CULVERT RATING GUIDE

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Culvert Rating Guide

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The members of the Project Monitoring Committee selected to monitor this project and to review this document were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the TxDOT Research and Technology Implementation Office.

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I. INTRODUCTION

A. PURPOSE

The purpose of this Culvert Rating Guide is to present a clear, repeatable and valid procedure for Texas Department of Transportation (TxDOT) engineers and their consultants to use for load rating culverts in the TxDOT roadway system.

The American Association of State Highway and Transportation Officials (AASHTO) defines "load rating" as the maximum truck tractor tonnage, expressed in terms of HS load designation, permitted across a bridge [culvert]. The load rating is expressed in terms of two separate ratings – an Inventory Rating and an Operating Rating. The Inventory Rating (IR) is the maximum truck load that can safely utilize the bridge [or culvert] for an indefinite period of time (AASHTO, 2003; TxDOT, 2002). The Operating Rating (OR) is the absolute maximum permissible truck load that may use the bridge [culvert]. Load ratings are based on the culvert's current condition and are determined through analysis and engineering judgment by comparing the culvert structure's capacity and dead load demand to live load demand.

B. TXDOT'S CULVERT DESIGN HISTORY

Culvert design has evolved throughout TxDOT's history. TxDOT archives reveal four eras of culvert design, each representing substantively different design approaches. These are the 1938 era, the 1946 era, the 1958 era, and the 2003 era.

Culvert designs from the 1938 era were designed using slightly unconservative earth loads, lower truck loads, but overly-conservative concrete construction that resulted in very durable culverts. The 1938 collection consists of 428 different culvert designs representing a diverse range of span lengths, number of spans, and barrel heights. Fill depths typically range from 0 to 6 feet.

During the mid-1940s, principally when the Farm-to-Market road system was being constructed, new culvert designs (59 total) were added to the body of 1938 designs. The 1946 era culverts were issued under the less conservative structural codes of the Texas Highway Department Supplement No. 1. These designs resulted in culverts which generally perform well, but which are not as robust as culverts designed per current AASHTO standards.

In 1958, coincident with the advent of the Interstate Highway System, TxDOT redesigned and reissued their full set of culvert construction drawings. The 1958 set consists of 380 designs representing a diverse range of span lengths, number of spans, and barrel heights, with fill depths from 0 to 6 feet. The 1958-era designs use slightly less conservative soil loads but more conservative structural considerations and HS-20 truck loads.

The most recent era of culvert designs dates from 2003. Once again TxDOT redesigned, expanded, and reissued their complete set of culvert construction drawings. The 2003 set consists of 610 culvert designs, including new designs for deep fill culverts with fill heights up to 23 feet. Culvert designs for the 2003 era are based on current AASHTO policy.

Although a culvert is only constructed one time, culvert inspection and rating is an ongoing activity which occurs periodically throughout the culvert's service life. Due to the historical differences and evolution in culvert design, it is necessary to articulate a uniform load rating procedure consistent with current AASHTO policy and which considers the fact that most of TxDOT's culverts have been performing adequately across the design generations.

C. Scope

This Culvert Rating Guide focuses on reinforced concrete box culverts, this being the culvert type most frequently used by TxDOT. The Guide is written to facilitate load rating for reinforced concrete box culverts from any design era, for any number of spans, for any culvert geometry, and for any range of backfill heights.

The procedures described herein specifically apply to load-rating in-service culverts with drained backfill conditions. It is assumed that culverts which are being load-rated will have had a visual inspection to establish the condition rating of the culvert.

This Guide includes limited information about load rating for alternative culvert structures with respect to shape (circular pipe, arch, etc.), material (aluminum, plastic, steel, etc.), backfill type, and drainage.

D. Rating Philosophy

Load ratings are determined by comparing the culvert capacity and dead load demand to the live load demand. Thus, the culvert load rating is strongly dependent on how culvert capacity, culvert dead load, and culvert live load are established.

Typical practice is to determine culvert capacity based on the details found on the original construction documents in combination with historical material property assumptions which are correlated by visual inspection of the culvert condition. The dead and live load demands on the structure are determined by analytical modeling. This means that the culvert load rating process requires engineering decisions about modeling practices and procedures, as well as the knowledgeable evaluation and selection of numerous design variables.

TxDOT policy provides guidance for many aspects of culvert load rating. The official policy concerning culvert load rating used by TxDOT is embodied in the AASHTO *Standard Specification for Highway Bridges* 17th Edition (*SSHB*) (AASHTO, 2002), and the AASHTO *Manual for Condition Evaluation of Bridges* (*MCEB*) (AASHTO, 2003).

Other aspects of the culvert load rating process are not directly addressed by policy, one example being selection of the analytical model for determining dead load and live load demand. Modeling can be approached in many ways, with each analytical method requiring its own degree of effort and yielding results that predict actual load demands with varying degrees of accuracy.

Recognizing the range of approaches available for culvert modeling, to promote efficiency this Guide identifies four analytical models of increasing complexity and sophistication. The first level is a “quick” calculation using a stylized two dimensional frame model. The second level uses a traditional two dimensional frame model with the culvert structure supported by soil springs. The third level uses a two dimensional finite element model that considers soil-structure interaction effects. That is, the soil surrounding the culvert structure is modeled with finite elements. With the guidance presented herein and an appropriate software package, any structural engineer should be able to confidently load rate a culvert using the first three levels.

The fourth level is the general case. Here, applications are open-ended and highly project-specific, and use is restricted to research or specialized applications. Level 4 assumes the use of more complex, two or three-dimensional finite element models with soil-structure interaction. The selection of modeling approach, model details, and rating parameters will be individualized and largely left to the discretion of the engineer rating the culvert.

E. About this Guide

It is the intent of this Culvert Rating Guide to assemble, summarize and clarify the necessary information for culvert load rating, both that portion specifically addressed by policy and that portion which is not, for each of the four levels of analysis. The chapters are as follows:

Chapter I provides an introduction and background to culvert load rating at TxDOT.

Chapter II is devoted to culvert rating policy. This chapter identifies the governing policy associated with culvert load rating, and summarizes applicable policy guidance.

Chapter III outlines the culvert rating procedure. Whereas the first two chapters provide pertinent background information, this chapter lays out how culverts should be load rated at TxDOT. This includes a flow chart which summarizes the culvert rating process.

Chapter IV presents the initial step for load rating a culvert. Beginning with culvert plans, construction details, and related documentation, this chapter shows how to obtain the necessary dimensional and material property data needed for culvert load rating.

Chapter V discusses culvert capacity calculations. For culvert load rating, capacity is based on equations and approaches specified in AASHTO policy. This chapter presents both the policy and a straightforward approach for determining culvert capacity to facilitate the load rating calculation.

Chapter VI presents the Level 1, Level 2, and Level 3 analytical modeling approaches recommended for determining demand loads associated with culvert load rating. The discussion addresses the assumptions associated with each modeling level, specification of the analytical model, assigning boundary conditions, and defining and applying dead load and live loads. This chapter also discusses available structural analysis software packages, and identifies the most common software package used in TxDOT for each level. Finally, this chapter presents a detailed, step-by-step procedure for calculating demand loads using the representative software package.

Chapter VII discusses Level 4 modeling. This includes generalized guidance about applications, modeling approaches, software selection, and analytical procedures.

Chapter VIII discusses limitations associated with use of this Guide. These include culvert type, deep fill culverts, submerged culverts, saturated soils and backfill soil modulus values.

This Guide includes six appendices. Appendix A presents an example of how to accomplish the first step in culvert rating; that is, obtaining the structural rating parameters from the design drawings. Appendix B continues this example, explaining how to calculate the culvert capacity. Appendices C through E continue the example by presenting how to perform demand load calculations and culvert load rating based on Level 1, Level 2, and Level 3 modeling approaches. Appendix F presents the policy source documents for culvert load rating.

II. Policy Requirements

A. Policy Source Documents

As noted, two documents present TxDOT's official policy for culvert load rating. These are:

- AASHTO *Standard Specification for Highway Bridges* 17th Edition (*SSHB*) (AASHTO, 2002)
- AASHTO *Manual for Condition Evaluation of Bridges* (*MCEB*) (AASHTO, 2003)

AASHTO's *SSHB* provides guidance for basic rating parameters including dead and live load values and distributions and strength reduction factors. AASHTO's *MCEB* provides the actual rating equations, load factors and material property assumptions. The policies do not provide direct guidance for analytical modeling.

TxDOT has published two other documents which refer to culvert design and rating. These documents are:

- *TxDOT Bridge Design Manual* (TxDOT, 2001)
- *TxDOT Bridge Inspection Manual* (TxDOT, 2002)

These documents provide culvert rating and design guidance that differs slightly from the AASHTO specifications. It is TxDOT's policy to satisfy the current AASHTO requirements. Therefore, the TxDOT publications should be referenced only for historical interest and clarity while the AASHTO standards should be relied upon for official policy. Appendix F contains scans of the actual policy documents.

The following sections of this document outline and interpret the AASHTO policy requirements for culvert load rating at TxDOT.

B. Failure Modes

The AASHTO *SSHB*, Section 8, defines three potential failure modes for culvert structural members. These are bending moment (or flexure), shear, and axial thrust. Culvert load rating calculations must consider all three failure modes, though typically, bending moment is the controlling case.

C. Critical Sections

Culverts can be modeled in two dimensions by taking a one-unit-wide "slice" normal to the culvert flowline as shown in Figure II-1. Several cross sections from this slice must be analyzed for both capacity and demand in order to establish the load rating for a culvert structure. Multiple load ratings must be calculated, with the lowest load rating from all cross sections becoming the culvert load rating.

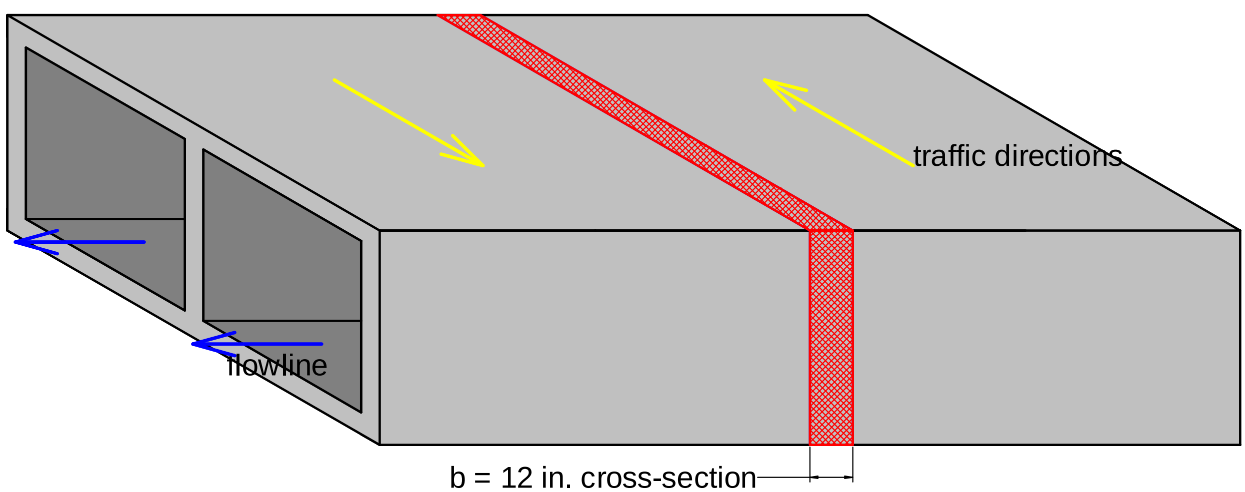

FIGURE II.1. THREE DIMENSIONAL VIEW OF A CULVERT INDICATING TWO DIMENSIONAL STRIP.

Experience suggests that the controlling location for culvert load rating, known as the critical sections, will typically be either near mid-span or at a corner of the culvert structure. According to the *SSHB* Section 16.6.4.5, the corner capacity and demand for moment may be taken "at the intersection of the haunch and uniform depth member" (AASHTO, 2002). In the case of culverts without haunches, it is taken at the face of the wall section. Figure II-2 summarizes the critical section locations for culvert load rating for culverts without haunches (Figure II-2.A) and culverts with haunches (Figure II-2.B).

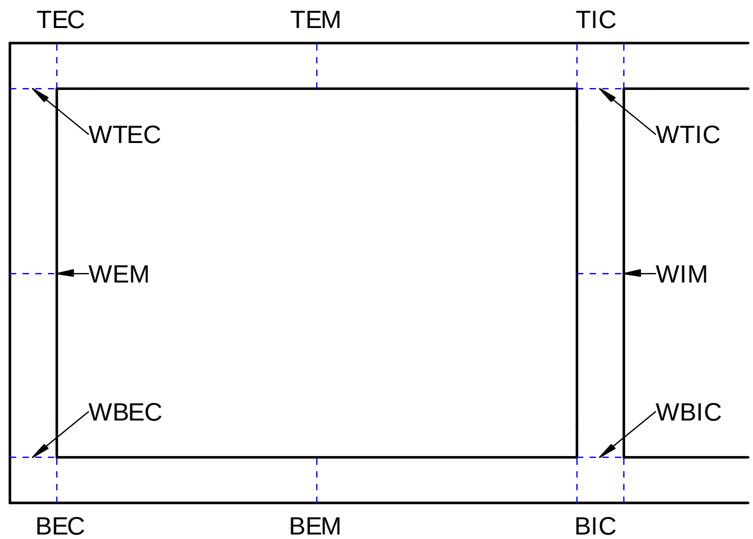

FIGURE II.2.A. MOMENT CRITICAL SECTIONS FOR CULVERTS WITHOUT HAUNCHES.

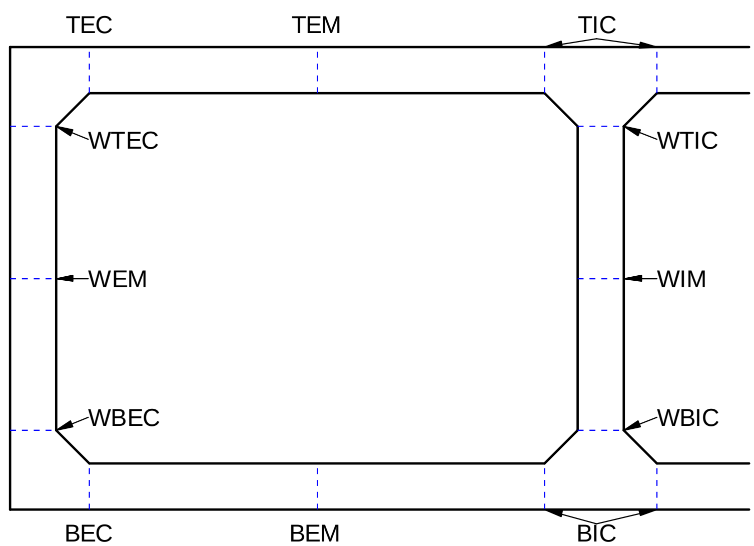

FIGURE II-2.B. MOMENT CRITICAL SECTIONS FOR CULVERTS WITH HAUNCHES.

Abbreviations for the typical critical sections shown in Figure II-2, listed clockwise, are: top exterior corner (TEC), top exterior mid-span (TEM), top interior corner (TIC), top interior mid-span (TIM), wall top interior corner (WTIC), wall interior mid-span (WIM), wall bottom interior corner (WBIC), bottom interior mid-span (BIM), bottom interior corner (BIC), bottom exterior mid-span (BEM), bottom exterior corner (BEC), wall bottom exterior corner (WBEC), wall exterior mid-span (WEM), and wall top exterior corner (WTEC). For multiple-span box culverts, the sections are designated as per the culvert span; e.g., TIC1, TIC2, BIC1, BIC2, etc.

The mid-span capacity and demand may, for convenience, be taken *at mid-span* for both top and bottom slabs and vertical walls. Technically, the analysis should identify the actual locations with the highest demand, but the error introduced by assuming the mid-span location is not significant.

Also, it is important to note that the corner critical sections shown in Figure II-2 correspond to desired locations for the calculation of bending moment capacity and demand, as opposed to shear. As has been noted, bending moment is the most common controlling failure mode for culvert load rating, so it is standard practice to use the moment critical sections for all three potential failure modes (moment, shear, axial), at least initially. This is a conservative approach when bending moment controls the rating, and it requires the least amount of effort. However, shear sometimes controls or appears to control the load rating. Section VI.C explicitly discusses the situation where shear appears to control the load rating, and provides AASHTO policy guidance on how to evaluate shear capacity and demand in that instance.

D. Load Cases

The AASHTO *SSHB*, Section 3.20.2, requires that two demand analyses must be made to determine the worst case loading condition for the culvert structure. These are the “total” load case and the “reduced lateral” load case. The total load case is designed to generate the maximum shear and axial demands in the whole culvert and the maximum moment demands in all but the top and bottom mid-spans. The reduced lateral load case is intended to generate the maximum moment demands in the top and bottom mid-spans. The load rater defines these load cases by combining basic dead and live loads differently. Section VI.D.5, provides detailed guidance about these load cases.

E. Rating Variables

AASHTO policy documents provide rating parameters in addition to location of the critical sections. Table II-1 summarizes the basic rating variables specified by policy. These variables provide guidance for either the demand or the capacity calculations.

