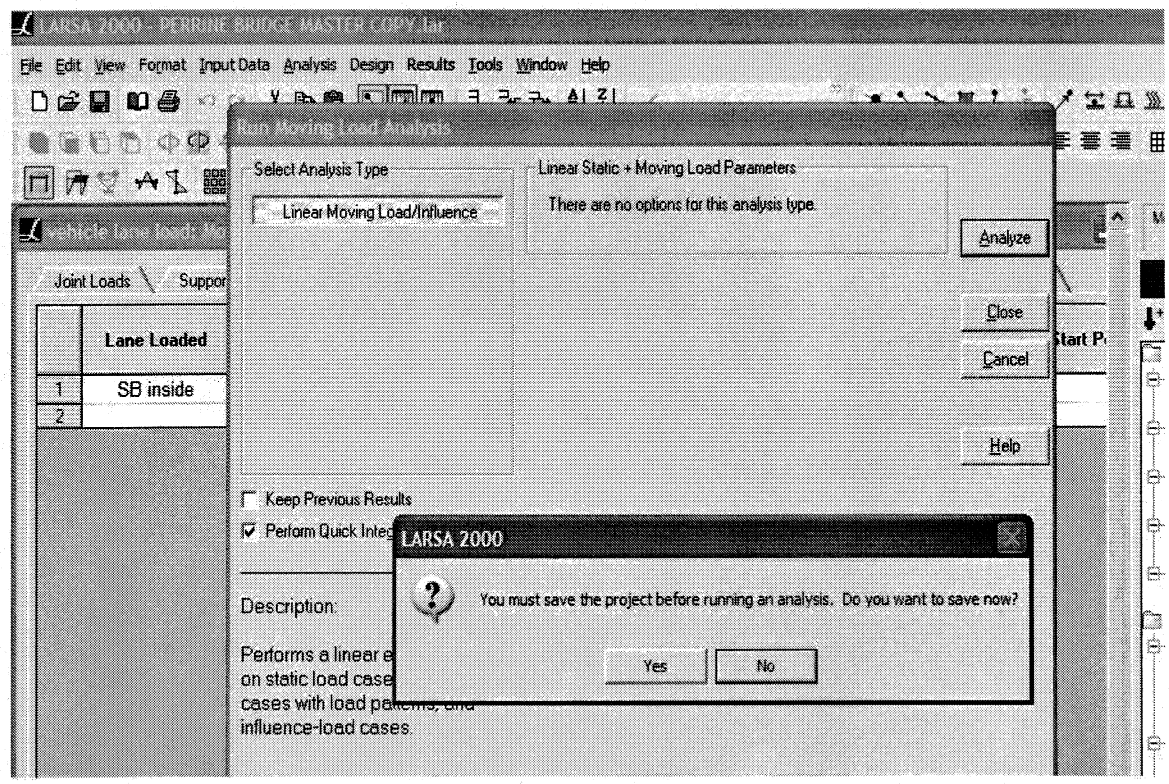


Figure O - 6: Run Moving Load Analysis

Preparing and running the moving load analysis is completed. The 212-ton vehicle pattern will run forward along the inside, striped, southbound lane. Starting at the beginning of the lane (0.00 ft), load cases are generated every ten feet (or at every position increment if the default is changed) until the last axle of the vehicle configuration travels off the bridge model.

O.2 Performing an LRFR Analysis

The enveloped results from a moving load analysis are used to calculate LRFR rating factors for structural members on the bridge. Dead, wearing surface, and live load effects are required to calculate LRFR rating factors; however, the dead and wearing surface load effects are already included in the Excel LRFR rating spreadsheet. The moving load analysis envelopes all applicable live load reaction data; imports the reaction data from LARSA into Excel; combines the live load envelope with the dead load effects and calculates LRFR rating factors for all bridge members. The instructions for performing an LRFR analysis in Excel are described in Section O.2.1. Section O.2.2 provides an example for performing an LRFR analysis.

O.2.1 Instructions: LRFR Analysis

Once a moving load analysis has been performed, an LRFR analysis should be performed to complete the load rating procedure. **LARSA 2000 Plus MUST remain open** after a moving load analysis is performed and during the Excel LRFR load rating analysis. If LARSA is closed, the Excel load rating spreadsheet will fail to operate.

1. Keep LARSA 2000 Plus open from the preceding moving load analysis.
2. Open Microsoft Excel.
3. Open "LOAD RATE_v1MASTER" file.
    a. File → Open → Local Disk (D:) → Perrine Bride Project folder → "LOAD RATE_v1MASTER"
    b. Enable Macros
4. Save the file under an appropriate name describing the truck being rated and the lane loaded, etc.
    a. File → Save As → [descriptive file name]
5. Find the "Control Panel" worksheet at the bottom of the screen; this worksheet is the leftmost worksheet.
6. Click the "Populate Results" button in the upper right corner of the worksheet.

The "Populate Results" button clears previous live load results from the Excel worksheets; imports the moving analysis results from LARSA; calculates load rating factors; and sorts load rating factors into the "Rating Summary" and "Critical Member Summary" worksheets. The "Populate Results" button imports the moving load analysis results from the active file open in LARSA. It is important that the appropriate moving load analysis have been performed just before the LRFR analysis to avoid errors when analyzing moving load results. An LRFR rating analysis takes approximately one hour and fifteen minutes. Run-time estimates are based on analyses performed on a computer with an Intel Pentium 4 running at 3.00 GHz with 1.00 GB of RAM.

O.2.2 Example: LRFR Analysis for the 212-Ton Vehicle

The following example will demonstrate an LRFR analysis for a 212-ton vehicle traveling southbound across the Perrine Bridge. This example assumes that the corresponding 212-ton vehicle moving load analysis has already been performed (see Example O.1.2) and that LARSA is open.

Start Excel and open the "LOAD RATE_v1MASTER" file. Find the worksheet titled "Control Panel" located at the bottom of the screen. This worksheet is the leftmost worksheet. Click the "Populate Results" button. This is illustrated in Figure O - 7.

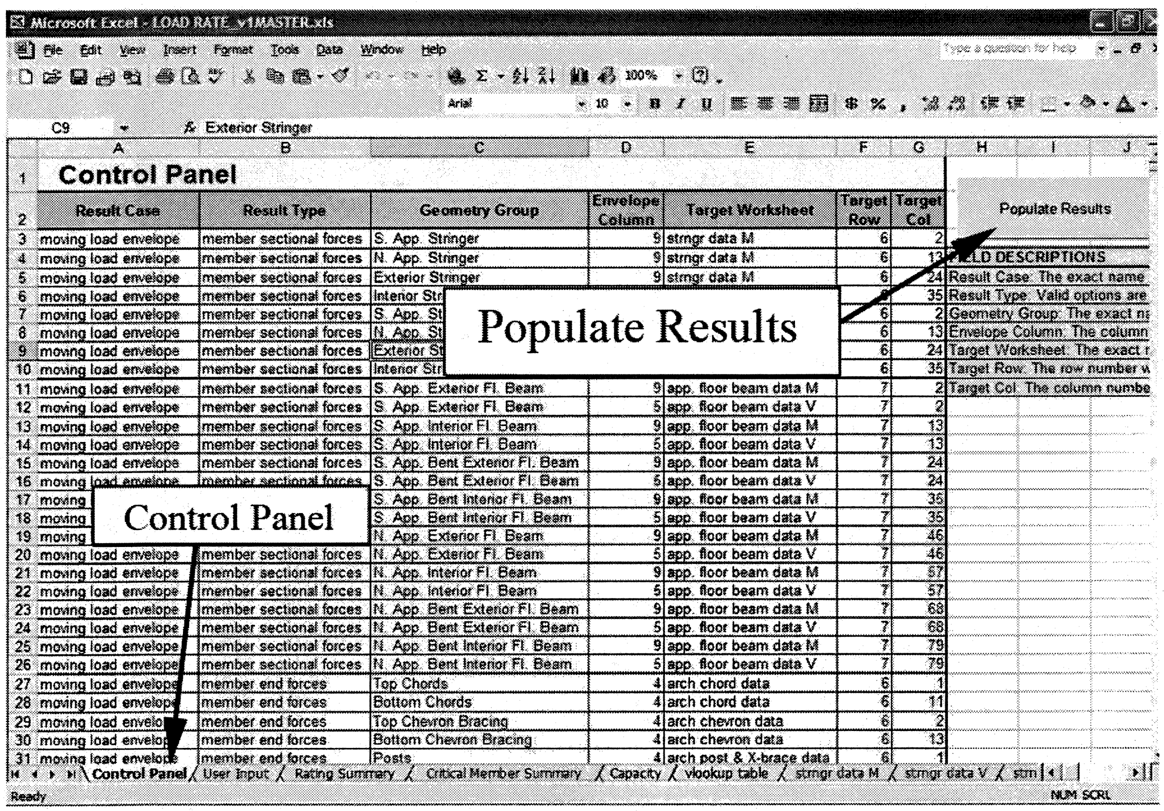


Figure O - 7: Excel Control Panel Worksheet

The "Populate Results" button performs the load rating procedure on the active file currently open in LARSA, in this case, the 212-ton vehicle.