TABLE II-1. CULVERT LOAD RATING VARIABLES PROVIDED BY POLICY

Description	***Value***	***Policy Section***
Demand Variables		
Live Load	HS-20	SSHB 3.7.6
Impact Factor	0 < D < 1' IM = 30% 1' < D < 2' IM = 20% 2' < D < 3' IM = 10% 3' < D IM = 0%	SSHB 3.8.2.3
Reduction in Load Intensity	1 or 2 lanes 100% of LL 3 lanes 90% of LL 4 lanes 75% of LL	SSHB 3.12.1
Near-Structure Lateral Live Load	additional 2 feet of surcharge to lateral load	SSHB 3.20.3
Vertical Earth Pressure	Calculate based on total unit weight for soil of 120 pcf	SSHB 6.2.1.B
Lateral Earth Pressure – Corner Moment	Calculate based on equivalent fluid weight for soil of 60 pcf	SSHB 6.2.1.B
Lateral Earth Pressure – Positive Moment	Calculate based on equivalent fluid weight for soil of 30 pcf	SSHB 6.2.1.B, 3.20.2
Live Load Distribution to the Top Slab (fill depth, D < 2')	direct contact	SSHB 6.4
Live Load Distribution to the Top Slab (fill depth, D > 2')	wheel load distributed over a square 1.75D to a side; overlapping areas are averaged	SSHB 6.4
Live Load Distribution to the Bottom Slab	wheel load distributed over a rectangle 1.75D by 1.75D + 2H	SSHB 16.6.4.3
Capacity Variables		
Shear Strength Reduction Factor	$\phi = .85$	SSHB 16.6.4.6
Flexure and Thrust Strength Reduction Factor	$\phi = .9$	SSHB 16.6.4.6
Assumed Concrete Strength, f'_c	Pre-1954 2,500 psi Post-1954 3,000 psi	MCEB 6.6.2.4
Assumed Reinforcing Steel Strength, F_y	Pre-1954 33 ksi Structural Gr. 36 ksi Gr. 40 40 ksi Gr. 50 50 ksi Gr. 60 60 ksi	MCEB 6.6.3.2

Where:

D is the depth of fill
IM is the impact factor
LL is the live load.

F. Rating Equations

The equations which determine the load rating and culvert capacity are provided by the AASHTO policy. Within the AASHTO policy, culverts are considered as a subset of bridges and are therefore load rated as a subset of bridge load rating.

The AASHTO MCEB provides the actual load rating equations. The rating factor identified in Equation II-1 is the central element of the culvert load rating process. This rating factor is the ratio of the structural capacity minus the dead load demand to the live load demand.

Equation II-1 is used to determine the rating factor at each critical section as identified in Figure II-2 for each potential failure mode (moment, shear and thrust), for each load case (total and reduced lateral), at each rating level (inventory and operating). The lowest inventory rating factor and the lowest operating rating factor control the load rating for the culvert.

EQUATION II-1. THE RATING FACTOR EQUATION (*MCEB* 6.5.1 EQ.6-1A).

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)}$$

where: *RF*= the rating factor
C = the structural capacity of the member
D = the dead load effect on the member
L = the live load effect on the member
I = the impact factor, IM from *SSHB* 3.8.2.3
A_1= 1.3 = factor for dead loads, from *MCEB* 6.5.3
A_2= 2.17 for Inventory Level = factor for live loads, from *MCEB* 6.5.3
= 1.3 for Operating Level = factor for live loads, from *MCEB* 6.5.3

Once the controlling (lowest) rating factors for the inventory and operating conditions are calculated, the inventory and operating load rating can be determined by multiplying the rating factor by the HS rating truck tractor tonnage as seen in Equation II-2:

EQUATION II-2. THE LOAD RATING EQUATION (*MCEB* 6.5.1 EQ.6-1B):

$$RT = RF * W$$

where: *RT*= the load rating in terms of an HS truck tonnage
RF= the rating factor from Equation II-1
W= the HS truck tractor tonnage; for HS-20, W = 20 tons

Note that the variables used in Equation II-2 are specific to HS loading as per customary TxDOT practice. This means that the load rating (RT) will be expressed in terms of an HS-designation rather than the gross weight of the vehicle.

III. Culvert Load Rating Procedure

A. Context for Load Rating

The typical situations at TxDOT for which it becomes necessary to load-rate a culvert are:

- The culvert fails visual inspection during the bi-annual bridge inspection cycle. This means the culvert receives a Condition Rating of 5 or less as described in the TxDOT *Bridge Inspection Manual* (TxDOT 2002).
- The culvert needs to be lengthened or otherwise modified as part of a road rehabilitation/ construction project.
- The culvert has structurally deteriorated since its previous inspection.
- The culvert has been structurally damaged due to vehicular impact.
- It is desirable to increase the operating rating (OR) of the culvert to negate the requirement for load posting.

Load posting is defined in Chapter 5 of the TxDOT *Bridge Inspection Manual* (TxDOT 2002) and consists of placing signage by the structure indicating the largest truck that may be permitted across the structure. The following flow chart from the *Bridge Inspection Manual* defines the culvert load posting process. Culverts may be load posted at the operating rating if the culvert condition rating is greater than that defined in the flow chart. Otherwise the culvert must be load posted at the inventory rating.

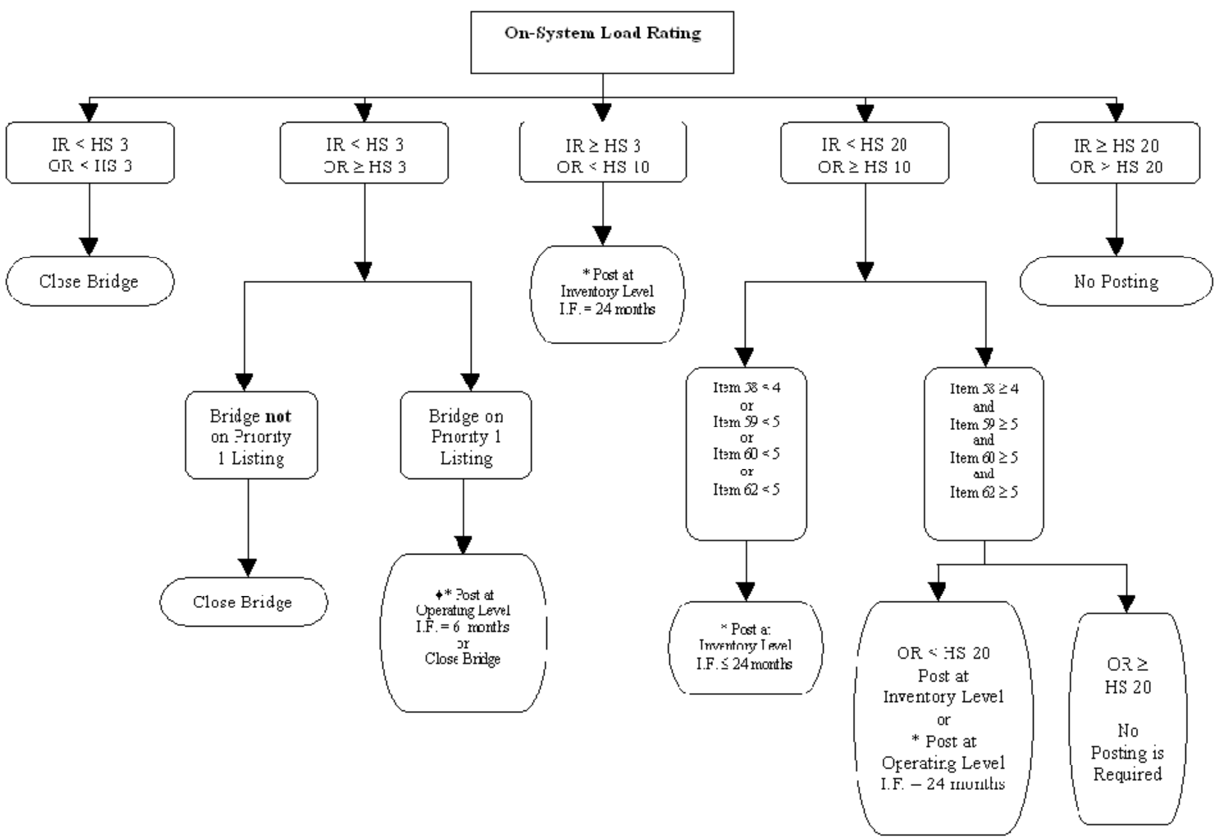

FIGURE III.1. LOAD POSTING GUIDELINES (TXDOT BRIDGE INSPECTION MANUAL FIG.5-3).

B. Visual Inspection of the Culvert

The culvert load rating process is a component of the inspection process and consists of determining the safe load carrying capacity of the culvert structure, determining whether specific legal or overweight vehicles can safely cross the culvert, and determining if the culvert needs to be restricted and what level of posting is required.

The TxDOT *Bridge Inspection Manual* (TxDOT 2002), Section 3.8.8 of the *MCEB* (AASHTO 2003), and Section 9.0 of the FHWA *Culvert Inspection Manual* (FHWA, 1986) provide specific guidance for performing condition evaluations of cast-in-place concrete box culverts. The typical types of distress to check for include vertical and horizontal misalignment of the culvert barrel, joint defects, cracks and spalls, concrete durability, and footing instability.

Section 6.5.4 of the MCEB specifically addresses the relationship between field inspection and the load rating and notes that "the condition and extent of deterioration of structural components of the bridge [culvert] should be considered in the computation of… capacity when force or moment is chosen for use in the basic rating equation." Any discrepancies from plan, or excessive distress such as thin sections, spalling, cracking, deflection, exposed reinforcing steel, and other items which may affect structural capacity, should be noted and considered when establishing actual section capacities.

C. The Culvert Load-Rating Process

The basic load rating procedure is as follows. Per Equation II-1, the main variables are culvert capacity, the dead load demand, and live load demand. Culvert capacity is established from equations set forth in AASHTO policy, whereas dead load and live load demands must be determined by structural modeling (computer analyses). While this seems simple enough, the challenge is to obtain reliable values for each of these variables.

The complexity inherent in the load rating process becomes apparent when one considers that rating calculations must be performed for each potential failure mode (moment, shear, and thrust), for multiple load cases (total and reduced lateral), for each critical section in the model (12 sections for a single-barrel culvert to 50 or more sections for multiple spans), for both inventory and operating conditions. The actual load rating (IR and OR) for the culvert will be the minimum values from these different sets of calculations. This means that to load rate a one-barrel box culvert – the simplest type – the load rater will create at least one computer model, conduct four separate computer analyses based on this model, interpret thousands of data points, and perform no fewer than 144 sets of load rating calculations.

A "road map" of the culvert rating process helps avoid confusion. Figure III-2 presents the load-rating process in terms of a flow chart. The first step is to identify the culvert that will be load rated. As noted, this might be because the culvert failed a scheduled inspection or for some other reason. Either way, a visual inspection of the culvert is necessary. For all intents and purposes, the culvert load rating process begins here.

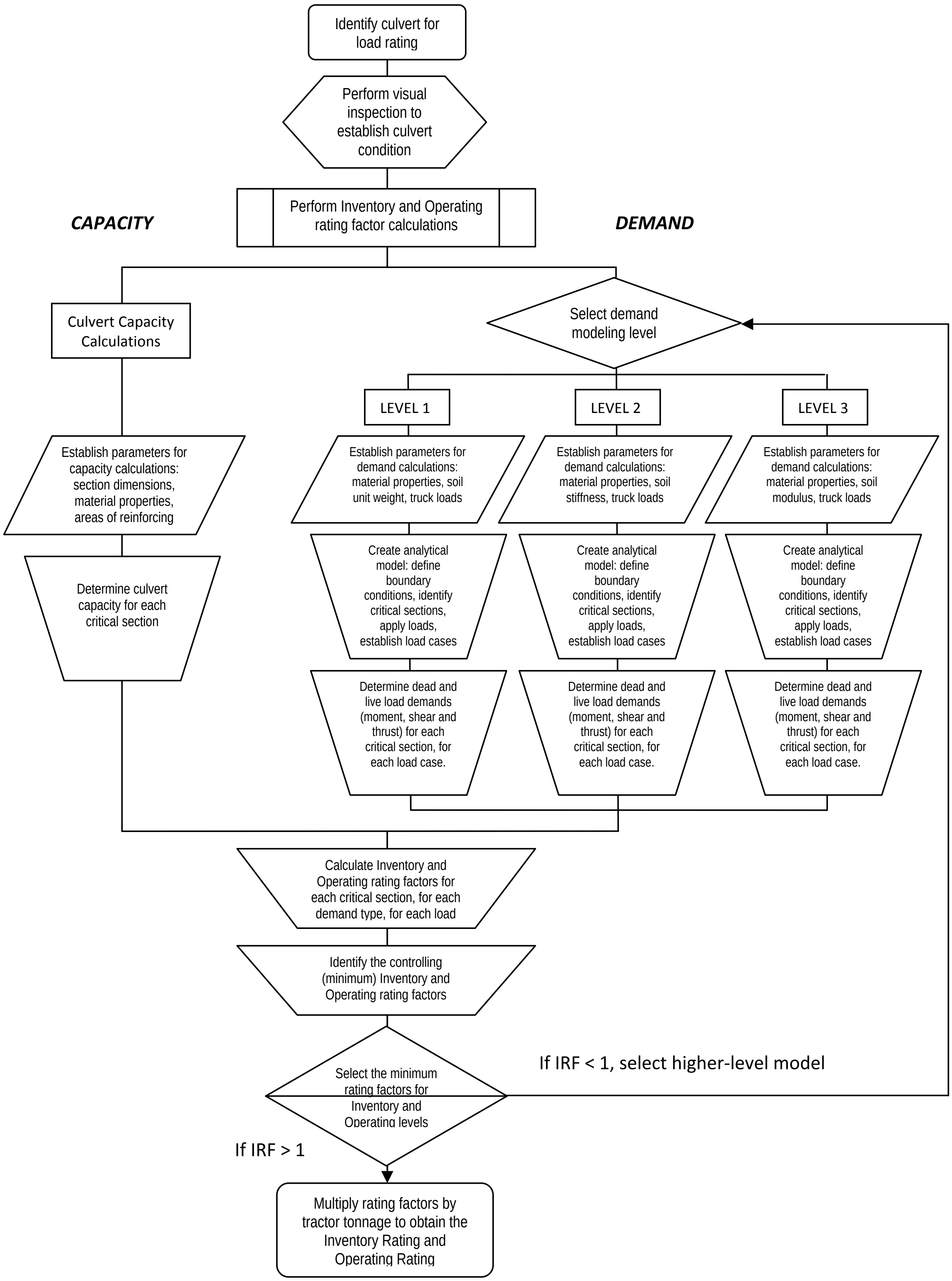

FIGURE III.2. FLOW CHART DEPICTING THE TXDOT CULVERT LOAD RATING PROCESS.

The load rating factor calculations require determination of both culvert capacity and dead and live load demands. It is helpful, therefore, to think of culvert capacity and demands as separate and distinct aspects of the load rating process.

Capacity calculations are based on equations established in AASHTO policy (see Chapter V). These do not require a computer model and are independent of the level of analysis selected for demand calculations. Inputs for capacity calculations are obtained from the construction drawings, visual inspection, and AASHTO policy and consist of strength properties for concrete and steel, culvert section dimensions, and the location and amount of reinforcing steel. The calculations determine moment, shear and thrust capacity for each critical section of the culvert structure.

Determination of dead and live load demands *do* require computer modeling. Thus the first decision to be made is to select the type of analytical model for the load rating process. This Guide describes three levels of analysis, each with increasing analytical sophistication. A trade-off exists between sophistication of analysis and required work effort. The advanced models require more work but typically yield more accurate results. Further comments about the hierarchy of analyses are presented below.

Once the level of analysis is chosen, it is necessary to gather data to facilitate creation of the analytical model. As discussed in Chapter IV of this Guide, modeling parameters include but are not limited to culvert dimensions, properties of the concrete and reinforcing steel, soil parameters, the location and amount of reinforcing steel, and culvert installation details.

With this information, the load rater can create the analytical (computer) model from which s/he will obtain demand moments, shears and thrusts (see Chapter VI). This involves laying out the model, specifying boundary conditions, identifying critical sections, applying loads, and defining load cases.

Determining the inventory and operating load rating factors requires multiple sets of calculations from the computer model. This is because demand loads and their corresponding capacity must be determined for each critical section, for each failure mode, and for multiple load cases. From these sets of calculations, the load rater selects the controlling (minimum) operating and inventory rating factor for each critical section, for each load case. The minimum operating and inventory rating factors from the critical sections are the rating factors for the culvert.

A decision is required at this point. If the inventory and operating rating factors are greater than 1.0, the culvert will not require load posting as per Section 5, Chapter 5 of the TxDOT *Bridge Inspection Manual* (2002). This means that the culvert load rating can be calculated by multiplying the rating factors by the tractor tonnage (for HS-20 trucks, this is 20 tons) to determine the operating (OR) and inventory (IR) load ratings. However, if either the inventory rating factor or the operating rating factor is less than 1.0, the culvert may require load posting. As an alternative to posting, the load rater may elect to perform the calculations again, using a higher level (more sophisticated) modeling approach.

D. Selection of the Proper Level of Analysis

This Guide recognizes a hierarchy of analysis for the demand calculations. The level of analysis chosen is a trade-off between sophistication of analysis and required work effort. The simpler methods are frequently selected as a first choice due to the need to analyze many structures with limited resources. When this analysis yields satisfactory results, there is no need to use a more sophisticated model. Satisfactory results would be the establishment of safe load carrying capacity that does not require posting the structure and does not unduly restrict the flow of permitted overweight trucks. A more sophisticated analysis is justified to avoid posting the structure or to ease restrictions on the flow of permitted overweight trucks.

Of course, the goal of the load rating process is not to force any particular culvert to "rate", but instead to establish a valid load rating for the culvert. The fact that more sophisticated models tend to more accurately model the moment, shear and thrust demands, and thus yield higher rating factors, cannot be pressed indefinitely. Load rating should reliably depict actual or expected culvert performance. Culverts which cannot safely support design traffic loads should be rated accordingly, and culverts which are not performing in a manner that indicates they can carry design traffic loads should not be rated as if they can.

E. Reviewing and Checking Calculations

The load rating process recognizes a balance between safety and economics. Standard quality control procedures require that both in-house and consultants' load rating results should be checked for accuracy.

Load rating analyses must be performed under the direct supervision of a Licensed Professional Engineer who is knowledgeable about the load rating process. Whenever possible, the load rater should perform long hand checks of a portion of the computer analysis to satisfy the load rater that the computer output is valid. It is of utmost importance that the load rater understands when computer results are reasonable. Blind faith in any computer program should be avoided.

An independent check of the analysis should be performed. The checker should verify all input data for computer programs, verify that the summary of load capacity information accurately reflects the analysis, and be satisfied with the accuracy and suitability of the computer output.