O.3 LRFR Analysis Results

LRFR rating factors are summarized in the "Critical Member Summary" worksheet. This worksheet is located three worksheets to the right of the "Control Panel" worksheet. The organization of structural members and "Critical Member Summary" worksheet is presented in Sections O.3.1 and O.3.2, respectively. Finally, a description of the LRFR rating factor is included in Section O.3.3.

O.3.1 Structure Groups

Related structural components of the bridge are organized in Structure Groups in LARSA. For example, stringers of the south approach, north approach, and main arch spans are in three separate structure groups. Similarly, each diaphragm of the bridge deck is in a separate structure group. Structure groups are labeled according to the structure type they represent (e.g. south approach stringer, arch span floor beam, or spandrel column). Structure groups allow for efficient selection of a group of similar structural components to retrieve analysis results. There are a total of fifty-one structure groups used to retrieve moving load analysis results. LRFR rating factors are summarized according the structure group organization.

O.3.2 Critical Member Summary Worksheet

The "Critical Member Summary" worksheet contains five pages listing the lowest five rating factors and their corresponding member identification numbers for all applicable reactions of every structure group. Examples of an applicable reaction would be bending moments and shear forces for bending members and axial compression and tensile forces for two-force members, such as diaphragm or arch truss members. The member identification number corresponds to a unique member within the LARSA model. The following list outlines the structure groups contained in the summary worksheet and which rating factors are calculated for each structure group.

Page 1: Stringers & Approach Floor Beams

- Shear
- Positive Moment
- Negative Moment*

Page 2: Arch Truss Members

Top/Bottom Chords, Top/Bottom Chevron Bracing, Cross-Bracing

- Axial Compression
- Axial Tension

Posts

- Axial Compression

Diagonals

- Axial Tension

Top/Bottom Struts

- Shear
- Strong Axis Bending Moment
- Weak Axis Bending Moment
- Axial Compression
- Axial Tension

Page 3: Spandrel Columns & Deck Bracing Members

Spandrel Columns

- Axial Compression

Deck Bracing Members

- Axial Compression
- Axial Tension

Page 4: Bent Columns, Bent Column Bracing, & Prismatic Plate Girders

Bent Columns

- Axial Compression

Bent Column Bracing

- Axial Compression
- Axial Tension

Prismatic Plate Girders

- Shear
- Positive Moment

- Negative Moment*

Page 5: Arch Floor Beams & Haunch Plate Girders

- Shear
- Positive Moment
- Negative Moment*

*Column headings for negative moment values are labeled "M(-):" the values in those columns are the absolute values as provide by LARSA.

O.3.3 Summarizing LRFR Rating Factor Results

The LRFR rating factor represents the ratio of the member's live load capacity divided by the maximum live load demand from the rating vehicle. Rating factors greater than one are satisfactory since the live load capacity is greater than the demand. Rating factors between zero and one indicate that the live load capacity is less than required for the rating vehicle. For the uncalibrated bridge model (Section 1.3.2), there are some rating factors that are negative. Negative rating factors mean that the self-weight and wearing surface load effects exceed a member's capacity. Design capacities are calculated according to the AASHTO LRFD Bridge Design Specifications (2005 Interim Revision). Negative numbers are displayed with parenthesis in the "Critical Member Summary" worksheet. Members with negative rating factors are misleading because the bridge can obviously support self-weight and wearing surface loads, in addition to live load demands. The uncalibrated bridge model may erroneously be estimating larger load effects for members with negative rating factors. In the calibrated bridge model, the acceptable range of rating factors will be numbers greater than or equal to zero. The procedure for calculating LRFR rating factors is described in Section 3.

The Excel spreadsheet sorts through all LRFR rating factors and displays the five lowest rating factors and their corresponding member identification numbers for each of the applicable reactions listed in Section O.3.2. **The operator must check the "Critical Member Summary" worksheet to ensure that there are <u>NO</u> rating factors below one.**

If an LRFR analysis results in rating factors less than one, additional LRFR analyses can be performed after changing the loaded lane in the moving analysis to determine if satisfactory rating factors can be achieved. The permit should require the vehicle to travel over the bridge in the lane that yields satisfactory rating factors. Additionally, parameters used in calculating LRFR rating factors can also be modified in a second LRFR analysis depending on the type of trip allowed across the bridge (e.g. single trip with traffic, single trip without traffic, and multiple trip permits). Additionally, the impact parameter may be reduced if the vehicle is restricted to speeds less than ten mph. Adjusting the parameters for LRFR rating factors is described in Section 3.2.

Appendix P. VISUAL BASIC PROGRAMS

The following Visual Basic programs are used to import reaction data from LARSA to EXCEL, load rate the entire I.B. Perrine Bridge, and summarize the load rating analysis.

In general the program modules in this spreadsheet (1) import the data from LARSA to the spreadsheet, and (2) sort, group and envelope the reactions. The maximum member reactions are used in the rating calculations in the spreadsheet. The program modules that import the data from LARSA (Sections P.1-P.5) were written by LARSA's technical support staff. When the LARSA updated their program to version 7.0, the older importing procedures no longer functioned. When alerted to the problem, LARSA's technical support personnel rewrote those modules within a few days, restoring the spreadsheet to full functionality. If and when LARSA undergoes major revisions, it is likely that the program modules in Sections P.1-P.5 will need to be updated by LARSA.

P.1 Perform Load Rating Analysis

This macro was created by LARSA's technical support staff. The entire load rating analysis is initialized and carried out by selecting one button. Once the user selects the "Populate Results" button, this macro uses a series of other macros to carry out the entire load rating/ summarizing procedure.

```
Option Explicit

Private Sub btnPopulateData_Click()
  Clear_Data

  Dim row As Long
  ' go through each row in the worksheet and populate data
  ' starting from row 3, load each row from the
  ' control panel spreadsheet one by one.
  For row = 3 To 100000
    ' get the data from the spreadsheet and
    ' put into more meaningful variables.
    Dim resultcase As String: resultcase = Me.Cells(row, 1).Text
    Dim resulttype As String: resulttype = Me.Cells(row, 2).Text
    Dim geogroup As String:   geogroup = Me.Cells(row, 3).Text
```

```
    Dim envcol As String:     envcol = Me.Cells(row, 4).Text
    Dim targetworksheet As String: targetworksheet = Me.Cells(row, 5).Text
    Dim targetrow As String: targetrow = Me.Cells(row, 6).Text
    Dim targetcol As String: targetcol = Me.Cells(row, 7).Text

    ' if the result case column on this row is empty
    ' then, we are possibly at the end of the data.
    ' so exit execution.
    If resultcase = "" Then Exit For

    ' send all the data to this function to
    PopulateSpreadsheet resultcase, resulttype, geogroup, envcol, targetworksheet, targetrow,
targetcol
  Next
  'Sort reaction input from Larsa
  Sort_Reactions

  'Envelope spans to calculate rating factors
  Envelope_Span
  Arch_and_Column_Force_Envelope
  Brace_Member_Envelope
  Prismatic_Plate_Girder_Envelope
  Arch_Floor_Beam_and_Haunch_Member_Envelope

  'Sort through and find lowest rating factors for each structure group
  Failure_Rating_Stringer_AppFloorBeam
  Failure_Rating_Arch_Column
  Failure_Rating_Brace_Member
  Failure_Rating_Bent
  Failure_Rating_Prismatic_Girder_1
  Failure_Rating_Prismatic_Girder_2
  Failure_Rating_Arch_Floor_Beam_and_Haunch_Member_1
  Failure_Rating_Arch_Floor_Beam_and_Haunch_Member_2

  MsgBox "ALL DONE..."
End Sub
```

P.2 Select Result Case by Structure Group

This macro reads information concerning the reaction type, structure group, and the location in the load rating worksheet into which reaction envelopes are imported (i.e., worksheet name, paste row, and paste column) are read from a database in Excel. This macro was also created by LARSA's technical support staff.

```
Option Explicit

Public Function PopulateSpreadsheet(resultcase As String, resulttype As String, _
                group As String, envcol As String, _
                targetworksheet As String, targetrow As String, targetcol As String) As Boolean

 ' select the result case
 Larsa2000.Application.SelectResultCase resultcase

 ' findout which result type we are looking for
 Dim rtype As RESULTDATA
 Select Case LCase(Trim(resulttype))
 Case "joint displacements": rtype = RESULTDATA_JOINT_DISPLACEMENTS
 Case "joint reactions": rtype = RESULTDATA_JOINT_REACTIONS
 Case "member sectional forces": rtype = RESULTDATA_MEMBER_SECTIONAL_FORCES
 Case "member end forces": rtype = RESULTDATA_MEMBER_END_FORCES
 Case Else: MsgBox "Could not recognize the specified result type : " & resulttype & "."
 End Select

 ' select the geometry group
 If Not SelectGeoGroup(group) Then Exit Function

 ' check if the envelope column is properly defined
 If Not IsNumeric(envcol) Then
   MsgBox "You must enter the column number to be used to envelope the results. " & envcol
& " is not valid." '
   Exit Function
 End If

 ' get reference to the target worksheet
 Dim w As Worksheet: Set w = Worksheets(targetworksheet)
```

```
  If w Is Nothing Then
    MsgBox "Cannot find worksheet " & targetworksheet & ". Make sure you have entered the
name correctly."
    Exit Function
  End If

  ' check if the target row and column are properly defined
  If Not IsNumeric(targetrow) Then
    MsgBox "Invalid target row specified. " & targetrow & " is not valid."
    Exit Function
  End If

  If Not IsNumeric(targetcol) Then
    MsgBox "Invalid target col specified. " & targetcol & " is not valid."
    Exit Function
  End If

  w.Activate ' make sure that the output panel is open/visible.

  ' we are all set to load the results table, lets give it a try.
  GetResultFromLARSA rtype, CLng(envcol)

  ' ok resut table is loaded into the clipboard.
  ' GetResultFromLARSA() function very conveniently does that for us.
  ' now we need to paste this table to the target worksheet.

  ' first put some information about this table... Like a header.
  w.Cells(CLng(targetrow), CLng(targetcol)).Activate
  w.Paste

End Function
```