IV. Culvert Details

A. Overview

In preparation for the load rating calculation process, culvert-specific variables must be established. It is common practice for most of the variables to be taken directly from the construction documents. These include culvert dimensions, material properties, reinforcing schedules and installation methods. However, as noted in the previous chapter, data from the construction documents should be confirmed during a visual inspection of the culvert, and any discrepancies from the construction documents should be addressed.

B. Units

It is appropriate to comment on units, both for measurement and analysis. Consistent with TxDOT practice, this Guide presents U.S. Customary Units throughout. Gross culvert dimensions including the clear span, clear height, and depth of cover soil should be measured in feet. Culvert structural dimensions, wall thicknesses, etc. should be measured in inches. The units of measurement will be the units of analysis.

With respect to material properties, concrete strengths should be presented and analyzed in terms of pounds per square inch (psi). Reinforcing steel strength should be presented and analyzed in terms of kips per square inch (ksi).

With respect to loads, soil unit weight is identified in terms of pounds per cubic foot (pcf) but analyzed in kips per cubic foot (kcf). Vehicle live loads are presented in kips, converted to stress in terms of kips per square foot (ksf), and analyzed in kips per square foot (ksf). Spacing between wheel loads should be presented and analyzed in feet.

Output data from the analytical programs (shear, moment, thrust) are presented in terms of kips and feet. Load ratings are expressed in terms of the HS tractor tonnage.

C. Dimensions

The culvert dimensions must be collected from the construction documents or established based on field measurements. The required dimensional information is summarized in Table IV-1 and Figure IV-1. These values will be adequate for defining the gross section properties for all levels of analytical modeling.

TABLE IV-1. CULVERT DIMENSIONS REQUIRED FOR LOAD RATING CALCULATIONS

Dimension	*Abbr.*	*Units*
number of spans	N	-
cover soil depth	D	ft
clear span	S	ft
clear height	H	ft
exterior wall thickness	T_{EW}	in.
interior wall thickness	T_{IW}	in.
top slab thickness	T_T	in.
bottom slab thickness	T_B	in.
top haunch dimension	F_T	in.
bottom haunch dimension	F_B	in.

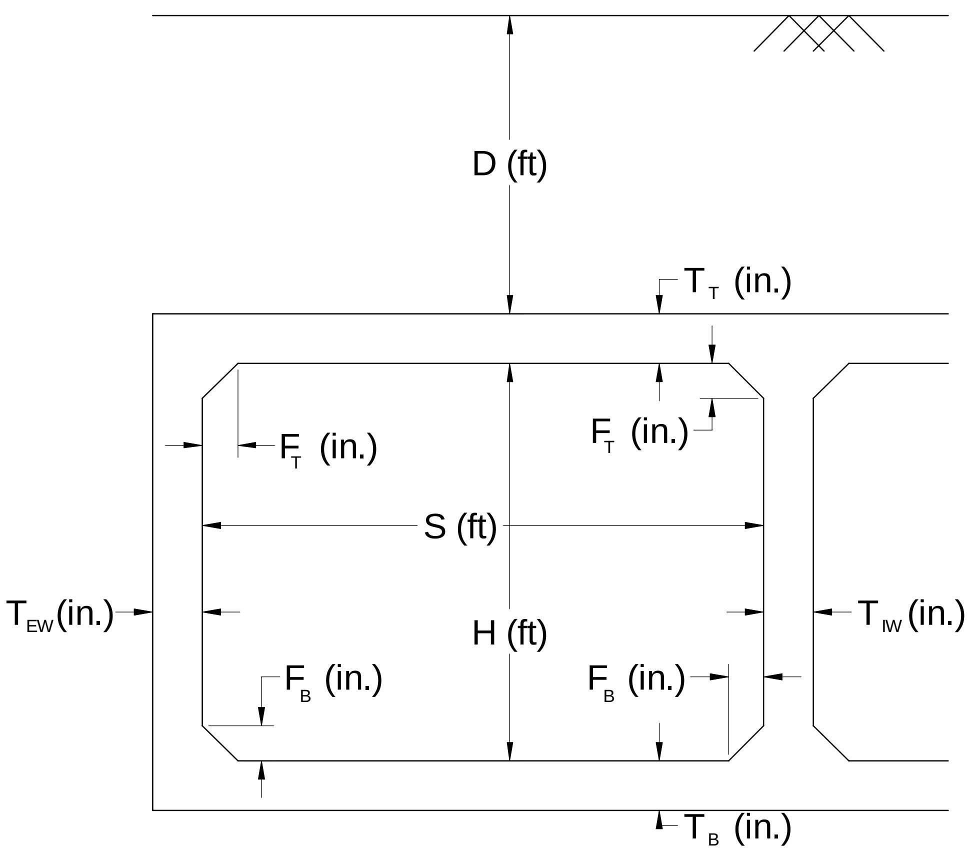

FIGURE IV.1. CULVERT DIMENSIONS.

D. Material (Structural) Properties

The required material properties are the yield strengths and modulus of elasticity for concrete and steel. The load rater should use the best (i.e., most accurate) material property information available for the constructed culvert.

In the absence of project-specific information from design plans or as-built drawings, the default material properties, per policy, are shown in Table II-1.

Most of the time original construction documents will be available which will identify the material properties. It is customary to rely on this information. In some cases, construction records including quality control data for concrete will be available. The load-rater should use the best available information.

In certain cases, it may be appropriate to obtain samples of the actual culvert materials and determine material properties based on laboratory tests. TxDOT test procedures published in the Departmental Material Specifications specify how samples should be taken and how these types of tests should be performed.

E. Soil Parameters

Soil parameters affecting culvert load rating primarily consist of the weight of soil around the culvert and the stiffness of soil used to provide bearing and lateral support to the culvert structure.

1. Soil Unit Weight

Regardless of level of analysis, soil weight is one of the applied loads for the culvert loading model. In the case of a Level 1 analysis, it is the *only* required soil parameter. The unit weight of soil for culvert load-rating analyses is defined per AASHTO policy and presented in Table II-1. Although it is possible to conduct a geotechnical exploration and directly measure the unit weight of the soils which are over, around and beneath the culvert, this is rarely done for culvert load rating. Recommended practice is to use the AASHTO value for soil unit weight.

2. Modulus of Subgrade Reaction

The Level 2 analysis models the culvert as being supported by soil "springs." The soil property used to define the stiffness of these springs is the modulus of subgrade reaction, k. The k-value used for analysis should represent the soil upon which the culvert is built; that is, the soil directly *beneath* the culvert. Table IV-2 provides representative k-values for low, medium, and high-strength soil. Use of this table requires at least a basic idea about the type of soil upon which the culvert was constructed, expressed in terms of soil classification by ASTM, AASHTO, or TxDOT methods.

TABLE IV-2. MODULUS OF SUBGRADE REACTION (K-VALUES) FOR LEVEL 2 MODEL.

Soil Support Description	Modulus of Subgrade Reaction, *k* (pci)	Unified Soil Classification (ASTM D 2487)	AASHTO Group Classification (AASHTO M 145)	Texas Triaxial Classification (TEX-117-E)
Low: Fine-grained soils in which highly-plastic silt and clay-sized particles predominate	75	CH, OH, MH, OL	A5, A6, A7, A8	> 5.0
Medium: Sands and sand-gravel mixtures with moderate amounts of silts and clay	150	CL, ML, SC, SP, SM	A3, A4	3.5 to 5.0
High: Gravels and sand-gravel mixtures relatively free of plastic fines	250	GW, GP, GM, GC, SW	A1, A2	< 3.5
(VanTil, 1972; Bowles, 1996; McCarthy, 2002)				

Typical practice for culvert load rating is to select a representative value for k from the table, or to estimate k based on correlation with other soil properties. Published data from research studies of beams on elastic foundations indicate that demand moments are not particularly sensitive to the k value(Bowles, 1996; McCarthy, 2002). Parametric analyses on a sample of TxDOT's culvert designs support this view. Having said that, if the load rater feels it is necessary, the k-value for soils supporting the culvert can be determined directly by performing a plate bearing test as per Test Method TEX-125-E.

3. Soil Elastic Modulus and Poisson's Ratio

The Level 3 culvert load rating analysis accounts for soil-structure interaction effects by modeling soil using linear elastic finite elements. The soil parameters used to define these finite elements are the elastic modulus (i.e., Young's modulus) and Poisson's ratio.

Given the stiffness of concrete box culverts, it is reasonable to assume that structure deflections will be very small and soil displacement will be correspondingly limited, thus the appropriateness of the linear-elastic approach. Of course, more sophisticated linear and non-linear soil constitutive models are available and can be used for specialized applications. These are discussed in Chapter VII relative to Level 4 analyses. But for a Level 3 culvert rating calculation, the Poisson's ratio and elastic modulus parameters are sufficient.

a. Poison's ratio

The Poison's ratio, ν, for soil ranges from 0.10 to 0.50. An acceptable, recommended value is 0.30. Parametric analyses on a sample of TxDOT's culvert designs support the use of 0.3 for Poisson's ratio, with one exception. Inventory ratings for deep fill culverts (fill heights greater than 6 feet) with large wall heights (greater than 8 feet) are sensitive to Poisson's ratio. For these conditions, it would be appropriate to determine Poisson's ratio based on knowledge of the actual soil backfill type. Published data (Bowles, 1996) suggest that clayey backfill soils for deep fill/large wall height culverts should be modeled using a Poisson's ratio of 0.50, and sandy backfill soils for such culverts should be modeled using a Poisson's ratio of 0.30.

b. Soil Modulus of Elasticity

The elastic modulus, E_{soil}, should represent the soil conditions above the culvert (vertical), beside the culvert (lateral), and below the culvert (bearing). It is possible to model different soil zones for each of these areas, to distinguish between native soil and backfill soil, and to introduce other refinements when delineating the soil model. In the absence of a project-specific subsurface soil profile establishing the soil zones, the load-rater may assume homogenous soil conditions around the culvert structure.

Table IV-3 provides representative E_{soil} values for low, medium, and high-strength soil. It is emphasized that modulus values vary widely for a given soil type and actual values depend on factors such as moisture content, unit weight, compressibility, stress level, etc. Use of this table requires at least a basic idea about the type of soil, expressed in terms of soil classification by ASTM, AASHTO, or TxDOT methods.

Table IV-3. Modulus of Elasticity for Soil for Level 3 Model.

Soil Support Description	Elastic Modulus E_{soil} (psi)	Unified Soil Classification (ASTM D2487)	AASHTO Group Classification (AASHTO M 145)	Texas Triaxial Classification (TEX-117-E)
Low: Fine-grained soils in which highly-plastic silt and clay-sized particles predominate	*Range*: 2,500-25,000+ *Typical:* 8,000	CH, OH, MH, OL	A5, A6, A7, A8	> 5.0
Medium: Sands and sand-gravel mixtures with moderate amounts of silts and clay	*Range*: 5,000-50,000+ *Typical:* 20,000	CL, ML, SC, SP, SM	A3, A4	3.5 to 5.0
High: Gravels and sand-gravel mixtures relatively free of plastic fines	*Range*: 10,000-70,000+ Typical: 36,000	GW, GP, GM, GC, SW	A1, A2	< 3.5
(NAVFAC, 1986; VanTil, 1972; Coduto, 2001; Bowles, 1996)				

Site-specific determination of the soil modulus is desirable for culvert load rating applications. Modulus values for soils may be estimated from empirical correlations, laboratory test results on undisturbed specimens, and from results of field tests. Laboratory tests that may be used to estimate the soil modulus are the California Bearing Ratio test, unconsolidated-undrained triaxial compression test, or the consolidated-undrained triaxial compression tests. Field tests include the static cone penetration test (CPT), standard penetration test (SPT), Texas cone penetrometer (TCP), dynamic cone penetrometer (DCP) test, falling weight deflectometer (FWD) test, and the pressuremeter test (PMT).

Parametric analyses on a sample of TxDOT's culvert designs show that the culvert inventory rating is highly sensitive to the soil modulus value used for demand calculations, especially for deep-fill culverts (fill heights greater than 6 feet). Unfortunately, geotechnical research studies associated with beams on elastic foundations indicate that soil modulus is difficult to explicitly determine, with modulus values established from different test methods varying by one to two orders of magnitude.

Data from a very limited culvert instrumentation and field test program (three culverts having different types of drained soil backfill) suggest that TxDOT's culverts are typically backfilled with on-site soil excavated during the culvert construction process. Superior backfill material should not be assumed without verification. Among the approaches identified above to determine modulus, data suggest that the falling weight deflectometer (FWD) provides the most reliable and repeatable soil modulus values. The data suggest that the soil modulus values in Table IV-3 are reasonable to define the linear elastic constitutive model used for Level 3 culvert rating analyses.

F. Reinforcing Steel Schedule

Table IV-4 identify the culvert rating variables associated with reinforcing steel quantities. Figure IV-2 shows a typical cross section to further explain these details. It is customary to do a "take-off" and determine the amount of reinforcing steel directly from the construction documents. The reinforcing steel quantities must be determined for each cross section shown in Figure II-2. This will require analyzing the bar schedules on the drawings to express bar size and spacing in terms of area of steel per foot of culvert, normal to the culvert cross section.

TABLE IV-4. STEEL REINFORCING VARIABLES

Variable	***Abbr.***	***Units***
Area of tensile steel per foot (normal to culvert cross section)	A_s	in.^{2} / ft
Area of compression steel per foot (normal to culvert cross section)	A'_s	in.^{2} / ft
Distance from the extreme compression fiber to the centroid of the tension reinforcement	*d*	in.
Distance from the extreme tension fiber to the centroid of the compression reinforcement	*d'*	in.

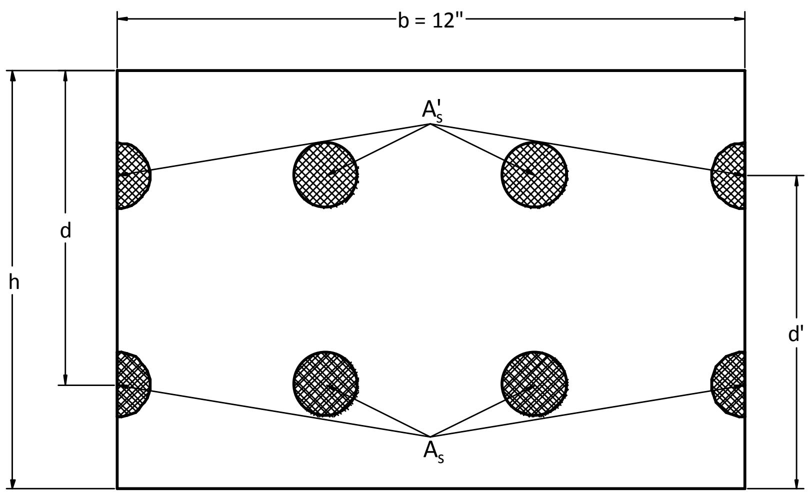

FIGURE IV.2. TOP SLAB CROSS SECTION LABELED FOR POSITIVE BENDING.

It may happen that construction drawings for a culvert are not available. Or, more specific information about the reinforcing steel may be required. In either case, both non-destructive and destructive test methods can be used to establish/verify the actual reinforcing steel schedule.

G. Culvert Installation Method

The typical application represented by this Guide is to load rate culverts that have been in service for many years. For these types of culverts, it is safe to assume that soil stresses associated with culvert installation are dissipated such that construction and installation loads no longer affect the rating. Thus, the culvert rating process for older, *in-service* culverts requires no consideration of the installation method.

For recently-installed culverts (in service for less than, say, five years), the installation method and construction process can significantly impact soil stresses around the culvert and thus affect the culvert load rating. The AASHTO *MCEB* Section 16.6.4.2 (AASHTO 2003) discusses modification of earth loads for soil structure interaction and identifies the following approaches:

- Embankment installation
- Trench installation

Culvert load rating parameters associated with these installation methods should be addressed as per the AASHTO *MCEB*.

V. CULVERT CAPACITY CALCULATIONS

A. POLICY GUIDANCE

The AASHTO *SSHB* provides equations to determine section capacities for each potential failure mode; that is, bending moment, shear, and axial thrust. These section capacities, C, are used in Equation II-1 when calculating the culvert rating factors. *SSHB* Section 8.16.3 discusses flexural capacity calculations considering maximum reinforcing limits in the terms of the balanced reinforcement ratio. The capacity discussion is split between singly and doubly reinforced beams. *SSHB* Section 8.16.4 provides guidelines for determining the thrust capacity. *SSHB* Section 8.16.4.3 in particular provides an equation for checking that the thrust load is small enough to not control over moment. *SSHB* Section 8.16.6.7 provides a complex equation for determining the shear capacity based on both the shear and moment demands. This equation has very simple upper and lower limits that may be used when shear does not control.

B. SIGN CONVENTION FOR LOAD RATING CALCULATIONS

The sign convention that will be used throughout this document is that the layer of the steel on the inside of the culvert is placed in tension during positive bending, while the outside layer of steel is placed in the tension during negative bending. Stated another way, when the tension face is *inside* the culvert, bending is positive. When the tension face is *outside* the culvert, bending is negative.

C. Culvert Capacity Calculations for Critical Sections

This section of the Guide discusses how to apply policy to calculate section capacities as part of the load rating process. The section capacity must be calculated for each failure mode (bending moment, shear, axial thrust) at each critical section of the culvert defined in Figure II-. The gross section properties are used to calculate the capacity of the culvert structure.

1. Bending Moment Capacity

Bending moment capacity must be calculated in each bending direction; that is, both positive and negative. Bending moment capacity may be determined using the following steps which have been derived to follow the AASHTO provisions in the *SSHB*. For reference, the *SSHB* equation number is shown in parenthesis in the equation identifier.

Capacity Step 1. Determine the centroid of the section at ultimate capacity using Equation V-1.

EQUATION V-1. CENTROID OF THE SECTION AT ULTIMATE CAPACITY (DERIVED FOR *SSHB* 8.16.3.4).

$$c = 0.5\left\{-\frac{(87{,}000 - 0.85f_c')A_s' - F_yA_s}{0.85f_c'\beta_1 b} + \sqrt{\left[\frac{(87{,}000 - 0.85f_c')A_s' - F_yA_s}{0.85f_c'\beta_1 b}\right]^2 + 4\left[\frac{87{,}000A_s'd'}{0.85f_c'\beta_1 b}\right]}\right\}$$

where: c = the centroid of the section (in.)
F_y = yield strength of the reinforcement (psi)
f'_c = the compressive strength of concrete (psi)
A_s = area of the tension reinforcement (in.^{2})
A'_s = area of the compression reinforcement (in.^{2})
d' = distance from the extreme compression fiber to the centroid of the compression reinforcement (in.)
b = width of the compression face member (typically 12 inches)
β_1 = 0.85 when $f_c' \leq 4{,}000\text{psi}$
= $1.05 - f_c' * (.0005)$ when $4{,}000\text{psi} \leq f_c' \leq 8{,}000\text{psi}$
= 0.65 when $f_c' \geq 8{,}000\text{psi}$ from *SSHB* 8.16.2.7

Capacity Step 2. Calculate the stress in the compression steel using Equation V-2.

EQUATION V-2. STRESS IN THE COMPRESSION STEEL (PSI) (DERIVED FOR *SSHB* 8.16.3.4).

$$0 \leq \mathrm{F_s'} = 87{,}000\frac{\mathrm{c - d'}}{\mathrm{c}} \leq \mathrm{F_y}$$

where: F_s' = the stress in the compression steel (psi)
F_y = yield strength of the reinforcement (psi)
c = the centroid of the section (in.)
d' = distance from the extreme compression fiber to the centroid of the compression reinforcement (in.)

Capacity Step 3. Calculate the balanced stress in the compression steel using Equation V-3. However, if F_s' equals zero (established from Capacity Step 2) then F_b' equals zero and the load rater may proceed to Capacity Step 4.

EQUATION V-3. STRESS IN COMPRESSION STEEL AT BALANCED STEEL (*SSHB* 8.16.3.4.3 EQ.8-28).

$$F_b' = 87{,}000\left[1 - \frac{d'}{d}\left(\frac{87{,}000 + F_y}{87{,}000}\right)\right] \le F_y$$

where: F_b'= the stress in the compression steel (psi)
F_y = yield strength of the reinforcement (psi)
c = the centroid of the section (in.)
d = distance from the extreme compression fiber to the centroid of the tension reinforcement (in.)
d' = distance from the extreme compression fiber to the centroid of the compression reinforcement (in.)
If $\mathrm{F}_s' = 0$ then $\mathrm{F}_b' = 0$

Capacity Step 4. Calculate the balanced steel ratio using Equation V-4.

EQUATION V-4. RHO BALANCED FOR DOUBLY REINFORCED SLABS (*SSHB* 8.16.3.4.3 EQ8-27).

$$\rho_b = \frac{0.85\beta_1 f_c'}{F_y}\left(\frac{87{,}000}{87{,}000 + F_y}\right) + \frac{A_s' F_b'}{bdF_y}$$

where: ρ_b= the balanced ratio of tensile reinforcement
f'_c= the compressive strength of concrete (psi)
F_y = yield strength of the reinforcement (psi)
F_b'= the stress in the compression steel (psi)
A'_s= area of the compression reinforcement (in.^{2})
d = distance from the extreme compression fiber to the centroid of the tension reinforcement (in.)
b = width of the compression face member (typically 12 inches)
β_1= 0.85 when $f_c' \le 4{,}000\text{psi}$
= $1.05 - f_c' * (.0005)$ when $4{,}000\text{psi} \le f_c' \le 8{,}000\text{psi}$
= 0.65 when $f_c' \ge 8{,}000\text{psi}$ from *SSHB* 8.16.2.7

Capacity Step 5. Check the balanced steel ratio using Equation V-5.

EQUATION V-5. MAXIMUM REINFORCING CHECK (DERIVED FROM *SSHB* 8.16.3.1.1).

$$\rho = \frac{A_s}{bd} \le 0.75\rho_b$$

where: ρ = the ratio of tensile reinforcement
A_s= area of the tension reinforcement (in.^{2})
b = width of the compression face member (typically 12 inches)
d = distance from the extreme compression fiber to the centroid of the tension reinforcement (in.)
ρ_b= the balanced ratio of tensile reinforcement per Equation V-4

Capacity Step 6. Calculate the moment capacity using Equation V-6 or Equation V-7.

EQUATION V-6. GENERALIZED MOMENT CAPACITY (DERIVED FROM *SSHB* 8.16).

$$\varphi M_n = \varphi \left\{ \left(A_s F_y - A_s' F_s'\right) \left[d - \frac{A_s F_y - A_s' F_s'}{2(0.85) f_c' b} \right] + A_s' F_s' (d - d') \right\} \left(\frac{'}{12"}\right) \left(\frac{\text{kip}}{1000\text{lb}}\right)$$

where: φM_n = the bending moment capacity of the section (kip-ft/ft)
φ = 0.9 = the strength reduction factor from *SSHB* 16.6.4.6
f'_c = the compressive strength of concrete (psi)
F_y = yield strength of the reinforcement (psi)
F_s' = the stress in the compression steel (psi)
A_s = area of the tension reinforcement (in.^{2})
A'_s = area of the compression reinforcement (in.^{2})
d = distance from the extreme compression fiber to the centroid of the tension reinforcement (in.)
d' = distance from the extreme compression fiber to the centroid of the compression reinforcement (in.)
b = width of the compression face member (typically 12 inches)

If no tensile steel reinforcing is provided, the moment capacity may be taken as the cracking moment such that any incidental moment in the unreinforced direction does not result in an over-conservative controlling load rating. This should be calculated using Equation V-7.

EQUATION V-7. MINIMUM CRACKING MOMENT CAPACITY.

$$\varphi M_n = \varphi h^2 \sqrt{f_c'} \left(\frac{\text{kip}}{1000\text{lb}}\right)$$

where: φM_n = the bending moment capacity of the section (kip-ft/ft)
φ = 0.9 = the strength reduction factor from *SSHB* 16.6.4.6
f'_c = the compressive strength of concrete (psi)
h = the thickness of the total section (in.)

2. Shear Capacity

Shear capacity must be calculated at each critical section as defined in Figure II-2. Technically these are the moment critical sections, and this is a conservative assumption if moment controls the load rating. In cases where shear appears to control the load rating, Section VI.C of this Guide provides additional information.

Capacity Step 7. Calculate the shear capacity using Equation V-8 or if needed Equation V-9.

Because shear usually does not control the load rating, AASHTO allows a simple, conservative calculation for the shear capacity using the minimum shear capacity shown in Equation V-8. If the culvert load rating is not controlled by shear, the simpler calculation is adequate.

EQUATION V-8. MINIMUM SHEAR CAPACITY (DERIVED FROM *SSHB* 8.16.6.7).

$\varphi V_n = \varphi 3bd\sqrt{f_c'}$ for single-span slabs cast monolithic with the culvert walls (typical for TxDOT designs).
$\varphi V_n = \varphi 2.5bd\sqrt{f_c'}$ for single-span slabs simply supported.

where: φV_n=the shear capacity of the section (lb)
φ = 0.85 from *SSHB* 16.6.4.6
f'_c= the compressive strength of concrete (psi)
d = the depth from compression face to tensile reinforcement in the direction of M_u (in.)
b = width of the compression face member (typically 12 inches)

If it turns out that the culvert load rating is controlled by shear, Equation V-9 can be used to determine shear capacity in the critical section. Equation V-9 will yield a more accurate, less conservative value for shear capacity but requires knowledge of the shear and moment demands at each section and is therefore tedious and time-consuming to apply.

EQUATION V-9. SHEAR CAPACITY EQUATION (*SSHB* 8.16.6.7.1 EQ.8-59).

$$\varphi V_n = \varphi\left(2.14\sqrt{f_c'} + 4{,}600\rho\frac{V_u d}{M_u}\right)bd \le 4bd\sqrt{f_c'}$$

where: φV_n=the shear capacity of the section (lb)
$\ge 3bd\sqrt{f_c'}$ for single-span slabs cast monolithic with the culvert walls
$\ge 2.5bd\sqrt{f_c'}$ for single-span slabs simply supported
φ = 0.85 from *SSHB* 16.6.4.6
f'_c= the compressive strength of concrete (psi)
ρ = the tensile steel ratio in the direction of M_u
d = the depth from compression face to tensile reinforcement in the direction of M_u (in.)
b = width of the compression face member (typically 12 inches)
V_u= the shear demand or load (kip)
M_u= the moment demand or load (kli)
$\frac{V_u d}{M_u} < 1.0$

3. Thrust Capacity

Axial thrust capacity must be calculated at each critical section as defined in Figure II-2. Technically these are the *moment* critical sections, but it is standard practice to also calculate axial thrust at these locations.

Capacity Step 8. Calculate the maximum thrust capacity using Equation V-10. There is only one thrust capacity per critical section. This is a compressive (-) capacity.

EQUATION V-10. THRUST CAPACITY (*SSHB* 8.16.4.2.1 EQ.8-31).

$$\varphi P_n = -\varphi\left[0.85 f_c'\left(A_g - A_s - A_s'\right) + (A_s + A_s')F_y\right]$$

where: φP_n= the thrust capacity of the section (lb)
f'_c= the compressive strength of concrete (psi)
F_y= yield strength of the reinforcement (psi)
A_g= the gross area of the section (in.^{2})
A_s= area of the tension reinforcement (in.^{2})
A'_s= area of the compression reinforcement (in.^{2})

Usually the thrust demand is much smaller than the thrust capacity. In fact, the thrust demand is typically less than the incidental axial load assumed in the AASHTO *SSHB* for beam calculations. The capacity specifications in this Guide assume this to be the case.

A "thrust check" for each critical section is provided in the AASHTO *SSHB* and is described in Section VI.B of this Guide (next chapter). If the thrust check is satisfied then the thrust demand is less than the assumed incidental axial load and the culvert slab slices may be accurately modeled for both capacity and demand as beam elements. This is the normal situation.

However, if the thrust check is not satisfied, the slab slices are no longer considered beams for analysis purposes, but instead must be modeled as beam-columns. If this is the situation, combined bending equations must be used from AASHTO *SSHB* Section 8.16.4.3.

VI. Analytical Modeling for Demand Loads

A. Overview

Analytical modeling is used to determine the dead load and live load demand on the structure. This Culvert Rating Guide describes a hierarchical approach to calculate the demand loads. The lowest tier, Level 1, uses a two-dimensional, structural frame model with AASHTO loadings, balancing bottom slab loads, and simply-supported boundary conditions. The next tier, Level 2, also uses a two-dimensional, structural frame model with AASHTO loading, but uses continuous spring supports for the bottom slab instead of balancing bottom slab loads. The Level 3 analysis uses a two-dimensional, finite element analysis model of the soil-structure system to determine demands. By modeling soil conditions, Level 3 considers soil-structure interaction effects such as soil arching. These are the main approaches discussed in this Guide.

As noted, this Guide also discusses the general case for culvert modeling, which is a Level 4 analysis. Level 4 is the most sophisticated of the modeling approaches and uses a two or three-dimensional finite element model of the soil-structure system. Level 4 modeling would typically be used for research or other specialized applications, and is discussed in Chapter VII of this Guide.

B. Generalized Step-by-Step Procedure for Determining Demand Loads

The flow chart in Figure III-2 provides an overview of the culvert load rating process. This chapter focuses on calculation of demand loads using analytical (computer) modeling. Regardless of the level of analysis, the following step-by-step procedure applies:

Demand Step 1. Obtain load rating parameters necessary to define each aspect of the computer model: dimensional data, strength properties for steel and concrete, soil properties, and loads.

Demand Step 2. Create the analytical model by laying out the nodes and members and identifying the critical sections for the culvert.

Demand Step 3. Apply appropriate boundary conditions.

Demand Step 4. Calculate the magnitude of dead and live loads for both vertical and lateral stress distributions.

Demand Step 5. Apply the dead and live load stress distributions to the culvert model.

Demand Step 6. Define load cases for the model. Briefly stated, this consists of one set of load cases designed to induce maximum moment at the culvert haunches, and a second set of load cases designed to induce maximum moment at culvert mid-spans.

Demand Step 7. Perform demand calculations for each load case. That is, perform separate computer runs as necessary to define demand moments, shears and axial thrusts at each critical section as defined for each load case. Four computer runs, minimum, are typically required.

Demand Step 8. After determining the demands, use Equation VI-1 to check that actual thrust demand is lower than the incidental axial load assumed in the moment capacity equations.

EQUATION VI-1. THRUST CONTROL LIMIT (*SSHB* 8.16.4.3 EQ.8-37).

$$P_u \geq 0.1 f_c' A_g$$

where: P_u= the thrust demand (lb)
f'_c= the compressive strength of concrete (psi)
A_g= the gross area of the section (in.^{2})

It is assumed throughout this Guide that Equation VI-1 will always be satisfied. However, if this check is not met, the capacities must be recalculated using beam-column theory as described in AASHTO *SSHB* section 8.16.4.3.

Demand Step 9. This step moves beyond calculation of the demand loads. Once the demand moments, shears and thrusts are established for each critical section in the culvert, for each load case, these must be combined with the corresponding capacity values to determine the rating factor for both inventory and operating conditions per Equation II-1.

Demand Step 10. The controlling rating factor for each critical section is determined by selecting the minimum rating factor, for both inventory and operating conditions, based on the maximum and minimum values for each type of load (moment, shear and thrust) for each load case.

Demand Step 11. If shear controls the inventory and operating ratings, the load rater should perform a less-conservative analysis of the shear failure mode based on shear critical sections as per the provisions in Section C of this chapter.

Demand Step 12. The controlling rating factors for the culvert are the minimum rating factors for both inventory and operating conditions.

The following sections of this chapter provide details of this step-by-step procedure, as it applies to each level of analysis.

C. Shear Failure Mode Analysis

Discussions about culvert load rating commonly acknowledge that in most cases, the mode of failure that controls the load rating is moment. The one exception is deep fill culverts which tend to fail in shear. Results from a parametric analysis of a representative sample of TxDOT's reinforced concrete box culvert designs support these points.

Because culvert load ratings are usually controlled by moment, it makes sense to perform initial load rating analyses for all failure modes (moment, axial thrust and shear) based on moment critical sections. These analyses will be technically accurate both for moment and axial demands, and conservative for shear.

The reason for this conservatism is that the shear critical section for culvert corners is actually located at a distance *d* away from the wall face consistent with AASHTO *SSHB* 8.8.2 and 8.16.6.1.2 (see Figure VI-1.A), rather than located at the wall face as is done for moment (see Figure II-2.A). This distinction only applies to culverts without haunches, which is the most common case for TxDOT. For culverts with haunches, the corner critical sections for shear are a distance *d* from the middle of the haunch (Figure VI-1.B). Mid-span critical section locations for moment and shear are the same.

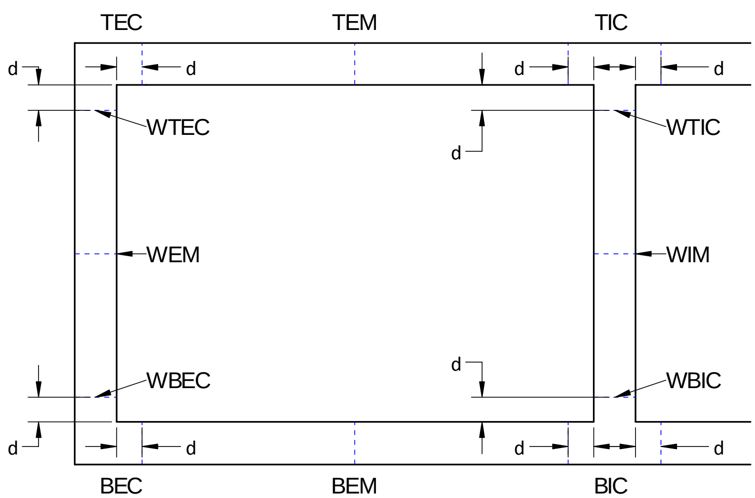

FIGURE VI.1.A SHEAR CRITICAL SECTIONS FOR CULVERTS WITHOUT HAUNCHES.

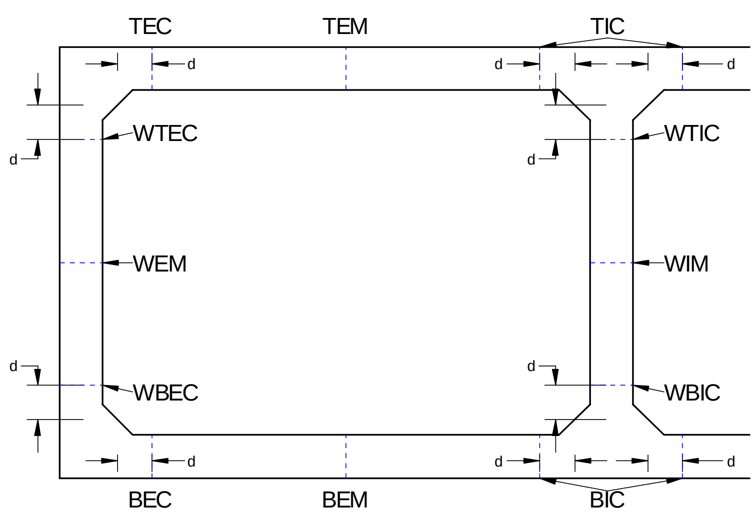

FIGURE VI-1.B SHEAR CRITICAL SECTIONS FOR CULVERTS WITH HAUNCHES.

Only in cases where shear ends up controlling the load rating would it be necessary to reanalyze shear demands based on the *shear* critical sections of Figure VI-1. The following steps capture this procedure.

Shear Provision 1. Assume that the controlling failure mode for the load rating is *moment*. Use the moment critical sections (see Figure II-2) as discussed in this Guide. Perform load rating analyses and check all failure modes (moment, axial thrust and shear).

Shear Provision 2. If *moment* controls the load rating, there is no need to further refine the shear demand or capacity analyses.