P.3 Select Result Case by Structure Group

Created by LARSA's technical support staff, this macro copies the specified reaction envelopes in LARSA that will later be pasted into the Excel spreadsheets.

```
'=====================================================================
```

```
Public Sub GetResultFromLARSA(resulttype As RESULTDATA, envcol As Long)
'=====================================================================

  ' this function will collect member sectional force results
  ' for the selected result case(s) in LARSA 2000. The results
  ' will be enveloped for the column number (envcol) provided to
  ' this function. for Mz send 9...

  ' the result data will be accumilated in one huge string
  ' so that it can easily be copied to clipboard and pasted on
  ' the excel spreadsheet.

  Dim x As Long

  ' creating a huge string with regular string data type
  ' is quite slow in VB. Instead we will use StringBuilder
  ' object from LARSA 2000.
  Dim output As String

  ' create a spreadsheet information data
  Dim info As RESULTS_SPREAD_INFO
  ' we want the result data in spreadsheet form.
  info.display = RDM_SPREADSHEET
  ' we would like to have member sectional forces
  info.rq.DataType = resulttype
  ' we would like to envelope member sectional forces for this column.
  info.rq.envelopeCol = envcol

  ' first we need to populate the spreadsheet row lookup table.
  ' this is basically a table that does not have the actual data
  ' but rather has information about how to load the actual data row by row.

  ' this will contain the spreadsheet row map
  Dim spreadrows() As SPREAD_ROW_LOOKUP, spreaddata() As Variant

  ' populate the spreadsheet.
  If Not ResultPopulateDataEx(info, "", "", spreadrows, spreaddata) Then
```

```
        MsgBox "Could not populated the spreadsheet."
        Exit Sub
    End If

    Dim i As Long, rowcount As Long

    ' now for each row lets load the actual data
    For i = 1 To UBound(spreadrows)

        ' this will contain the actual data at i'th row.
        Dim data() As Variant

        ' request the actual data from LARSA for this row of the spreadsheet.
        If Not ResultRowDataEx(i, info, data, spreadrows, spreaddata) Then
            MsgBox "Failed the load data at row " & i & "!"
            Exit Sub
        End If

        ' add the data into our string
        For x = 1 To UBound(data)
            output = output & data(x) & vbTab
        Next
        output = output + vbNewLine

        DoEvents ' this will avoid going into non-responsive mode.
    Next

    ' copy data into the clipboard
    Dim clipboard As DataObject
    Set clipboard = New DataObject
    clipboard.SetText output
    clipboard.PutInClipboard
End Sub
```

P.4 Select Structure Group in LARSA

This selects the structure group in LARSA. This macro was created by LARSA's technical support staff.

```
'============================================================================
Public Function SelectGeoGroup(geo As String) As Boolean
'============================================================================
  Dim i As Long

  SelectGeoGroup = True

  ' unselect everything
  project.Members.setAllSelected False

  ' go through each geometry group in the project and
  ' find the one with the provided name
  For i = 1 To project.geoGroups.Count
    Dim g As clsGeoGroup: Set g = project.geoGroups.itemByIndex(i)

    ' is the name of this group same as the provided name
    If LCase(Trim(g.Name)) = LCase(Trim(geo)) Then
      g.setSelected True ' if same, then select this group
      Exit Function     ' and exit function
    End If
  Next

  SelectGeoGroup = False
  MsgBox "Could not find geometry group '" & geo & "' in the project."
End Function
```

P.5 Select Reaction Envelope in LARSA

This selects the reaction envelope in LARSA. This macro was created by LARSA's technical support staff.

```
'=========================================================================
Public Function SelectResultCase(rcase As String) As Boolean
'=========================================================================
  Dim found As Boolean: found = False
  SelectResultCase = True
```

```
  Dim i As Long
  ' go over all the result cases in the project
  For i = 0 To analysis.caseCount - 1   ' see if the name of the project matches the provide result
case name
    If Trim(LCase(rcase)) = Trim(LCase(analysis.resultcase(i).Name)) Then
      analysis.resultcase(i).Selected = True: found = True ' if matches select the result case.
    Else
      analysis.resultcase(i).Selected = False            ' if not , make sure it is unselected.
    End If
  Next
  Dim comb As intResultCase
  ' we have to do the same thing for linear result combinations
  For i = 1 To project.resultCombinations.Count
    Set comb = project.resultCombinations.itemByIndex(i)
    If Trim(LCase(rcase)) = Trim(LCase(comb.Name)) Then
      comb.Selected = True: found = True ' if matches select the result case.
    Else
      comb.Selected = False            ' if not , make sure it is unselected.
    End If
  Next
  ' and also extreme effect groups
  For i = 1 To project.resultEnvelopes.Count
    Set comb = project.resultEnvelopes.itemByIndex(i)
    If Trim(LCase(rcase)) = Trim(LCase(comb.Name)) Then
      comb.Selected = True: found = True ' if matches select the result case.
    Else
      comb.Selected = False            ' if not , make sure it is unselected.
    End If
  Next
  If Not found Then
    MsgBox "Cannot find result case '" & rcase & "' in the project."
    SelectResultCase = False
  End If
End Function
```

P.6 Sort Analytic Elements into Real Beam Spans

Member reactions envelopes are pasted into Excel in no particular order. In order to easily sort through the analytic elements defining a real beam span, the element reactions must be sorted into a sequential order by member identification number. This macro is a sample of the sorting program for all stringer elements in the bridge model. This macro was created by Nick McDowell.

```
Public Sub Sort_Reactions()
'==========================================================================
'This function will sort the data in the excel worksheets once it is imported from Larsa
'Reaction data must be sorted so that the enveloping functions keep span identities in the
'proper order.
'==========================================================================
'Sorts South Approach stringer (moving load data) by span ID, then member ID
Worksheets("strngr data M").Range("A6:J2755").Sort _
  Key1:=Worksheets("strngr data M").Range("A6"), _
  Key2:=Worksheets("strngr data M").Range("B6")
Worksheets("strngr data V").Range("A6:J2755").Sort _
  Key1:=Worksheets("strngr data V").Range("A6"), _
  Key2:=Worksheets("strngr data V").Range("B6")

'Sorts North Approach stringer (moving load data) by span ID, then member ID
Worksheets("strngr data M").Range("L6:U3415").Sort _
  Key1:=Worksheets("strngr data M").Range("L6"), _
  Key2:=Worksheets("strngr data M").Range("M6")
Worksheets("strngr data V").Range("L6:U3415").Sort _
  Key1:=Worksheets("strngr data V").Range("L6"), _
  Key2:=Worksheets("strngr data V").Range("M6")

'Sorts Arch Exterior stringer (moving load data) by span ID, then member ID
Worksheets("strngr data M").Range("W6:AF2646").Sort _
  Key1:=Worksheets("strngr data M").Range("W6"), _
  Key2:=Worksheets("strngr data M").Range("X6")
Worksheets("strngr data V").Range("W6:AF2646").Sort _
  Key1:=Worksheets("strngr data V").Range("W6"), _
  Key2:=Worksheets("strngr data V").Range("X6")
```

```
'Sorts Arch Interior stringer (moving load data) by span ID, then member ID
Worksheets("strngr data M").Range("AH6:AQ9597").Sort _
   Key1:=Worksheets("strngr data M").Range("AH6"), _
   Key2:=Worksheets("strngr data M").Range("AI6")
Worksheets("strngr data V").Range("AH6:AQ9597").Sort _
   Key1:=Worksheets("strngr data V").Range("AH6"), _
   Key2:=Worksheets("strngr data V").Range("AI6")
```