Shear Provision 3. If *shear* controls the load rating, and the inventory rating is greater than or equal to HS-20 (inventory rating factor (IRF) ≥1), the culvert will not require load posting and there is no need to further refine the shear demand or capacity analyses.

Shear Provision 4. If *shear* controls the load rating, and the inventory rating is less than HS-20 (IRF<1), redo the shear analyses based on the shear critical sections as defined in AASHTO *SSHB* 8.8.2 and 8.16.6.1.2, with the critical section located at a distance *d* away from the point of support (see Figure VI-1). Moment and axial demands are unchanged. Select the new lowest rating factors from all failure modes.

Shear Provision 5. If, based on the revised shear analysis, *shear* continues to control the load rating and the inventory rating is still less than HS-20 (IRF<1), use the demand-dependant shear capacity equation (Equation V-9) to generate capacity values used to calculate rating factors. This approach does not include any of the conservative shear assumptions. Select the new lowest rating factors from all failure modes.

D. Level 1 Analysis: Two-Dimensional, Simply-Supported Structural Frame Model

This level of analysis uses a relatively simple two-dimensional frame analysis model and AASHTO loading parameters. It is designed to provide a quick, conservative, repeatable load rating.

1. Assumptions

The following assumptions are made in the two-dimensional structural frame analysis stage:

- AASHTO loads are applied.
- Gross section properties control structure behavior at ultimate strength.
- Culvert corners are considered rigid.
- Supporting soil pressures are uniform over the length of the bottom slab.
- All assumptions inherently involved in two-dimensional, frame analysis.
    - Reinforced concrete behaves elastically with stress related linearly to strain.
    - Reinforced concrete behaves identically regardless of direction of the applied load.
    - All deformations are small.
    - Beams are long relative to their depth.
    - Plane sections remain plane.
- A one foot (b = 12 in.) section of the culvert may be analyzed as a frame.
- No hydrostatic pressure (water) exists inside the culvert.
- Supporting soils are fully drained, i.e. no hydrostatic pressure outside the culvert.
- Moments resulting in tension on the inside face of the culvert are positive.
- Moments resulting in tension on the outside face of the culvert are negative.

Though reinforced concrete does not generally satisfy the first two, two-dimensional, frame analysis assumptions – namely, elasticity and homogeneity – this model will predict approximate and conservative moment, shear and thrust demands.

2. Model Dimensions

The Level 1 model should be developed so that beam nodes are at the centerline of the slab sections they are modeling. Each section should use the gross area properties of a one-foot wide strip of culvert.

A node should be placed at each critical section so that the resultant forces and moments will be calculated automatically at those points. The location of the corner critical sections can be determined directly as illustrated in Figure VI-2. As noted earlier, AASHTO specifies that the mid-span critical sections must be determined by locating the maximum combined (dead and live load) moment in the mid-span region. However, for the purposes of this Guide, the mid-span critical section is always assumed to be located at mid-span.

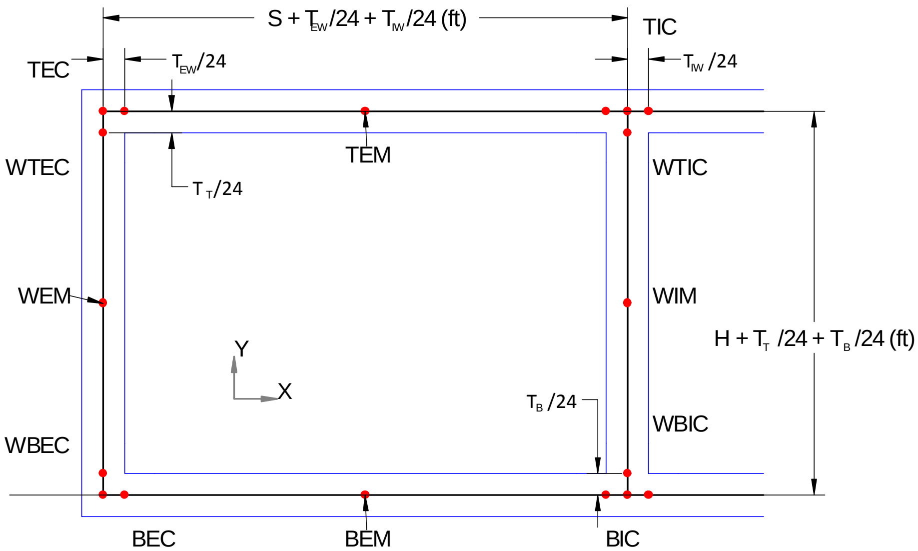

FIGURE VI.2.A MODEL DIMENSIONS FOR A LEVEL 1 ANALYSIS FOR CULVERTS WITHOUT HAUNCHES.

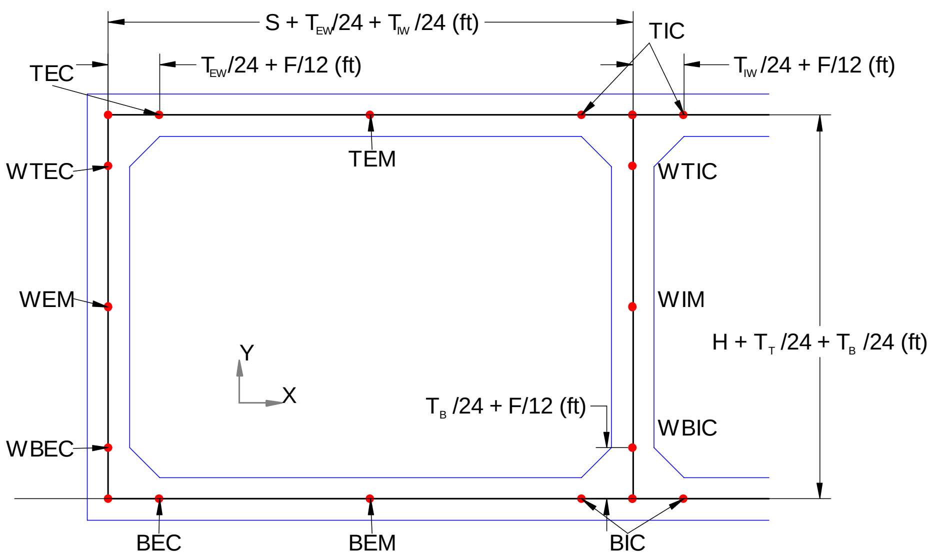

FIGURE VI-2.B MODEL DIMENSIONS FOR A LEVEL 1 ANALYSIS FOR CULVERTS WITH HAUNCHES.

3. Boundary Conditions

In the Level 1 model, the primary function of the boundary conditions is to maintain global stability. Reactions are of no concern. The model should be simply supported, with a pin at the bottom left corner (restrain in global X and Y directions) and rollers at other bottom wall centerlines (restrain in global Y direction only). See Figure VI-3.

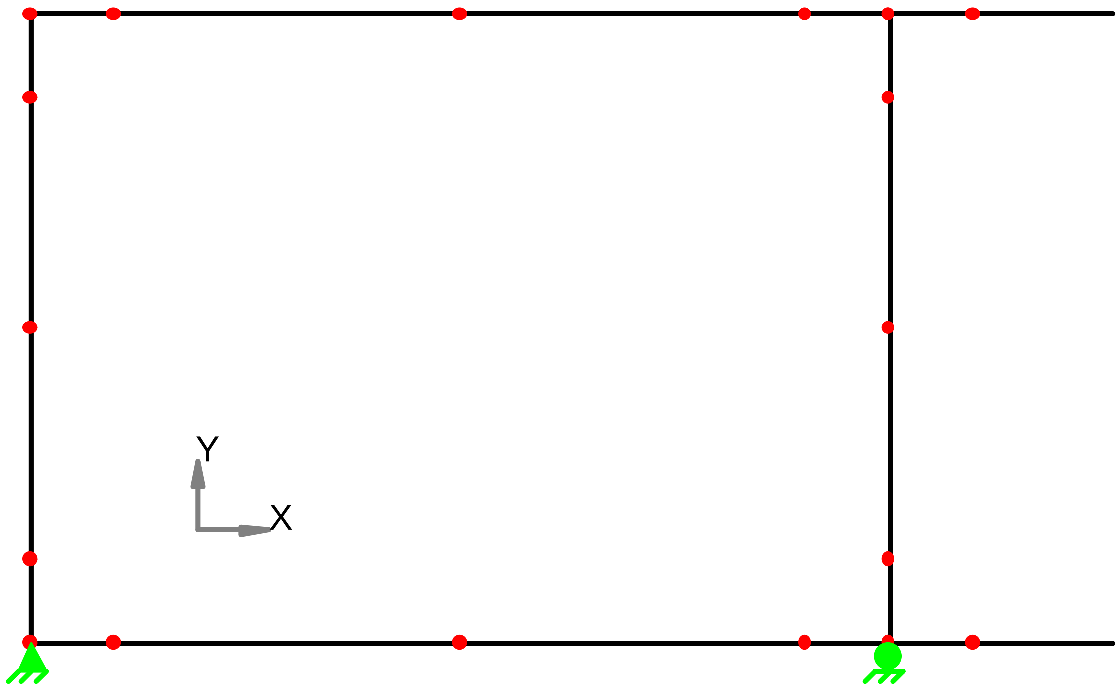

FIGURE VI.3. BOUNDARY CONDITIONS FOR TWO DIMENSIONAL, SIMPLY-SUPPORTED STRUCTURAL FRAME MODEL.

4. Loads

The loads placed on the structure for Level 1 modeling correspond directly to the provisions of the AASHTO policy. Figure VI-4 shows the load location and direction conventions. The loads are as follows:

- DL_V … Vertical Dead Load
- DL_{hT} …Horizontal Dead Load, top of culvert
- DL_{hB} … Horizontal Dead Load, bottom of culvert
- LL_{VT} … Vertical Live Load, top slab
- LL_{VB} … Vertical Live Load, bottom slab
- LL_h … Horizontal Live Load
- SW… Self Weight of the culvert

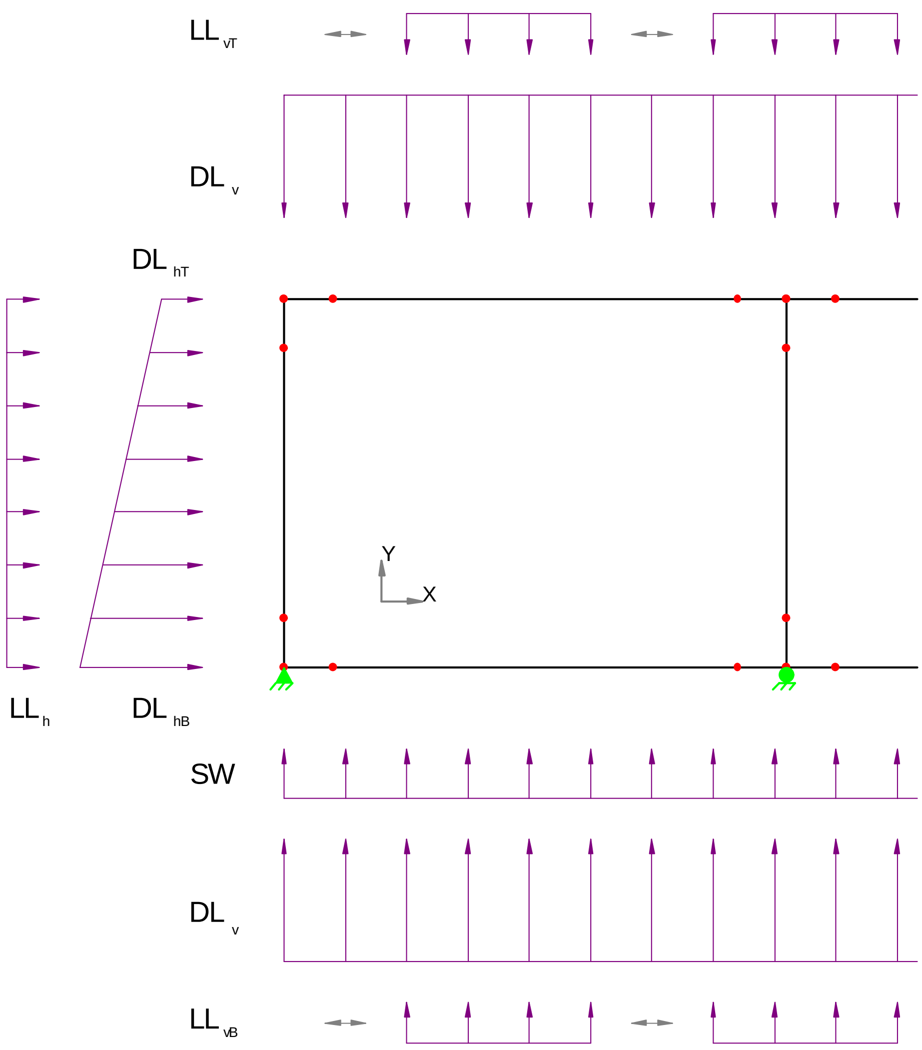

FIGURE VI.4. LOAD CONVENTIONS FOR TWO DIMENSIONAL, SIMPLY-SUPPORTED STRUCTURAL FRAME MODEL.

A unique aspect of the Level 1 model is that in order to account for upward soil pressure support, whatever load is placed downward on the structure should also be placed upward on the bottom slab, uniformly. The result is balanced vertical loading and no reactions in the supports. The boundary conditions only keep the model stable. They should not contribute significantly to the support of the structure.

c. Dead Load

Per Equation VI-2, the first load represents the weight of the soil on top of the structure. According to AASHTO *SSHB* 6.2.1.B, the unit weight of soil is 120 pcf. This load must be placed downward on the top slab, and balanced by placing it upward on the bottom slab.

EQUATION VI-2. VERTICAL DEAD LOAD, DL_V (KSF)

$$DL_v = cover\ soil = (0.120\ kcf)\ D$$

where: DL_v = the vertical dead load (ksf)
D = cover soil depth (ft)

Per Equation VI-3, the second load represents the self-weight of the structure. If the chosen analysis tool has a gravity feature, this should be used to accurately distribute the self-weight across the structure. Otherwise, the weight of the slabs and walls should be applied manually in the downward direction, expressed in terms of a uniformly distributed load. Whether the self-weight is applied automatically or manually, the total self-weight of the culvert should also be applied upward across the bottom slab, expressed in terms of a uniformly distributed load.

EQUATION VI-3. SELF-WEIGHT OF THE CULVERT, *SW* (KSF)

$$SW = \left\{(T_T + T_B) * \left[S + \frac{2 * T_{EW}}{12} + (N-1)\frac{T_{IW}}{12}\right] + 2 * T_{EW} * H + (N-1) * T_{IW} * H\right\}\frac{0.150\ kcf}{S * 12}$$

where: *SW* = the vertical dead load (ksf)
T_T = top slab thickness (in.)
T_B = bottom slab thickness (in.)
T_{IW} = interior wall thickness (in.)
T_{EW} = exterior wall thickness (in.)
S = the clear span of a single box (ft)
H = the clear height of a single box (ft)
N = the number of box spans

The third load is the horizontal dead load. This dead load is a trapezoidal load placed on the outside walls of the culvert facing inward. The load is determined using the equivalent fluid weight of soil listed in AASHTO *SSHB* 6.2.1.B and 3.20.2. Equation VI-4 and Equation VI-5 define the horizontal load at the top and bottom of the slab. In these equations, the D and H values are in feet, and the T values are in inches. Intermediate points may be determined by linear interpolation as necessary.

EQUATION VI-4. HORIZONTAL DEAD LOAD AT THE TOP APPLIED TO THE EXTERIOR WALLS OF THE CULVERT, DL_{HT} (KSF)

$$DL_{hT} = full\ lateral\ pressure = (0.060\ kcf)\ (D + \frac{T_T}{24})$$

where: DL_{hT} = the horizontal dead load at the top of the exterior walls (ksf)
T_T = top slab thickness (in.)
D = cover soil depth (ft)

EQUATION VI-5. HORIZONTAL DEAD LOAD AT THE BOTTOM APPLIED TO THE EXTERIOR WALLS OF THE CULVERT, DL_{HB} (KSF)

$$DL_{hB} = full\ lateral\ pressure = (0.060\ kcf)\ \left(D + H + \frac{T_T}{12} + \frac{T_B}{24}\right)$$

where: DL_{hB} = the horizontal dead load at the bottom of the exterior walls (ksf)
T_T = top slab thickness (in.)
T_B = bottom slab thickness (in.)
H = the clear height of a single box (ft)
D = cover soil depth (ft)

d. Live Load

The live load on the structure as required by AASHTO *SSHB* 3.7.6 is an HS-20 truck. There are three live loads due to the HS-20 truck: (1) the horizontal live load, LL_h; (2) the vertical live load applied to the top slab, LL_{vT}; and (3) the vertical live load applied to the bottom slab, LL_{vB}. The impact factor, *IM*, and all other variables used in the live load equations are defined in Table III-1.

Per Equation VI-6, the first live load is the horizontal live load, LL_h *(ksf)*. This load is constant regardless of the number of trucks passing over the culvert. AASHTO *SSHB* 3.20.3 provides a 2 ft surcharge allowance for trucks which are approaching, but not directly above, the culvert.

EQUATION VI-6. HORIZONTAL LIVE LOAD APPLIED TO THE EXTERIOR WALLS, LL_H (KSF)

$$LL_h = 2' * (.060\ kcf) = .120\ ksf$$

where: LL_h = the horizontal live load on the exterior walls (ksf)

The vertical live load applied to the top of the culvert, LL_{vT} *(ksf),* is the second type of live load. The magnitude of the vertical live load depends on the depth of fill, the wheel load, the culvert span, the impact factor, the number of lanes, and the number of trucks. For this Guide, the vertical live load has been expressed in terms of 15 distinct equations derived from AASHTO SSHB 3.7.6, 3.12.1, 3.24.3.2 and 6.4, including the lane reduction factor described in AASHTO SSHB 3.12.1. These 15 equations are collectively designated as Equation VI-7.

For a given culvert, the load rater must select one of the 15 equations to determine the magnitude of the vertical live load. Two variables govern selection of the appropriate live load equation. The first is the number of lanes passing over the culvert. Section 3.6 of the SSHB provides guidance for determining traffic lanes. Generally, the number of lanes is determined by the number of whole, 12-foot-wide lanes that will fit across the roadway. Roadways between 20' and 24' will have two lanes. The second variable is the depth of fill, D. This fill depth will yield the proper load configuration as per the AASHTO stress distribution. Taken together, the number of lanes and the fill depth establish the controlling number of trucks and identify the proper equation to use for LLvT.

Once the magnitude of the live load has been established, it is necessary to define the area over which the live load acts. The vertical live load should be applied as a moving load across the top of the culvert structure. This will have the effect of creating a moment envelope, with both maximum and minimum values. The length over which the pressure should be applied, the center-to-center spacing for the distributed loads, and the wheel load, P, used to calculate each load are illustrated in Figure VI-5, Figure VI-6 and Figure VI-7 for different cover depths.

The final live load is the vertical live load applied upward to the bottom slab, LLvB (ksf). This live load is derived from AASHTO SSHB 16.6.4.3. For this Guide, the magnitude of the upward live load has been expressed in terms of 15 distinct equations. These 15 equations are collectively designated as Equation VI-8. The load is placed upward on the bottom slab to balance the vertical live load on the top slab as illustrated in Figure VI-5, Figure VI-6 and Figure VI-7. Again, the load rater must select one of the 15 equations. The selected equation should correspond to the culvert's fill height and number of lanes.

EQUATION VI-7. VERTICAL LIVE LOAD APPLIED TO THE TOP SLAB, LL_{vT} (KSF).

No of Traffic Lanes	*Depth of Fill, D (ft)*	*Magnitude, LL_{vT} (ksf)*	*Controlling No. of Trucks*
1	0 < D < 2’	$LL_{vT} = \frac{(1+IM)*P}{4+0.06*S}$	1 truck
1	2’ < D < 3.4’	$LL_{vT} = \frac{(1+IM)*P}{(1.75*D)^2}$	1 truck
1	3.4’ < D < 8’	$LL_{vT} = \frac{2*P}{1.75*D*(1.75*D+6')}$	1 truck
1	8’ < D	$LL_{vT} = \frac{4.5*P}{(1.75*D+28')*(1.75*D+6')}$	1 truck
2	0 < D < 2’	$LL_{vT} = \frac{(1+\mathrm{IM})*\mathrm{P}}{4+0.06*\mathrm{S}}$	1 truck
2	2’ < D < 2.3’	$LL_{vT} = \frac{(1+\mathrm{IM})*\mathrm{P}}{(1.75*\mathrm{D})^2}$	1 truck
2	2.3’ < D < 3.4’	$LL_{vT} = \frac{(1+\mathrm{IM})*2*\mathrm{P}}{1.75*\mathrm{D}*(1.75*\mathrm{D}+4')}$	2 trucks
2	3.4’ < D < 8’	$LL_{vT} = \frac{4*\mathrm{P}}{1.75*\mathrm{D}*(1.75*\mathrm{D}+16')}$	2 trucks
2	8’ < D	$LL_{vT} = \frac{9*\mathrm{P}}{(1.75*\mathrm{D}+28')*(1.75*\mathrm{D}+16')}$	2 trucks
3+	0 < D < 2’	$LL_{vT} = \frac{(1+IM)*P}{4+0.06*S}$	1 truck
3+	2’ < D < 2.3’	$LL_{vT} = \frac{(1+IM)*P}{(1.75*D)^2}$	1 truck
3+	2.3’ < D < 3.4’	$LL_{vT} = \frac{(1+IM)*2*P}{1.75*D*(1.75*D+4')}$	2 trucks
3+	3.4’ < D < 7.2’	$LL_{vT} = \frac{4*\mathrm{P}}{1.75*\mathrm{D}*(1.75*\mathrm{D}+16')}$	2 trucks
3+	7.2’ < D < 8’	$LL_{vT} = \frac{6*.9*P}{1.75*D*(1.75*D+26')}$	3 trucks
3+	8’ < D	$LL_{vT} = \frac{13.5*.9*P}{(1.75*D+28')*(1.75*D+26')}$	3 trucks

where: $\boldsymbol{LL_{vT}}$ = the vertical live load on the top slab (ksf)
IM = the impact factor from Table II-1
S = the clear span of a single box (ft)
P = either 4 or 16 kips as indicated in Figure VI-5 through Figure VI-7
D = cover soil depth (ft)

EQUATION VI-8. VERTICAL LIVE LOAD APPLIED TO THE BOTTOM SLAB, LL_{vB} (KSF)

No. of Traffic Lanes	*Depth of Fill, D (ft)*	*Magnitude, LL_{vB} (ksf)*	*Controlling No. of Trucks*
1	0 < D < 2’	$LL_{vB} = \frac{(1+IM)*P}{2*H*(4+0.06*S)}$	1 truck
1	2’ < D < 3.4’	$LL_{vB} = \frac{(1+IM)*P}{1.75*D*(1.75*D+2*H)}$	1 truck
1	3.4’ < D < 8’	$LL_{vB} = \frac{2*P}{(1.75*D)*(1.75*D+6'+2*H)}$	1 truck
1	8’ < D	$LL_{vB} = \frac{4.5*P}{(1.75*D+28')*(1.75*D+6'+2*H)}$	1 truck
2	0 < D < 2’	$LL_{vB} = \frac{(1+IM)*P}{2*H*(4+0.06*S)}$	1 truck
2	2’ < D < 2.3’	$LL_{vB} = \frac{(1+IM)*P}{1.75*D*(1.75*D+2*H)}$	1 truck
2	2.3’ < D < 3.4’	$LL_{vB} = \frac{(1+IM)*2*P}{(1.75*D)*(1.75*D+4'+2*H)}$	2 trucks
2	3.4’ < D < 8’	$LL_{vB} = \frac{4*P}{(1.75*D)*(1.75*D+16'+2*H)}$	2 trucks
2	8’ < D	$LL_{vB} = \frac{9*P}{(1.75*D+28')*(1.75*D+16'+2*H)}$	2 trucks
3+	0 < D < 2’	$LL_{vB} = \frac{(1+IM)*P}{2*H*(4+0.06*S)}$	1 truck
3+	2.1’ < D < 2.3’	$LL_{vB} = \frac{(1+IM)*P}{1.75*D*(1.75*D+2*H)}$	1 truck
3+	2.3’ < D < 3.4’	$LL_{vB} = \frac{(1+IM)*2*P}{(1.75*D)*(1.75*D+4'+2*H)}$	2 trucks
3+	3.4’ < D < 7.2’	$LL_{vB} = \frac{4*P}{(1.75*D)*(1.75*D+16'+2*H)}$	2 trucks
3+	7.2’ < D < 8’	$LL_{vB} = \frac{6*.9*P}{(1.75*D)*(1.75*D+26'+2*H)}$	3 trucks
3+	8’ < D	$LL_{vB} = \frac{13.5*.9*P}{(1.75*D+28')*(1.75*D+26'+2*H)}$	3 trucks

where: $\boldsymbol{LL_{vB}}$ = the vertical live load on the bottom slab (ksf)
IM = the impact factor from Table II-1
S = the clear span of a single box (ft)
H = the clear height of a single box (ft)
P = either 4 or 16 kips as indicated in Figure VI-5 through Figure VI-7
D = cover soil depth (ft)

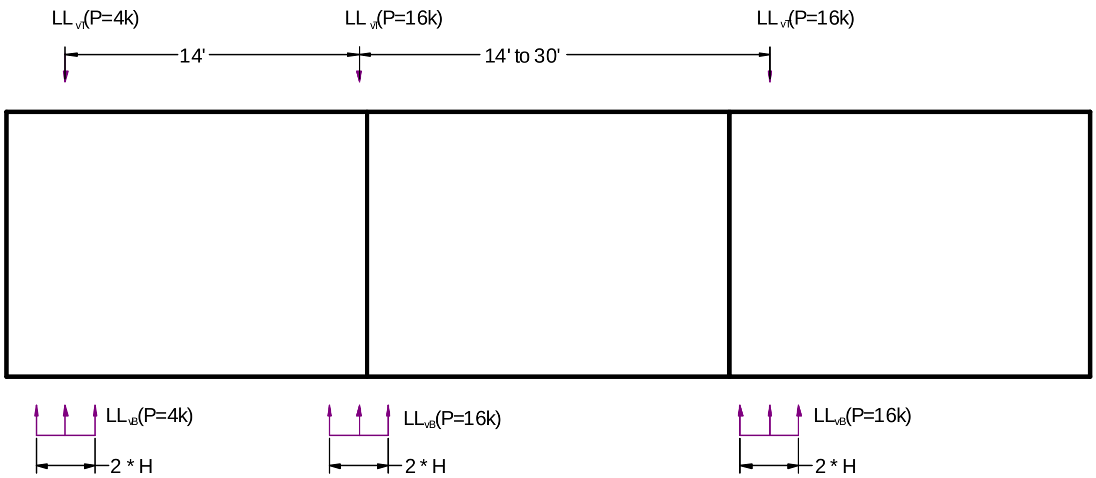

FIGURE VI.5. LIVE LOAD DISTRIBUTION FOR *D* < 2' FOR TWO DIMENSIONAL, SIMPLY-SUPPORTED STRUCTURAL FRAME ANALYSIS.

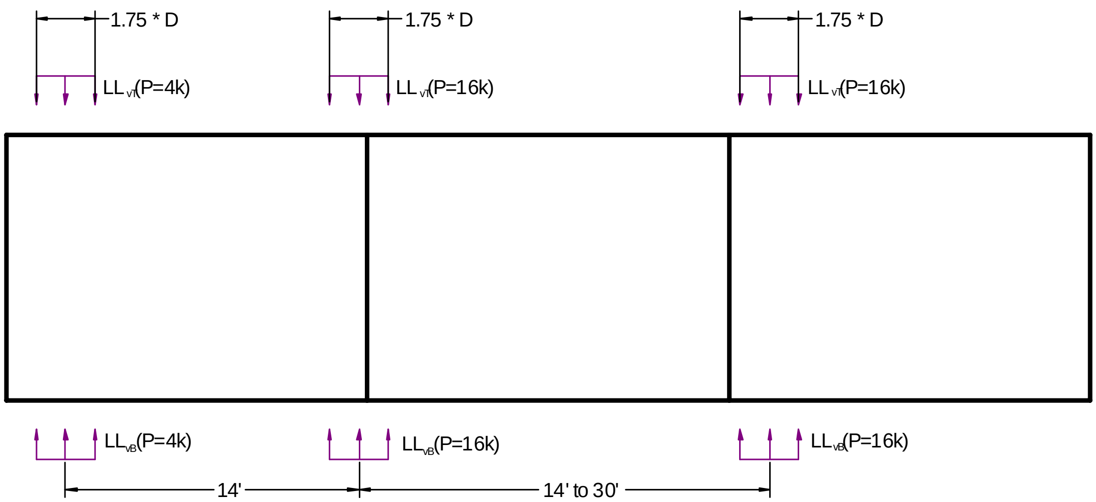

FIGURE VI.6. LIVE LOAD DISTRIBUTION FOR 2' < *D* < 8' FOR TWO DIMENSIONAL, SIMPLY-SUPPORTED STRUCTURAL FRAME ANALYSIS .

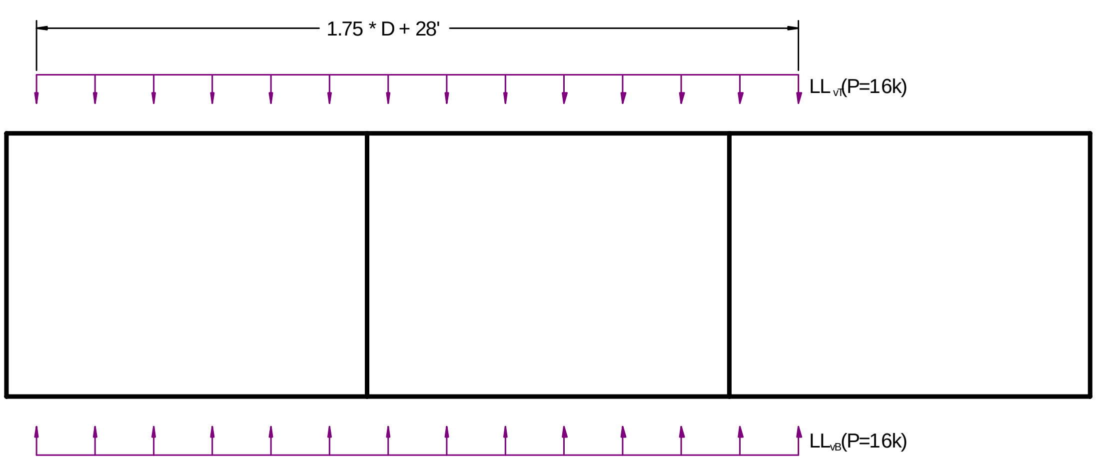

FIGURE VI.7. LIVE LOAD DISTRIBUTION FOR *D* > 8' FOR TWO DIMENSIONAL, SIMPLY-SUPPORTED STRUCTURAL FRAME ANALYSIS.

5. Load Cases

In accordance with ASSHTO *SSHB* 3.20.2, two demand analyses must be made to determine the worst case loading conditions on the culvert structure. The two analyses combine the basic loadings differently to produce conservative demand moments, shears and thrusts.

a. Total Load Case

The first analysis is the "total load case." The total load case is designed to create the maximum demand in most of the culvert structure; that is, critical sections TEC, TIC, BIC, BEC, WIM, WEM, WTIC, WBIC, WBEC, WTEC as per Figure II-. The total load case is designed to yield the maximum shear and axial demands in the whole culvert and the maximum moment demands in all but the top and bottom mid-spans (TEM, TIM, BEM and BIM). This load case applies the above-described loads with a load factor of 1.

b. Reduced Lateral Load Case

The second analysis is the "reduced lateral load case" as described in AASHTO *SSHB* 6.2.1.B, 3.20.2. The reduced lateral load case is designed to create the maximum demand moment for the positive moment sections (TEM, TIM, BEM, BIM). See Figure II-. This load case is intended to create a worst case scenario for the slab moments by reducing the amount of reverse curvature created by lateral pressure on the culvert walls. The reduced lateral load case uses the following load factors:

- Vertical dead load applied to top slab is the same as Equation VI-2. Load factor of 1.
- The self-weight of the culvert is the same as Equation VI-3. Load factor of 1.
- Horizontal dead load at the top applied to the exterior walls of the culvert is one-half the value calculated by Equation VI-4. Load factor of 0.5.
- Horizontal dead load at the bottom applied to the exterior walls of the culvert is one-half the value calculated by Equation VI-5. Load factor of 0.5.
- No horizontal live load is applied to the exterior walls of the culvert. Load factor of 0.
- Vertical live load applied to the top slab is the same as Equation VI-7. Load factor of 1.
- Vertical live load applied upward to the bottom slab is the same as Equation VI-8. Load factor of 1.

c. Interpretation of Load Case Data

As a general rule, it is necessary to perform multiple computer runs (at least two) to calculate moment, shear and thrust demands for each load case. Once the data are obtained, it is customary practice to interpret the data holistically. That is, the load rater should evaluate the maximum and minimum moments, shears and thrusts for each critical section, treating the two load cases independently.

Stated another way, even though the two load cases are designed with the intent to achieve maximum demands at either the corners or the mid-spans, the load rater should not make an *a priori* decision to only evaluate the data this way. This means that the load rater should look at more than just the corner critical sections for the total load case results, and more than just the mid-span critical sections for the reduced lateral load case results. Instead, the load rater should evaluate moment, shear and thrust demands for all of the critical sections for each load case.

6. Demand Load Calculations

Having created the analytical model, defined the boundary conditions, determined the magnitude and extent of loads, and specified the load cases, the next step is to calculate the moment, shear and thrust demands. This requires application of an appropriate structural analysis software package, as discussed in the following section of the Guide.

7. Analytical Program – CULV-5

a. Overview

The two-dimensional, simply-supported frame model (Level 1) can be analyzed using several commercially-available structural analysis software programs. Examples include RISA-2D, BRASS, BOXCAR, CULV-5 (TxDOT's program) or older frame analysis programs.

At TxDOT, the program most adept for Level 1 calculations is CULV-5. Therefore, specific guidance will be provided for this tool. If a user is more comfortable with another frame analysis program, the designer is free to use it.

CULV-5 is an MS-DOS program developed and distributed by the Texas Department of Transportation. The heart of the program is a two-dimensional frame analysis. Documentation supporting CULV-5 includes the Version 1.71 Readme file (TxDOT, 2004), Input Guide (TxDOT, 2003), and CULV5 – Concrete Box Analysis Program (TxDOT, 2003). The load rater who intends to use CULV-5 should become familiar with this documentation to better understand the input, analysis approach, and program output.

b. CULV-5 Strengths and Limitations

The CULV-5 program has some notable strengths that make it the ideal first choice for a Level 1 culvert load rating program. These are:

- Quick and conservative
- Program inputs are very simple
- Appropriate live and dead loads are automatically calculated and applied
- Influence lines are used to determine maximum moments, shears and thrust
- A more conservative bottom slab live load is used
- The sign convention used is the same as the sign convention outlined in Section V.B

Notwithstanding its many strengths, the CULV-5 program also has some notable limitations that must be recognized and addressed:

- Demand at the critical corner sections is not automatically calculated.
- The use of influence lines to calculate live load moments results in an overly conservative live load applied to the bottom of the structure.
- Only culverts with 4 or fewer barrels may be analyzed directly. Culverts with more than 4 barrels may be approximated using a 4 barrel model at the expense of slightly more conservative results.

The limitations may be overcome. Determining the critical section demand requires linear interpolation between the 10th point demands which the program does produce. If the culvert fails to rate, not much time has been spent and the user may move on to the higher-level models.

c. CULV-5 Step-by-Step Instructions

Culvert load rating for Level 1 using the CULV-5 program can be accomplished by following these steps. This sequence assumes the load-rater has already defined the input parameters and is prepared to create the Culv-5 input file.