P.7 Envelope Prismatic Spans

Once all reaction envelopes are copied and pasted into Excel from LARSA and analytic elements are sorted, this macro will sort through the analytic elements comprising a real prismatic beam span and select the largest reaction. This is a sample program for enveloping the south approach stringer spans; there are additional enveloping programs for other prismatic bridge spans. This macro was created by Nick McDowell

```
Public Sub Envelope_Span()
' This program will return three values, the maximum positive moment,
' the maximum negative moment, and the maximum shear force for all stringer spans
' and floor beam spans. Maximum (+/-) moment and shear will be retrieved for three load cases
' dead load/ self wieght, wearing surface, and moving load.
Dim SA_span, i, j As Integer
Dim pos_moment, neg_moment, shear As Double

'==========================================================================
' STRINGER SPAN ENVELOPING
' SOUTH APPROACH SPANS
'==========================================================================
' Use for loop to increment through SOUTH APPROACH spans/ initialize variables
' South Approach spans are numbered from 1 to 60
SA_span = 60
j = 0
' This loop will retrieve reactions from the dead load/ self wieght load case
For i = 1 To SA_span
```

```
  pos_moment = 0 'Initialize span envelope values before each span iteration begins
  neg_moment = 0
  shear = 0
  Do While Worksheets("strngr data M").Cells(j + 6, 45) = i
    If Worksheets("strngr data M").Cells(j + 6, 54) > pos_moment Then
      pos_moment = Worksheets("strngr data M").Cells(j + 6, 54)
    End If ' Returns largest positive moment (local Z)for span

    If Worksheets("strngr data M").Cells(j + 6, 54) < neg_moment Then
      neg_moment = Worksheets("strngr data M").Cells(j + 6, 54)
    End If ' Returns largest negative moment (local Z)for span

    If Abs(Worksheets("strngr data M").Cells(j + 6, 50)) > shear Then
      shear = Abs(Worksheets("strngr data M").Cells(j + 6, 50))
    End If ' Returns largest shear force (local Y) for span
    j = j + 1
  Loop
  Worksheets("strngr rating").Cells(6 + i, 13) = pos_moment
  Worksheets("strngr rating").Cells(6 + i, 14) = neg_moment
  Worksheets("strngr rating").Cells(6 + i, 12) = shear

Next i

' This loop will retrieve reactions from the wearing surface load case
j = 0
For i = 1 To SA_span
  pos_moment = 0 'Initialize span envelope values before each span iteration begins
  neg_moment = 0
  shear = 0
  Do While Worksheets("strngr data M").Cells(j + 6, 89) = i
    If Worksheets("strngr data M").Cells(j + 6, 98) > pos_moment Then
      pos_moment = Worksheets("strngr data M").Cells(j + 6, 98)
    End If ' Returns largest positive moment (local Z) for span

    If Worksheets("strngr data M").Cells(j + 6, 98) < neg_moment Then
      neg_moment = Worksheets("strngr data M").Cells(j + 6, 98)
    End If ' Returns largest negative moment (local Z) for span
```

```
        If Abs(Worksheets("strngr data M").Cells(j + 6, 94)) > shear Then
            shear = Abs(Worksheets("strngr data M").Cells(j + 6, 94))
        End If ' Returns largest shear force (local Y) for span
        j = j + 1
    Loop
    Worksheets("strngr rating").Cells(6 + i, 17) = pos_moment
    Worksheets("strngr rating").Cells(6 + i, 18) = neg_moment
    Worksheets("strngr rating").Cells(6 + i, 16) = shear

Next i

' This loop will retrieve reactions from the moving load envelope
j = 0
For i = 1 To SA_span
    pos_moment = 0 'Initialize span envelope values before each span iteration begins
    neg_moment = 0
    Do While Worksheets("strngr data M").Cells(j + 6, 1) = i
        If Worksheets("strngr data M").Cells(j + 6, 10) > pos_moment Then
            pos_moment = Worksheets("strngr data M").Cells(j + 6, 10)
        End If ' Returns largest positive moment (local Z) for span

        If Worksheets("strngr data M").Cells(j + 6, 10) < neg_moment Then
            neg_moment = Worksheets("strngr data M").Cells(j + 6, 10)
        End If ' Returns largest negative moment (local Z) for span
        j = j + 1
    Loop

    Worksheets("strngr rating").Cells(6 + i, 22) = pos_moment
    Worksheets("strngr rating").Cells(6 + i, 23) = neg_moment
Next i
j = 0
For i = 1 To SA_span
    shear = 0
    Do While Worksheets("strngr data V").Cells(j + 6, 1) = i
        If Abs(Worksheets("strngr data V").Cells(j + 6, 6)) > shear Then
            shear = Abs(Worksheets("strngr data V").Cells(j + 6, 6))
```

```
    End If ' Returns largest shear force (local Y) for span
    j = j + 1
  Loop
  Worksheets("strngr rating").Cells(6 + i, 21) = shear
Next i
```

P.8 Envelope Haunch Spans

Only the reactions located at center span of analytic elements modeling portions of haunch spans are used for load rating haunch girders. The following is a sample of a macro that selects the reaction envelopes at center span of the south approach exterior haunched girder. This macro was created by Nick McDowell.

```
Public Sub Arch_Floor_Beam_and_Haunch_Member_Envelope()
' This program will return the forces in the local Mz, local Fy for use in the plate girder rating
spreadsheets
' The the force data contains reactions at the member sections along a member's length, there
are 10 sections/member.
' For the dead load and wearing surface case there are 10 section reactions/ member;
' one set of reactions (Fx, Fy, Fz, Mx, My, Mz) at each section.
' For the moving load envelope, two reactions at each section; one is the maximum positive
reaction and
' one is the maximum negative reaction for the envelope specified at each section.
Dim span, i, j As Integer
Dim shear, pos_moment, neg_moment As Double
'=============================================================================
'SOUTH APPROACH EXTERIOR HAUNCH PLATE GIRDER
'=============================================================================
'There are 20 members
'Analytical members that represent structural haunch members use the average web height between two
'points on the haunch.  Therefore, the actual web height of the modeled haunch member corresponds to the
'center span of the analytical member in the model.  Section forces are given at 10 equally spaced
'intervals along the length of an element.  Section 5 corresponds to the mid-span section reactions.
'Only reactions at section 5 will be used in the LRFR load rating procedure.
'DEAD LOAD/SELF WEIGHT

span = 20
```

```
For i = 1 To span
   shear = 0.00001     'Initialize reaction values before each member iteration begins
   pos_moment = 0.00001
   neg_moment = -0.00001
   If Worksheets("haunch girder M data").Cells(i * 11, 89) > pos_moment Then
      pos_moment = Worksheets("haunch girder M data").Cells(i * 11, 89)
   End If
   If Worksheets("haunch girder M data").Cells(i * 11, 89) < neg_moment Then
      neg_moment = Worksheets("haunch girder M data").Cells(i * 11, 89)
   End If
   If Abs(Worksheets("haunch girder M data").Cells(i * 11, 85)) > shear Then
      shear = Abs(Worksheets("haunch girder M data").Cells(i * 11, 85))
   End If

   Worksheets("haunch girder rating").Cells(i + 5, 14) = shear
   Worksheets("haunch girder rating").Cells(i + 5, 15) = pos_moment
   Worksheets("haunch girder rating").Cells(i + 5, 16) = neg_moment
Next i

'WEARING SURFACE

For i = 1 To span
   shear = 0.00001     'Initialize reaction values before each member iteration begins
   pos_moment = 0.00001
   neg_moment = -0.00001
   If Worksheets("haunch girder M data").Cells(i * 11, 169) > pos_moment Then
      pos_moment = Worksheets("haunch girder M data").Cells(i * 11, 169)
   End If
   If Worksheets("haunch girder M data").Cells(i * 11, 169) < neg_moment Then
      neg_moment = Worksheets("haunch girder M data").Cells(i * 11, 169)
   End If
   If Abs(Worksheets("haunch girder M data").Cells(i * 11, 165)) > shear Then
      shear = Abs(Worksheets("haunch girder M data").Cells(i * 11, 165))
   End If

   Worksheets("haunch girder rating").Cells(i + 5, 18) = shear
   Worksheets("haunch girder rating").Cells(i + 5, 19) = pos_moment
```

```
  Worksheets("haunch girder rating").Cells(i + 5, 20) = neg_moment
Next i

'MOVING LOAD ANALYSIS
For i = 0 To span - 1
  shear = 0.00001     'Initialize reaction values before each member iteration begins
  pos_moment = 0.00001
  neg_moment = -0.00001
  If Worksheets("haunch girder M data").Cells(17 + i * 22, 9) > pos_moment Then
    pos_moment = Worksheets("haunch girder M data").Cells(17 + i * 22, 9)
  End If
  If Worksheets("haunch girder M data").Cells(16 + i * 22, 9) < neg_moment Then
    neg_moment = Worksheets("haunch girder M data").Cells(16 + i * 22, 9)
  End If
  If Abs(Worksheets("haunch girder V data").Cells(16 + i * 22, 5)) > shear Then
    shear = Abs(Worksheets("haunch girder V data").Cells(16 + i * 22, 5))
  End If
  If Abs(Worksheets("haunch girder V data").Cells(17 + i * 22, 5)) > shear Then
    shear = Abs(Worksheets("haunch girder V data").Cells(17 + i * 22, 5))
  End If
  Worksheets("haunch girder rating").Cells(i + 6, 23) = shear
  Worksheets("haunch girder rating").Cells(i + 6, 24) = pos_moment
  Worksheets("haunch girder rating").Cells(i + 6, 25) = neg_moment
Next i
```

P.9 Envelope Axial Force Members

Axial force envelopes only provide the minimum and maximum reactions at the ends of a member, as opposed to bending force members where reactions are provided at ten sections throughout the element length. This program selects the greatest tensile and compression force that occurs at the ends of a member. The following is a sample of a column brace envelope program. This macro was developed by Nick McDowell.

```
Public Sub Brace_Member_Envelope()
```

' This program will return the forces in the local x for use in the brace rating spreadsheets
' The forces in a member's local x direction correspond to axial forces
' According to the Larsa User's Manual, for the start joint, positive values are compressive
' and for the end joint, positive values are tensile.