CULV5 Step 1. Using data obtained for the culvert as discussed in Chapter IV of this Guide, write the CULV-5 input file in a basic text editor (eg, Notepad) according to the form in Figure VI-8. Alternatively, the load rater may use the "Culv5 Input" program developed by TechMRT and hosted on the TxDOT Bridge Division website to create the input file.

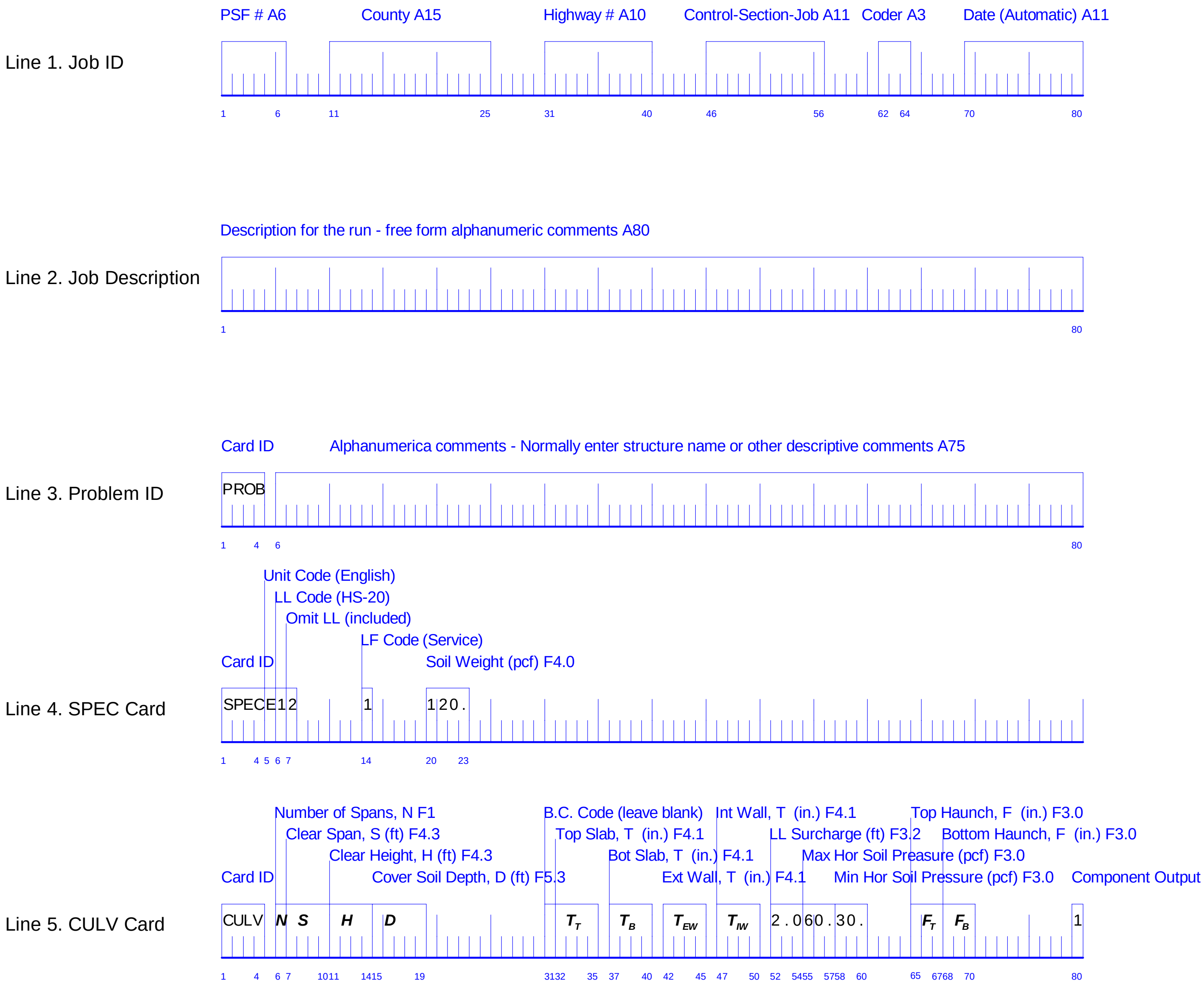

FIGURE VI.8. CULV-5 INPUT FORMAT.

CULV5 Step 2. Run the CULV-5 program using the input file created in Step 1. One of the positive features of CULV-5 is that it is heavily preprogrammed. For culvert rating, this means that all calculations can be made based on output from running the CULV-5 program only one time.

CULV5 Step 3. Interpretation of the CULV-5 output requires establishing both corner and mid-span critical sections.

a. Using Figure VI-9 select the 10th points to set up the linear interpolation associated with the corner critical sections.

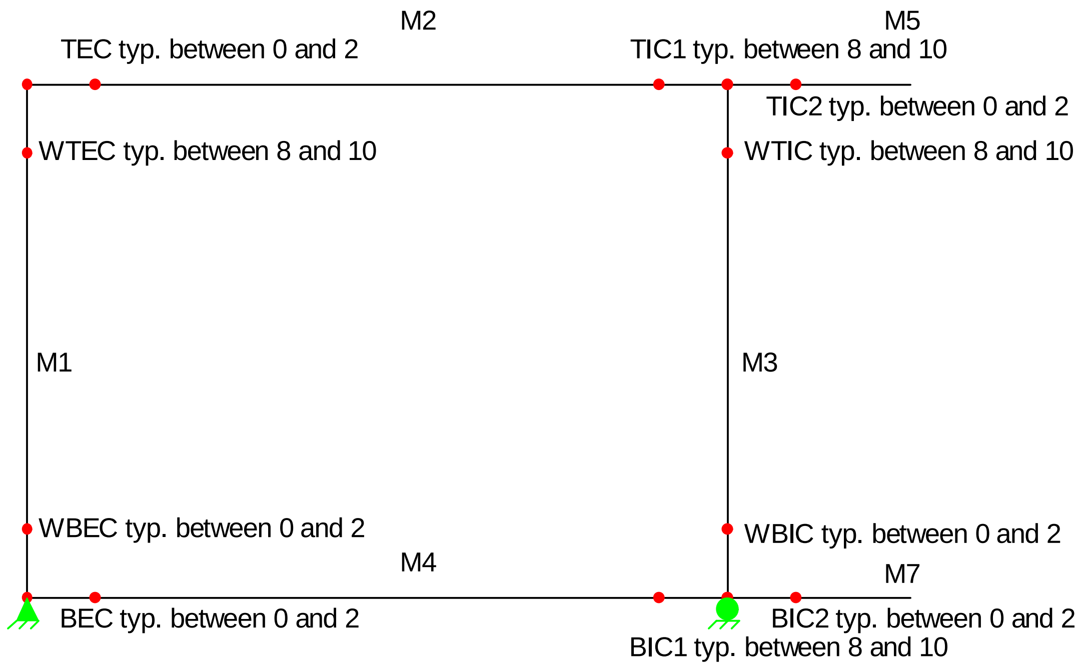

FIGURE VI.9. CULV-5 MEMBER AND CRITICAL SECTION DESIGN.

With reference to Figure VI-9, linear interpolation to establish the corner critical sections must work from the corner to the nearest corner critical section. For example, the upper right corner section, TIC1, for member 2 (M2) might be located between nodes 8 and 9. In this example, the calculation would start with the demands at node 9 and add the fraction between nodes 9 and 8.

b. Critical sections for mid-span demands (TEM, TIM, BEM, BIM, WEM and WIM) do not require interpolation. These may be selected at mid-span (node 5).

CULV5 Step 4. From the CULV-5 output file, select the **SUMMARY OF INDIVIDUAL UNFACTORED MOMENTS, SHEAR AND AXIAL FORCES** tables. Record the vertical dead load (VDL), lateral dead load (LDL), maximum vertical live load (+VLL), minimum vertical live load (-VLL), and lateral live load (LLL) demands at each critical section.

CULV5 Step 5. Calculate the dead and live load demand for each demand type (moment, shear and axial), for each load case at each critical section using Equation VI-9 and Equation VI-10. *D* is the dead load demand and *L* is the live load demand required for rating in Equation II-1. Note that the live load demands will have a maximum and minimum because these derive from a moving load which produces an "envelope" type solution. To maintain a systematic approach, typical practice is to determine both the maximum and minimum live loads for each type of demand at each critical section and to select the minimum (controlling) value when calculating rating factors.

EQUATION VI-9. TOTAL LOAD CASE .

$$D = VDL + LDL$$
$$L = VLL + LLL$$

where: *D* = the dead load demand
L = the live load demand
VDL = the vertical dead load demand from CULV5 output
LDL = the lateral dead load demand from CULV5 output
VLL = the vertical live load demand from CULV5 output. (Load rating calculations for this variable must be done twice, for the maximum and minimum values.)
LLL = the lateral live load demand from CULV5 output

EQUATION VI-10. REDUCED LATERAL LOAD CASE .

$$D = VDL + \frac{1}{2}LDL$$
$$L = VLL$$

where: *D* = the dead load demand
L = the live load demand
VDL = the vertical dead load demand from CULV5 output
LDL = the lateral dead load demand from CULV5 output
VLL = the vertical live load demand from CULV5 output. (Load rating calculations for this variable must be done twice, for the maximum and minimum values.)

CULV5 Step 6. Use Equation VI-1 to verify that actual thrust demand is lower than the incidental axial load assumed in the moment capacity equations.

CULV5 Step 7. This step goes beyond calculation of demand loads and has to do with calculating the culvert load rating. Per the culvert rating flow chart (Figure III-2) proceed to calculate Inventory and Operating rating factors for each critical section, for each demand type, for each load case per Equation II-1.

When calculating the rating factors, exercise extreme care regarding the signs for both demands and capacities.

- Live load and capacity must be in the same sign and direction.
- If the live load and dead load are in opposite directions or the calculated rating factor is negative, a check should be made to insure that the structure has adequate capacity to support the dead load. I.E. $C \geq 1.3D$

CULV5 Step 8. Select the controlling inventory and operating rating factors for each section.

CULV5 Step 9. Select the overall controlling rating factors for the culvert.

CULV5 Step 10. If shear controls the load rating, the load rater should perform a less-conservative analysis of the shear failure mode based on shear critical sections as per the provisions in Section VI.C.

CULV5 Step 11. Calculate the Inventory and Operating Ratings per Equation II-2.

Appendix C of this Guide presents an example Level 1 culvert load rating calculation based on Culv-5.

E. Level 2 Analysis: Two Dimensional Structural Frame Model with Soil Springs

The Level 2 analysis uses a two-dimensional structural frame model and AASHTO loading parameters, but with compression springs to model vertical soil support instead of balanced loading. This is only slightly different from the Level 1 model, but the introduction of the soil springs somewhat reduces the over-conservatism in Level 1. Level 2 is designed to provide a quick, accurate, repeatable load rating, and can be considered the general case for two-dimensional structural frame analysis of reinforced concrete box culverts.

1. Assumptions

The assumptions in the two dimensional structural frame analysis with soil springs (Level 2) are identical to those in the two-dimensional, simply-supported structural frame analysis (Level 1) with the exception of the boundary conditions. The Level 2 model assumes that soil is more accurately modeled using compressive springs at intermediate locations along the bottom slab.

It should be noted that CULV-5 software will not support a Level 2 analysis because of input limitations; namely, CULV-5 does not allow for intermediate compression springs on the bottom slab. Thus, a Level 2 analysis requires the use of two-dimensional frame analysis software programs other than CULV-5.

2. Model Dimensions

Model layout including identifying members and nodes for the Level 2 model is exactly the same as for the Level 1 model. Refer to Section VI.D.2 and Figure VI-2 for details.

3. Boundary Conditions

In the Level 2 model, the boundary conditions serve two primary functions. One is to maintain global stability. To that end, the bottom left hand corner of the model should be restrained in the global X direction. The second function is to provide displacement-dependent resistance to the vertical loads by supporting the culvert with compression springs.

Relative to laying out the model and establishing boundary conditions, extra nodes should be added along the bottom element of the model (depicting the bottom slab) to create 10 spaces. These nodes should be restrained using compression springs in the global Y direction. The compression springs must have a stiffness associated with an appropriate modulus of subgrade reaction, *k*, as per Table IV-2. The spring constant, κ (pli), can then be determined using Equation VI-11. See Figure VI-10.

Note that this model does not place lateral springs on elements used to model the culvert sidewalls. This is because the culvert sidewalls are modeled to receive lateral soil loads (trapezoidal pressure distribution). Good structural modeling practice dictates that loads and boundary conditions springs are not applied at the same location.

EQUATION VI-11. SPRING CONSTANT EQUATION

$$\kappa = k * s * b$$

Where: κ = the spring constant (pli)

k = the modulus of sub-grade reaction (pci)

s = the tributary length associated with the node (in.) (This is equal to the span length divided by 10)

b = the unit slab width (12 inches)

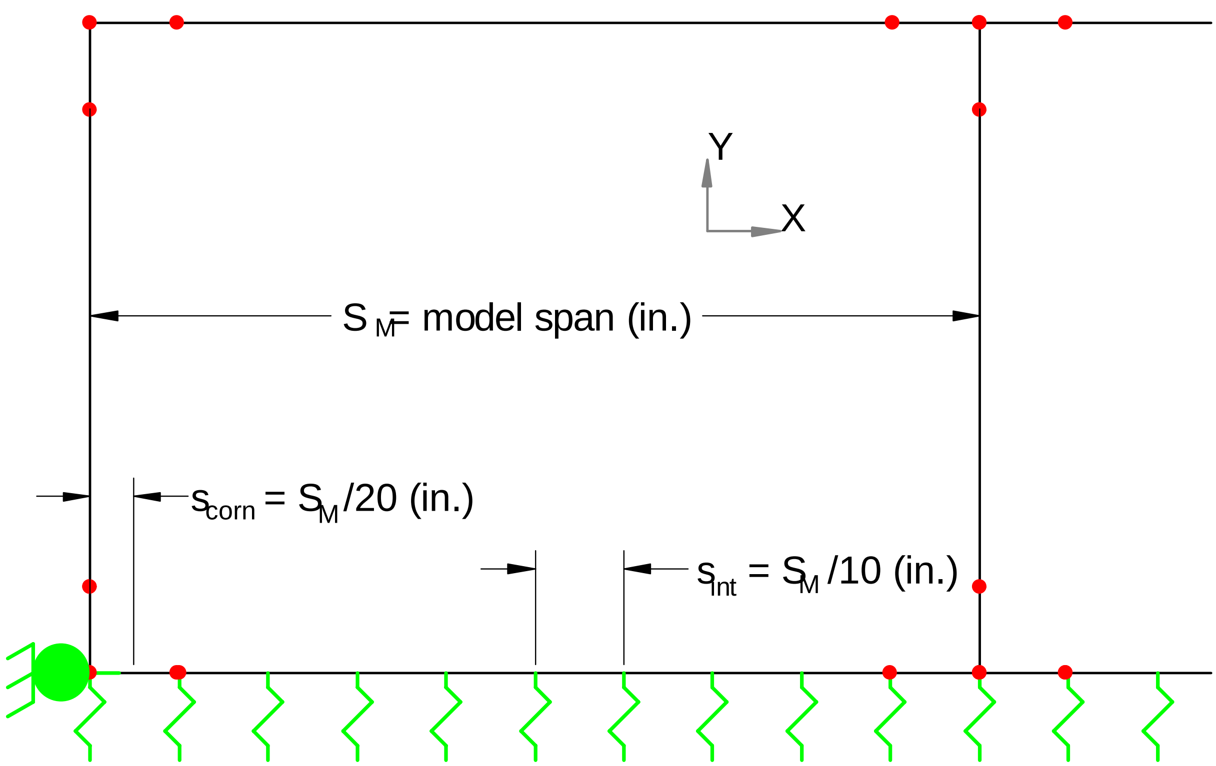

FIGURE VI.10. BOUNDARY CONDITIONS FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

4. Loads

The loads placed on the structure for Level 2 modeling correspond directly to the provisions of the AASHTO policy. The loads are as follows:

- DL_V … Vertical Dead Load
- DL_{hT} …Horizontal Dead Load, top of culvert
- DL_{hB} … Horizontal Dead Load, bottom of culvert
- LL_{VT} … Vertical Live Load, top slab
- LL_h … Horizontal Live Load

The loads for a Level 2 model are the same as those used for Level 1 with one exception. In the Level 2 model, no upward loads are needed on the bottom slab. The spring supports automatically provide the necessary uplift, and they do so more realistically. The spring support eliminates the need to calculate the self-weight and the live load applied to the bottom of the slab. Otherwise the details and loading philosophy is the same as for the Level 1 model. See the Level 1 model for a more thorough explanation. Figure VI-11 illustrates the loading convention for a Level 2 model.