' The the force data contains reactions at the member end nodes only. For the dead load case
' and the wearing surface case there are two node reactions for one member corresponding to the
' one reaction throughout the beam. For the moving load envelope, there are four reactions,
' two different reactions at each node. These reactions correspond to the maximum tensile and
' compressive forces at a node.
' SPANDREL COLUMN BRACING ENVELOPING
' The load rating for spandrel column BRACING considers BOTH axial compression AND tensile forces.

```
'========================================================================
' SPANDREL COLUMN BRACING Enveloping
'========================================================================
' There are 50 columns bracing members that have a counting ID from 1 - 50
' Return the maximum axial reactions for the DEAD LOAD CASE
Dim column_bracing, i, j As Integer
Dim axial_comp, tensile As Double
column_bracing = 50
j = 0
For i = 1 To column_bracing
  axial_comp = 0.001 'Initialize axial force values before each member iteration begins
  tensile = 0.001
  'CHECK FIRST "I" NODE OF MEMBER
    If Worksheets("column brace data").Cells(j + 6, 15) < 0 Then
'tensile forces are negative at "i" end of a member
      tensile = Abs(Worksheets("column brace data").Cells(j + 6, 15))
    End If
    If Worksheets("column brace data").Cells(j + 6, 15) > 0 Then
'compressive forces are positive at "i" end of a member
      axial_comp = Worksheets("column brace data").Cells(j + 6, 15)
    End If
    'CHECK SECOND "J" NODE OF MEMBER
    If Worksheets("column brace data").Cells(j + 7, 15) > 0 Then
      If Worksheets("column brace data").Cells(j + 7, 15) > tensile Then
'tensile forces are positive at "j" end of member
        tensile = Worksheets("column brace data").Cells(j + 7, 15)
```

```
        End If
      End If     ' recovers maximum tensile force
      Worksheets("column brace rating").Cells(5 + i, 12) = tensile
'send max. tensile force to rating spreadsheet
      If Worksheets("column brace data").Cells(j + 7, 15) < 0 Then
        If Abs(Worksheets("column brace data").Cells(j + 7, 15)) > axial_comp Then
'compression forces are negative at "j" end of member
          axial_comp = Abs(Worksheets("column brace data").Cells(j + 7, 15))
        End If
      End If     ' recovers maximum compressive force
      Worksheets("column brace rating").Cells(5 + i, 11) = axial_comp
      j = j + 2
Next i


' Return the maximum axial reaction for the WEARING SURFACE LOAD CASE
j = 0
For i = 1 To column_bracing
   axial_comp = 0.001 'Initialize axial force values before each member iteration begins
   tensile = 0.001
   'CHECK FIRST "I" NODE OF MEMBER
      If Worksheets("column brace data").Cells(j + 6, 27) < 0 Then
'tensile forces are negative at "i" end of a member
        tensile = Abs(Worksheets("column brace data").Cells(j + 6, 27))
      End If
      If Worksheets("column brace data").Cells(j + 6, 27) > 0 Then
'compressive forces are positive at "i" end of a member
        axial_comp = Worksheets("column brace data").Cells(j + 6, 27)
      End If
      'CHECK SECOND "J" NODE OF MEMBER
      If Worksheets("column brace data").Cells(j + 7, 27) > 0 Then
        If Worksheets("column brace data").Cells(j + 7, 27) > tensile Then
'tensile forces are positive at "j" end of member
          tensile = Worksheets("column brace data").Cells(j + 7, 27)
        End If
      End If ' recovers maximum tensile force
      Worksheets("column brace rating").Cells(5 + i, 15) = tensile
```

```
'send max. tensile force to rating spreadsheet
      If Worksheets("column brace data").Cells(j + 7, 27) < 0 Then
         If Abs(Worksheets("column brace data").Cells(j + 7, 27)) > axial_comp Then
'compression forces are negative at "j" end of member
            axial_comp = Abs(Worksheets("column brace data").Cells(j + 7, 27))
         End If
      End If ' recovers maximum compressive force
      Worksheets("column brace rating").Cells(5 + i, 14) = axial_comp
      j = j + 2
Next i

' Return the maximum axial reaction for the MOVING LOAD ENVELOPE

' In the moving load envelope the maximum tensile and compression forces are included at each
' node.
column_bracing = 50
j = 0
For i = 1 To column_bracing
   axial_comp = 0.001     'Initialize axial force values before each member iteration begins
   tensile = 0.001
      'CHECK FIRST "I" NODE OF MEMBER
      If Worksheets("column brace data").Cells(j + 6, 4) < 0 Then
'tensile forces are negative at "i" end of a member
         tensile = Abs(Worksheets("column brace data").Cells(j + 6, 4))
      End If
      If Worksheets("column brace data").Cells(j + 7, 4) > 0 Then
'compressive forces are positive at "i" end of a member
         axial_comp = Worksheets("column brace data").Cells(j + 7, 4)
      End If
      'CHECK SECOND "J" NODE OF MEMBER
      If Worksheets("column brace data").Cells(j + 8, 4) > 0 Then
         If Worksheets("column brace data").Cells(j + 8, 4) > tensile Then
'tensile forces are positive at "j" end of member
            tensile = Worksheets("column brace data").Cells(j + 8, 4)
         End If
      End If        ' recovers maximum tensile force
```

```
    If Worksheets("column brace data").Cells(j + 9, 4) < 0 Then
      If Abs(Worksheets("column brace data").Cells(j + 9, 4)) > axial_comp Then
'compression forces are negative at "j" end of member
        axial_comp = Abs(Worksheets("column brace data").Cells(j + 9, 4))
      End If
    End If ' recovers maximum compressive force
    Worksheets("column brace rating").Cells(5 + i, 18) = axial_comp
    Worksheets("column brace rating").Cells(5 + i, 19) = tensile
    j = j + 4
Next i
```

P.10 Summarize Load Rating Factors

Each structure group contains numerous rating factors. This macro selects the five lowest rating factors and their corresponding member identification numbers for each reaction of each structure group and pastes them in a summary worksheet. The following is a sample macro for the structure group of south approach stringers. This macro was developed by Nick McDowell.

```
Public Sub Failure_Rating_Stringer_AppFloorBeam()
'This program will return the five lowest load rating factors for each structure member type
'For example, the lowest five south approach stringer load ratings will be returned
'The purpose for this program is determine what members are sensitive to live loads
'and to determine if members are exceeding their live load capacities
'This program will operate on all stringers and approach floor beams, due to the size
'of the file, the critical rating factor program is broken into four program operations.
'===========================================================================
' SOUTH APPROACH STRINGERS
'===========================================================================
' The critical member identification will select the 5 stringer spans with the lowest
' load rate factor.  RF_5 will be the lowest rate factor and RF_1 will be the greatest

' There are 60 spans
' Return critical spans for SHEAR LOAD RATING FACTOR
Dim span, i, span1, span2, span3, span4, span5 As Integer
Dim RF_1, RF_2, RF_3, RF_4, RF_5 As Double
span = 60
j = 0
```

```
'Initialize critical rating factors
RF_1 = 1000
RF_2 = 1000
RF_3 = 1000
RF_4 = 1000
RF_5 = 1000
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_5, IF SO REPLACE RF_5 WITH SMALLER RATE
FACTOR IN CELL
    If Worksheets("Rating Summary").Cells(i + 6, 2) < RF_5 Then
      RF_5 = Worksheets("Rating Summary").Cells(i + 6, 2)
      span5 = Worksheets("Rating Summary").Cells(i + 6, 1)
    End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_4, IF SO REPLACE RF_4 WITH SMALLER RATE
FACTOR IN CELL
    If Worksheets("Rating Summary").Cells(i + 6, 2) < RF_4 Then
      If Worksheets("Rating Summary").Cells(i + 6, 2) > RF_5 Then
        RF_4 = Worksheets("Rating Summary").Cells(i + 6, 2)
        span4 = Worksheets("Rating Summary").Cells(i + 6, 1)
      End If
    End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_3, IF SO REPLACE RF_3 WITH SMALLER RATE
FACTOR IN CELL
    If Worksheets("Rating Summary").Cells(i + 6, 2) < RF_3 Then
      If Worksheets("Rating Summary").Cells(i + 6, 2) > RF_4 Then
        RF_3 = Worksheets("Rating Summary").Cells(i + 6, 2)
        span3 = Worksheets("Rating Summary").Cells(i + 6, 1)
      End If
    End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_2, IF SO REPLACE RF_2 WITH SMALLER RATE
FACTOR IN CELL
```

```
        If Worksheets("Rating Summary").Cells(i + 6, 2) < RF_2 Then
          If Worksheets("Rating Summary").Cells(i + 6, 2) > RF_3 Then
            RF_2 = Worksheets("Rating Summary").Cells(i + 6, 2)
            span2 = Worksheets("Rating Summary").Cells(i + 6, 1)
          End If
        End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_1, IF SO REPLACE RF_1 WITH SMALLER RATE
FACTOR IN CELL
        If Worksheets("Rating Summary").Cells(i + 6, 2) < RF_1 Then
          If Worksheets("Rating Summary").Cells(i + 6, 2) > RF_2 Then
            RF_1 = Worksheets("Rating Summary").Cells(i + 6, 2)
            span1 = Worksheets("Rating Summary").Cells(i + 6, 1)
          End If
        End If
Next i
Worksheets("Critical Member Summary").Cells(7, 2) = RF_5
Worksheets("Critical Member Summary").Cells(8, 2) = RF_4
Worksheets("Critical Member Summary").Cells(9, 2) = RF_3
Worksheets("Critical Member Summary").Cells(10, 2) = RF_2
Worksheets("Critical Member Summary").Cells(11, 2) = RF_1
Worksheets("Critical Member Summary").Cells(7, 1) = span5
Worksheets("Critical Member Summary").Cells(8, 1) = span4
Worksheets("Critical Member Summary").Cells(9, 1) = span3
Worksheets("Critical Member Summary").Cells(10, 1) = span2
Worksheets("Critical Member Summary").Cells(11, 1) = span1
' Return critical spans for POSITIVE MOMENT LOAD RATING FACTOR
'Initialize critical rating factors
RF_1 = 1000
RF_2 = 1000
RF_3 = 1000
RF_4 = 1000
RF_5 = 1000
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_5, IF SO REPLACE RF_5 WITH SMALLER RATE
FACTOR IN CELL
```

```
    If Worksheets("Rating Summary").Cells(i + 6, 3) < RF_5 Then
      RF_5 = Worksheets("Rating Summary").Cells(i + 6, 3)
      span5 = Worksheets("Rating Summary").Cells(i + 6, 1)
    End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_4, IF SO REPLACE RF_4 WITH SMALLER RATE
FACTOR IN CELL
    If Worksheets("Rating Summary").Cells(i + 6, 3) < RF_4 Then
      If Worksheets("Rating Summary").Cells(i + 6, 3) > RF_5 Then
        RF_4 = Worksheets("Rating Summary").Cells(i + 6, 3)
        span4 = Worksheets("Rating Summary").Cells(i + 6, 1)
      End If
    End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_3, IF SO REPLACE RF_3 WITH SMALLER RATE
FACTOR IN CELL
    If Worksheets("Rating Summary").Cells(i + 6, 3) < RF_3 Then
      If Worksheets("Rating Summary").Cells(i + 6, 3) > RF_4 Then
        RF_3 = Worksheets("Rating Summary").Cells(i + 6, 3)
        span3 = Worksheets("Rating Summary").Cells(i + 6, 1)
      End If
    End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_2, IF SO REPLACE RF_2 WITH SMALLER RATE
FACTOR IN CELL
    If Worksheets("Rating Summary").Cells(i + 6, 3) < RF_2 Then
      If Worksheets("Rating Summary").Cells(i + 6, 3) > RF_3 Then
        RF_2 = Worksheets("Rating Summary").Cells(i + 6, 3)
        span2 = Worksheets("Rating Summary").Cells(i + 6, 1)
      End If
    End If
Next i
For i = 1 To span
```

```
  'CHECK IF CELL IS LESS THAN RF_1, IF SO REPLACE RF_1 WITH SMALLER RATE
FACTOR IN CELL
      If Worksheets("Rating Summary").Cells(i + 6, 3) < RF_1 Then
         If Worksheets("Rating Summary").Cells(i + 6, 3) > RF_2 Then
            RF_1 = Worksheets("Rating Summary").Cells(i + 6, 3)
            span1 = Worksheets("Rating Summary").Cells(i + 6, 1)
         End If
      End If
Next i
Worksheets("Critical Member Summary").Cells(7, 4) = RF_5
Worksheets("Critical Member Summary").Cells(8, 4) = RF_4
Worksheets("Critical Member Summary").Cells(9, 4) = RF_3
Worksheets("Critical Member Summary").Cells(10, 4) = RF_2
Worksheets("Critical Member Summary").Cells(11, 4) = RF_1
Worksheets("Critical Member Summary").Cells(7, 3) = span5
Worksheets("Critical Member Summary").Cells(8, 3) = span4
Worksheets("Critical Member Summary").Cells(9, 3) = span3
Worksheets("Critical Member Summary").Cells(10, 3) = span2
Worksheets("Critical Member Summary").Cells(11, 3) = span1
' Return critical spans for NEGATIVE MOMENT LOAD RATING FACTOR
'Initialize critical rating factors
RF_1 = 1000
RF_2 = 1000
RF_3 = 1000
RF_4 = 1000
RF_5 = 1000
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_5, IF SO REPLACE RF_5 WITH SMALLER RATE
FACTOR IN CELL
      If Worksheets("Rating Summary").Cells(i + 6, 4) < RF_5 Then
         RF_5 = Worksheets("Rating Summary").Cells(i + 6, 4)
         span5 = Worksheets("Rating Summary").Cells(i + 6, 1)
      End If
Next i
For i = 1 To span
  'CHECK IF CELL IS LESS THAN RF_4, IF SO REPLACE RF_4 WITH SMALLER RATE
FACTOR IN CELL
```

```
        If Worksheets("Rating Summary").Cells(i + 6, 4) < RF_4 Then
            If Worksheets("Rating Summary").Cells(i + 6, 4) > RF_5 Then
                RF_4 = Worksheets("Rating Summary").Cells(i + 6, 4)
                span4 = Worksheets("Rating Summary").Cells(i + 6, 1)
            End If
        End If
Next i
For i = 1 To span
    'CHECK IF CELL IS LESS THAN RF_3, IF SO REPLACE RF_3 WITH SMALLER RATE
FACTOR IN CELL
        If Worksheets("Rating Summary").Cells(i + 6, 4) < RF_3 Then
            If Worksheets("Rating Summary").Cells(i + 6, 4) > RF_4 Then
                RF_3 = Worksheets("Rating Summary").Cells(i + 6, 4)
                span3 = Worksheets("Rating Summary").Cells(i + 6, 1)
            End If
        End If
Next i
For i = 1 To span
    'CHECK IF CELL IS LESS THAN RF_2, IF SO REPLACE RF_2 WITH SMALLER RATE
FACTOR IN CELL
        If Worksheets("Rating Summary").Cells(i + 6, 4) < RF_2 Then
            If Worksheets("Rating Summary").Cells(i + 6, 4) > RF_3 Then
                RF_2 = Worksheets("Rating Summary").Cells(i + 6, 4)
                span2 = Worksheets("Rating Summary").Cells(i + 6, 1)
            End If
        End If
Next i
For i = 1 To span
    'CHECK IF CELL IS LESS THAN RF_1, IF SO REPLACE RF_1 WITH SMALLER RATE
FACTOR IN CELL
        If Worksheets("Rating Summary").Cells(i + 6, 4) < RF_1 Then
            If Worksheets("Rating Summary").Cells(i + 6, 4) > RF_2 Then
                RF_1 = Worksheets("Rating Summary").Cells(i + 6, 4)
                span1 = Worksheets("Rating Summary").Cells(i + 6, 1)
            End If
        End If
Next i
```

```
Worksheets("Critical Member Summary").Cells(7, 6) = RF_5
Worksheets("Critical Member Summary").Cells(8, 6) = RF_4
Worksheets("Critical Member Summary").Cells(9, 6) = RF_3
Worksheets("Critical Member Summary").Cells(10, 6) = RF_2
Worksheets("Critical Member Summary").Cells(11, 6) = RF_1
Worksheets("Critical Member Summary").Cells(7, 5) = span5
Worksheets("Critical Member Summary").Cells(8, 5) = span4
Worksheets("Critical Member Summary").Cells(9, 5) = span3
Worksheets("Critical Member Summary").Cells(10, 5) = span2
Worksheets("Critical Member Summary").Cells(11, 5) = span1
```

Appendix Q. Program Flow Charts

The operation of the Visual Basic programs in Appendix P is summarized in the following flowcharts. Each flowchart lists the Visual Basic function or subroutine call and the module name from the Excel spreadsheet. The module name identifies which programming module contains the function or subroutine call. Each programming module can be selected and edited in a separate window in the Visual Basic programming environment.

The flowcharts for the Envelope Span (Section Q.4) and the Failure Rating (Section Q.5) subroutines are typical of several subroutines differing only in the member type referenced in the subroutine.

Q.1 Populate Results

From "Control Panel" Worksheet,
"Populate Results" button calls
btnPopulateData_Click()

Read each row of "Control Panel" worksheet for data to import from Larsa:
Column a: Result Case
Column b: Result Type
Column c: Geometry Type
Column d: Envelope Column

and Excel worksheet location to place data:
Column e: Target Worksheet
Column f: Target Row
Column g: Target Column

Put Larsa data in spreadsheets

Call Subroutine **PopulateSpreadsheets**

Sort moving load data for each geometry type.
(Element reaction data from Larsa must be sorted so that the enveloping functions keep the span identities in the proper order)

Call Subroutine **Sort_Reactions()** (Module9)
Use Excel "Sort" function to sort each element type by Larsa Span ID, then Member ID

Find maximum moments, shears or axial forces in each physical element
(i.e., the moment, shear or axial force envelope)

Stringer Spans: Call Subroutine **Envelope_Span** (Module1)
Call Subroutine **Arch_and_Column_Force_Envelope** (Module2)
Call Subroutine **Brace_Member_Envelope** (Module3)
Call Subroutine **Prismatic_Plate_Girder_Envelope** (Module10)
Call Subroutine **Arch_Floor_Beam_and_Haunch_Member_Envelope** (Module13)

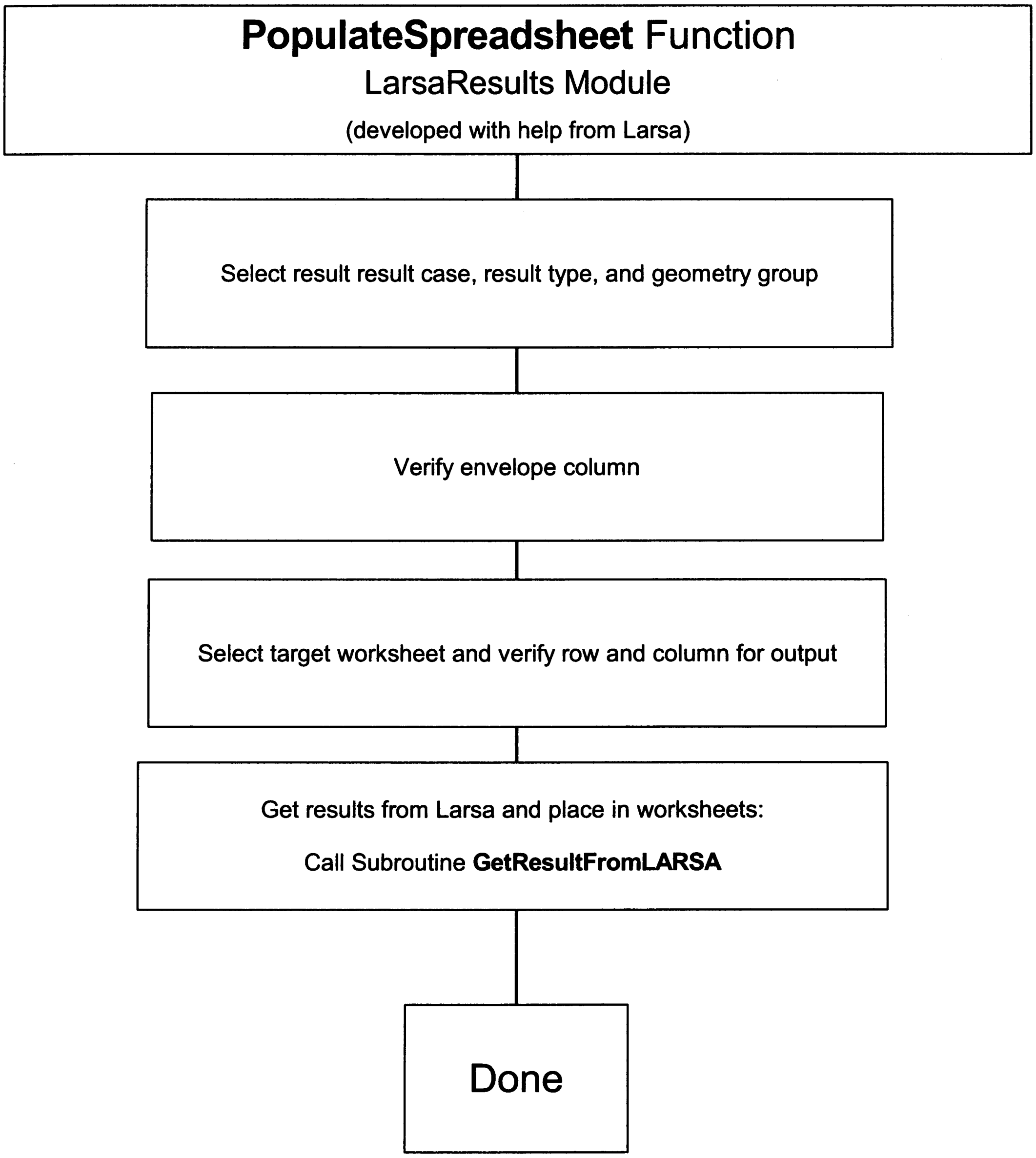
Calculate rating factors for maximum moment, shear or axial force in each geometry type in Excel Spreadsheets
e.g., Sheet5: strngr rating

Sort to find lowest rating factor for each structure group

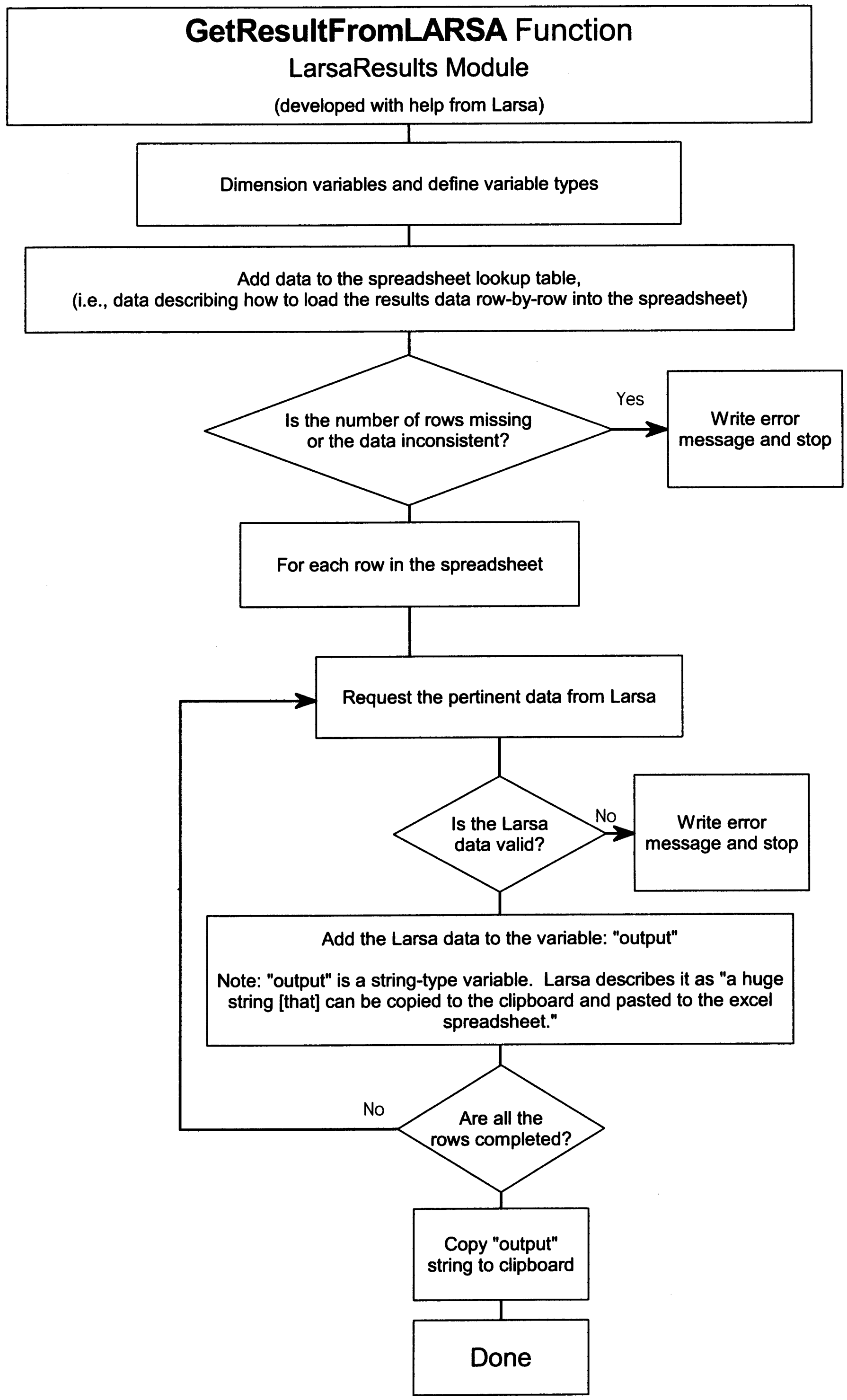
Call Subroutine **Failure_Rating_Stringer_AppFloorBeam** (Module4)
Call Subroutine **Failure_Rating_Arch_Column** (Module5)
Call Subroutine **Failure_Rating_Brace_Member** (Module6)
Call Subroutine **Failure_Rating_Bent** (Module7)
Call Subroutine **Failure_Rating_Prismatic_Girder_1** (Module11)
Call Subroutine **Failure_Rating_Prismatic_Girder_2** (Module12)
Call Subroutine **Failure_Rating_Arch_Floor_Beam_and_Haunch_Member_1** (Module14)
Call Subroutine **Failure_Rating_Arch_Floor_Beam_and_Haunch_Member_2** (Module15)

Done

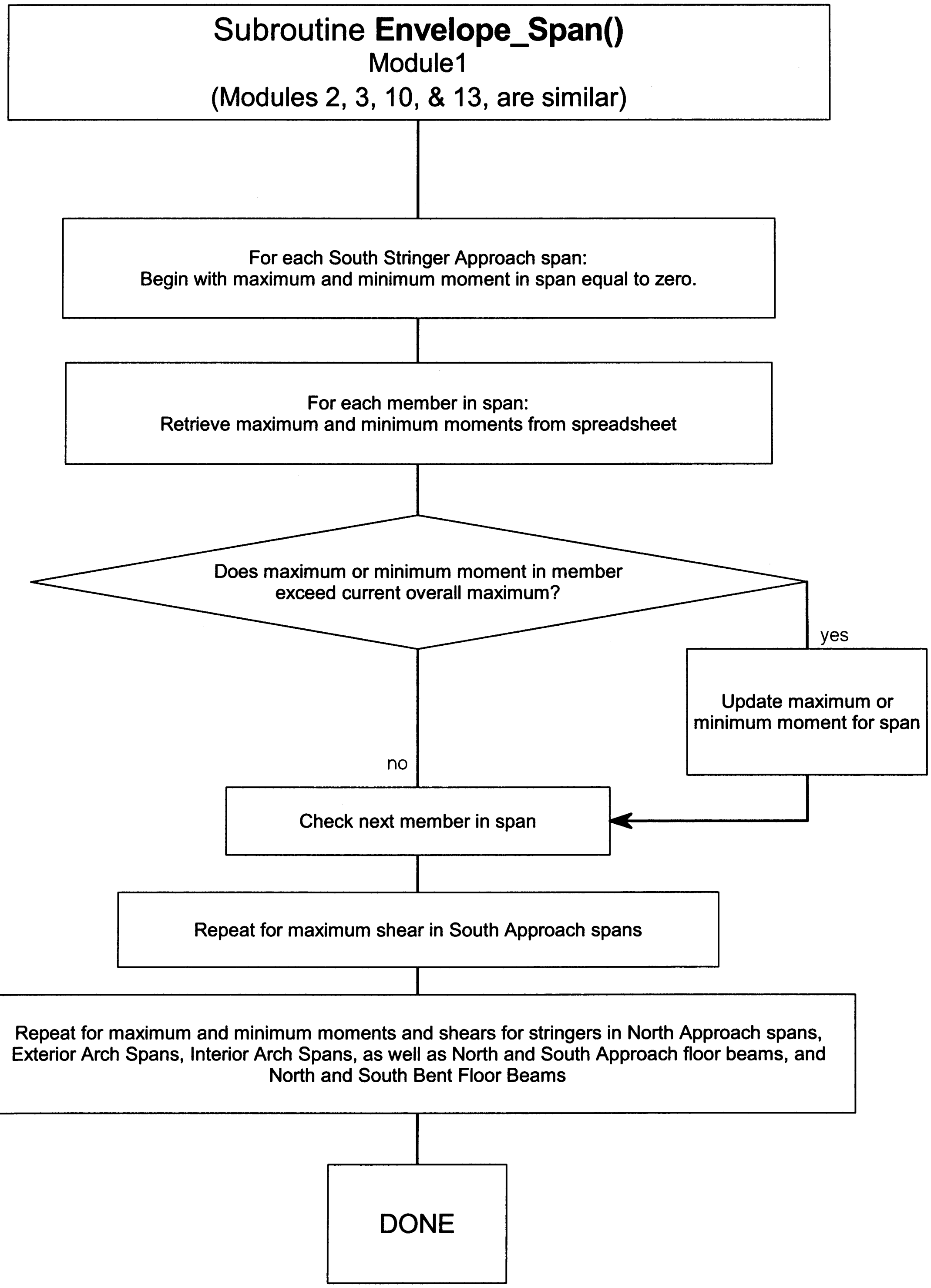
Q.2 Populate Spreadsheet



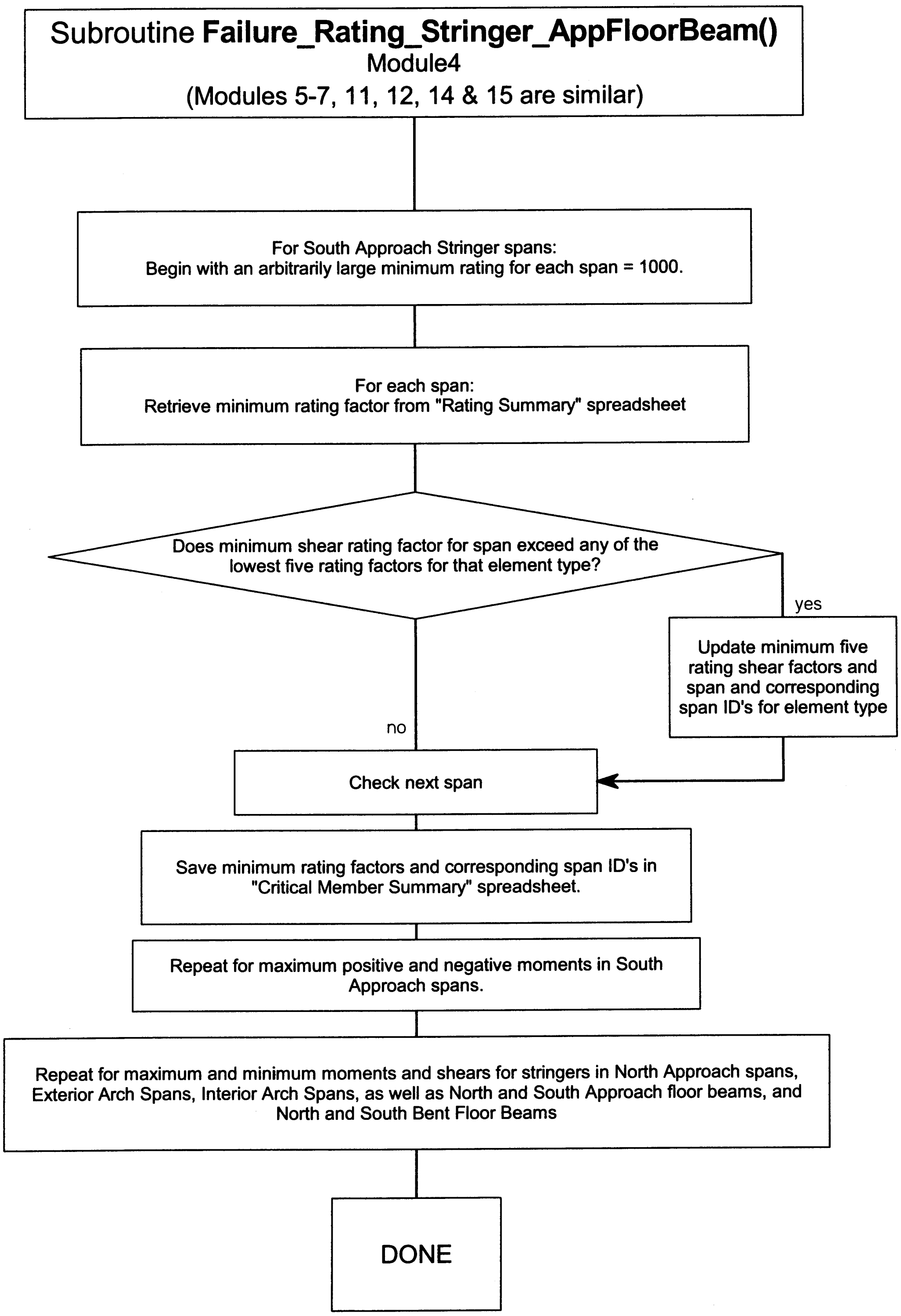
Q.3 Get Results From Larsa



Q.4 Envelope Span



Q.1 Failure Rating



Appendix R. File Management

Input and output files are created each time the rating process is repeated, and systematic procedures will be needed to manage the resulting data.

R.1 LARSA Input Files

Each truck entered for a moving load is saved as a separate record in the LARSA database: "movedata_Perrine_Bridge.dml." Typically this database is saved in the same directory as the "PERRINE BRIDGE MASTER COPY" file.

The "movedata_Perrine_Bridge.dml." should be periodically archived. To archive this database:

1. Copy the database to an archive directory or CD:
   a. Open Windows Explorer and go to the working directory containing the database and right click on "movedata_Perrine_Bridge.dml"
   b. Select "copy" from the drop-down menu.
   c. "Paste" the copy in the desired archive directory.
   d. Rename the archived file, possibly with a filename indicating when the archive was made.
   e. If the archive directory is a CD, write (i.e., "burn") the files on a blank CD.

After the database has been archived and renamed it can be replaced by the "original" database containing only the standard rating trucks, reducing the number of trucks in the database.

2. Retrieve a copy of the original database:
   a. Open Windows Explorer and select the CD (or a copy of the CD) provided by the University of Idaho. Open the directory containing the original moving load database and right click on "movedata_Perrine_Bridge.dml"
   b. Select "copy" from the menu.
3. Paste the copy of the original moving load database in the working directory.
   a. In the directory where the working moving load database was found, select "paste"
   b. When prompted, select "copy and replace" to overwrite the existing database.

LARSA will open the moving load database, which will only contain the standard rating trucks. New permit trucks can be added as needed. If LARSA does not automatically link to the moving load database, the link to that database will have to be redefined (see Appendix E).

R.2 Excel Output Files

As suggested inAppendix O, a new Excel rating file will be created for every rating analysis. In some cases, several rating cases will be considered for a single truck to allow for different lanes, trip conditions, or vehicle speeds. Each of the Excel rating files is large (about 95 MB). A systematic archiving process will be needed to manage the large amount of data this generates. File compression significantly reduces the size of the Excel rating files.