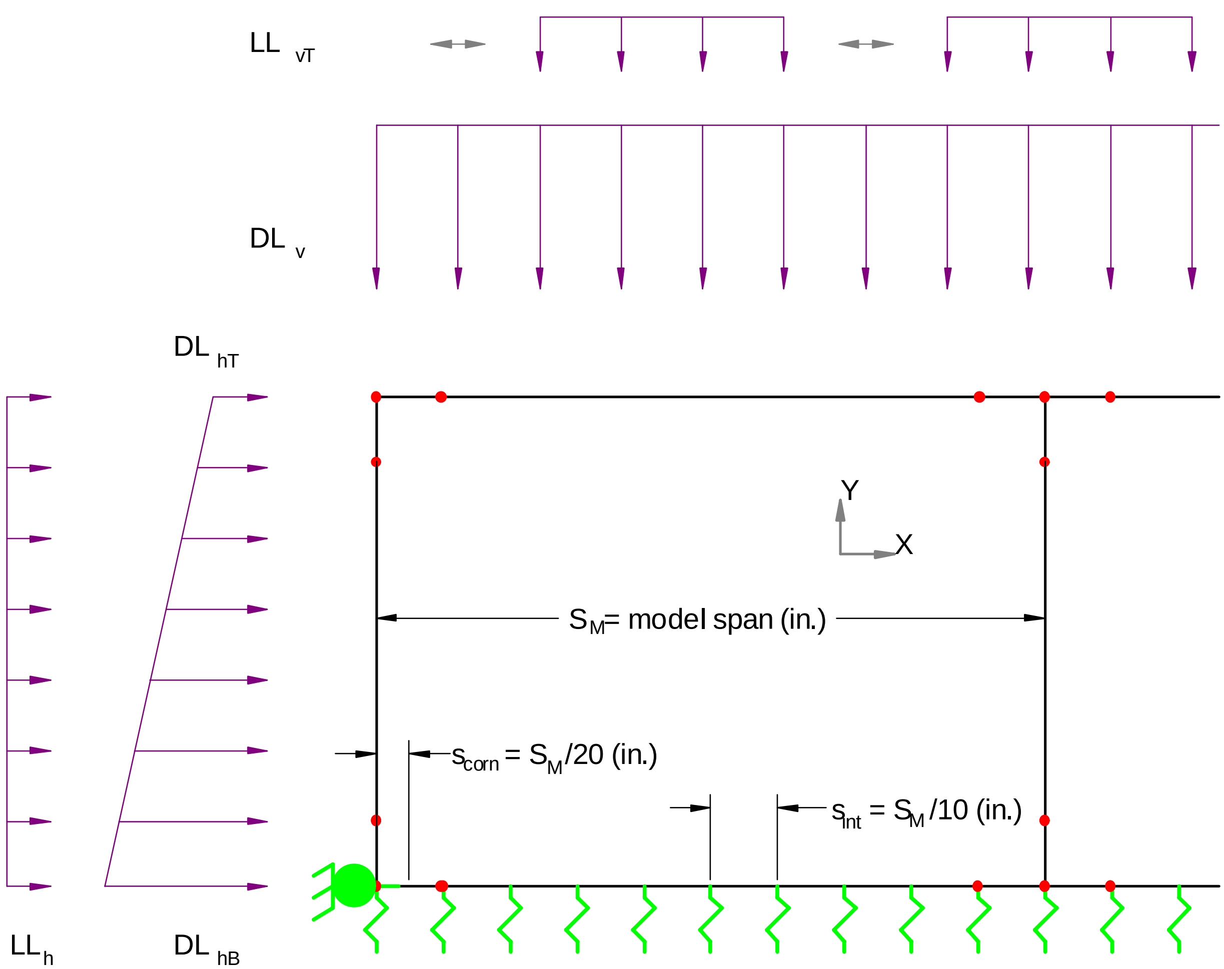

FIGURE VI.11. LOAD CONVENTIONS FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

a. Dead Load

The first load represents the weight of the soil on top of the structure, DL_V. Please see Equation VI-2.

The second load represents the self-weight of the structure (this is not shown in Figure VI-11 for the purposes of clarity). If the chosen analysis tool has a gravity feature, this should be used to accurately distribute the self-weight across the structure. Otherwise, the weight of the top slab and walls should be applied downward to the top slab and the weight of the bottom slab should be placed downward on the bottom slab. See Equation VI-3.

The third load is the horizontal dead load. Please see Equation VI-4 and Equation VI-5.

b. Live Load

There are two live loads due to the HS-20 truck: (1) the horizontal live load, LL_h, and (2) the vertical live load applied to the top slab, LL_{vT}. The impact factor, *IM*, and all other variables used in the live load equations are defined in Table III-1.

Per Equation VI-6, the first live load is the horizontal live load, LL_h *(ksf)*. This load is constant regardless of the number of trucks passing over the culvert. AASHTO *SSHB* 3.20.3 provides a 2 ft surcharge allowance for trucks which are approaching, but not directly above, the culvert.

The vertical live load applied to the top of the culvert, LL_{vT} *(ksf),* is the second type of live load. The magnitude of the vertical live load depends on the depth of fill, the wheel load, the culvert span, the impact factor, the number of lanes, and the number of trucks. For this Guide, the vertical live load has been expressed in terms of 15 distinct equations derived from AASHTO *SSHB* 3.7.6, 3.12.1, 3.24.3.2 and 6.4, including the lane reduction factor described in AASHTO *SSHB* 3.12.1. These 15 equations are collectively designated as Equation VI-7. For a given culvert, the load rater must select one of the 15 equations to determine the magnitude of the vertical live load. This is the same as for the Level 1 model.

Once the magnitude of the live load has been established, it is necessary to define the area over which the live load acts. The vertical live load should be applied as a *moving load* across the top of the culvert structure with the load moving from left to right and from right to left. This will have the effect of creating a moment envelope, with both maximum and minimum values. The length over which the pressure should be applied, the center-to-center spacing for the distributed loads, and the wheel load, *P*, used to calculate each load are illustrated in Figure VI-12, Figure VI-13 and Figure VI-14 for different cover depths.

5. Load Cases

Load cases for the Level 2 model are the same as for the Level 1 model.

6. Demand Load Calculations

Having created the analytical model, defined the boundary conditions, determined the magnitude and extent of loads, and specified the load cases, the next step is to calculate the moment, shear and thrust demands. This requires application of an appropriate structural analysis software package, as discussed in the following section of the Guide.

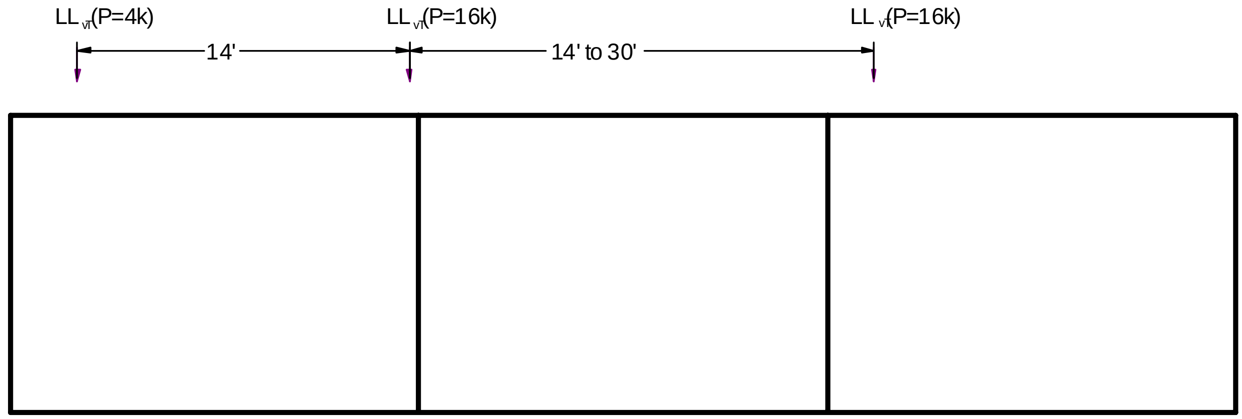

FIGURE VI.12. LIVE LOAD DISTRIBUTION FOR *D* < 2' FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

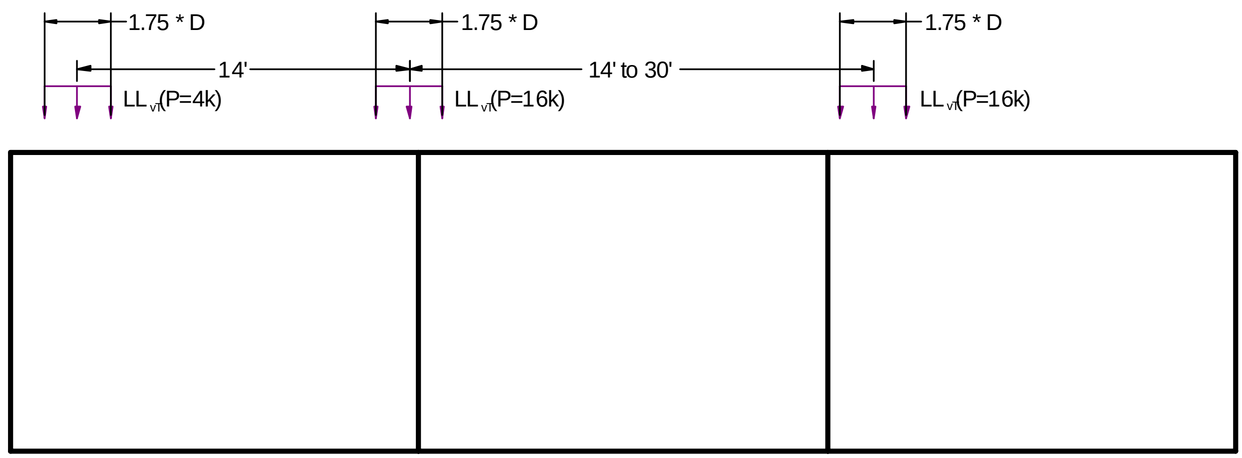

FIGURE VI.13. LIVE LOAD DISTRIBUTION FOR 2' < *D* < 8' FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

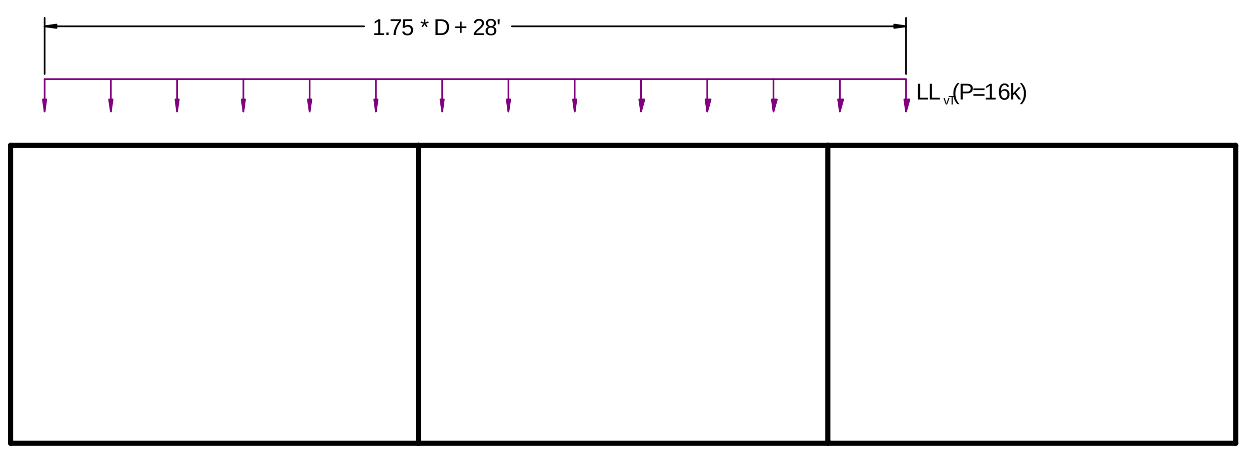

FIGURE VI.14. LIVE LOAD DISTRIBUTION FOR *D* > 8' FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

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7. Analytical Program – RISA-2D with Spring Supports

a. Overview

Most analytical frame programs with compression spring foundation capabilities, such as RISA-2D, STRUDL and others, should be adaptable to the specifications outlined for this modeling level. Because it is widely available throughout TxDOT, specific guidance will be given for RISA-2D. If a load rater is more comfortable with another frame analysis program, he/she is free to use it.

RISA-2D is a commercially available two-dimensional frame analysis program. Versatility of input makes RISA-2D a strong contender for culvert load rating analysis, though it does require more preparatory hand calculations than CULV-5.

b. RISA-2D Strengths and Limitations

The RISA-2D program has some notable strengths that make it the ideal first choice for a Level 2 culvert load rating program. These are:

- Program inputs are graphically based
- Deflections, shear, thrust and moments can be represented and analyzed graphically
- Generality allows for intermediate boundary conditions
- Critical section demands can be determined directly

As noted, the one limitation to RISA-2D is that it requires more preparatory calculations than CULV-5. Recall that CULV-5 is heavily pre-programmed so that the analytical model is generated just by specifying one line of code on a punch card. RISA-2D, however, requires that all nodes and beam elements be individually specified.

c. RISA-2D/Springs Step-by-Step Instructions

Culvert load rating for Level 2 using the RISA-2D program can be accomplished by following these steps. This sequence assumes the load-rater has already defined the input parameters and is prepared to create the RISA-2D analytical model.

RISA-2D Spring Step 1. Calculate all loads using Equation VI-2, Equation VI-4, Equation VI-5, Equation VI-6 and Equation VI-7.

RISA-2D Spring Step 2. Create a model consistent with Figure VI- and Figure VI-10:

a. Disable cracked sections and shear deformations within the global parameters. Reduce output to three points per member.
b. Lay out corner nodes.
c. Connect nodes using members with rectangular cross sections and appropriate concrete properties according to Table II-1 and Table IV-1. Draw members counterclockwise around the center of the culvert to produce consistent moment sign conventions as per Figure VI-15. More specifically:
    - Bottom elements, lay out left to right
    - Top elements, lay out right to left
    - Wall elements left of center, lay out top to bottom
    - Wall elements right of center, lay out bottom to top
    - Wall elements at center (even spans), lay out top to bottom
d. Using the “split member” function, add support nodes to the bottom members and set boundary conditions according to Figure VI-11 with spring constants from Equation VI-11.

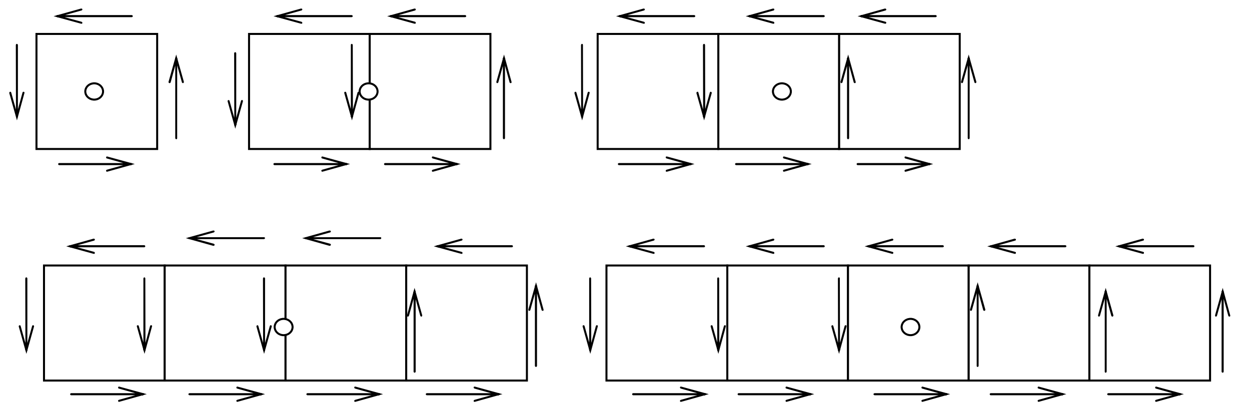

FIGURE VI.15. RISA-2D BEAM ELEMENT LAYOUT PATTERN FOR 1, 2, 3, 4 AND 5-SPAN CULVERTS

RISA-2D Spring Step 3. Apply the loads according to Figure VI-11 in separate Basic Load Cases.

a. Vertical Dead Load, DL_v (Equation VI-2). Be sure to include the self-weight gravity loading by including a factor of (-1) in the Y-gravity direction.
b. Horizontal Dead Load, DL_h (Equation VI-4 and Equation VI-5)
c. Horizontal Live Load, LL_h (Equation VI-6)

RISA-2D Spring Step 4. Vertical Live Load, LL_{VT} (Equation VI-7) must be calculated and placed as a *moving* load as seen in Figure VI-12, Figure VI-13 and Figure VI-14. The moving load will be approximated by creating a *moving load pattern* of 10 equivalent, uniformly-spaced, point loads over the length of each load as seen in Figure VI-16, Figure VI-17 and Figure VI-18. These figures show the moving load discretized and grouped in terms of the 10 equivalent, uniformly-spaced, point loads. The load should be applied moving from right to left and from left to right.

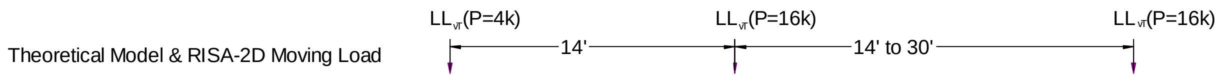

FIGURE VI.16. RISA-2D MOVING LOAD PATTERN FOR *D* < 2' FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

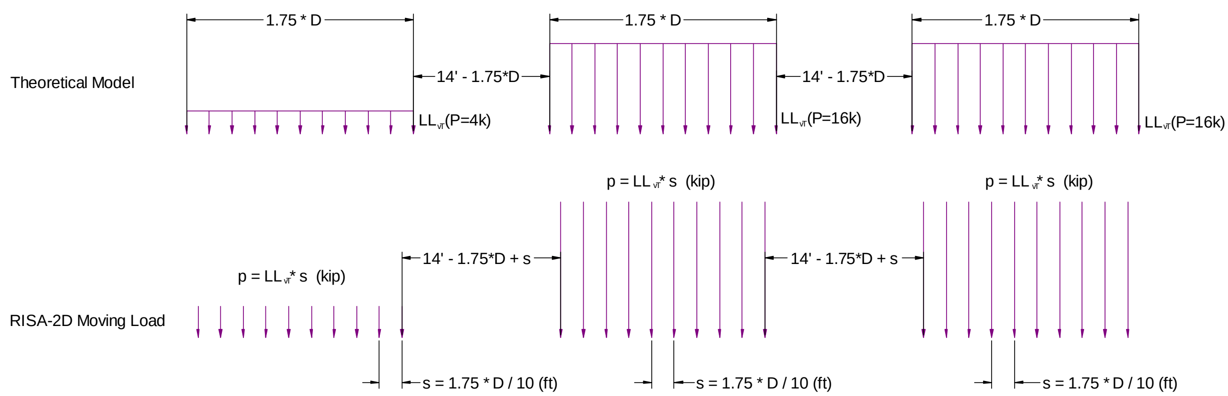

FIGURE VI.17. RISA-2D MOVING LOAD PATTERN FOR 2' < *D* < 8' FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

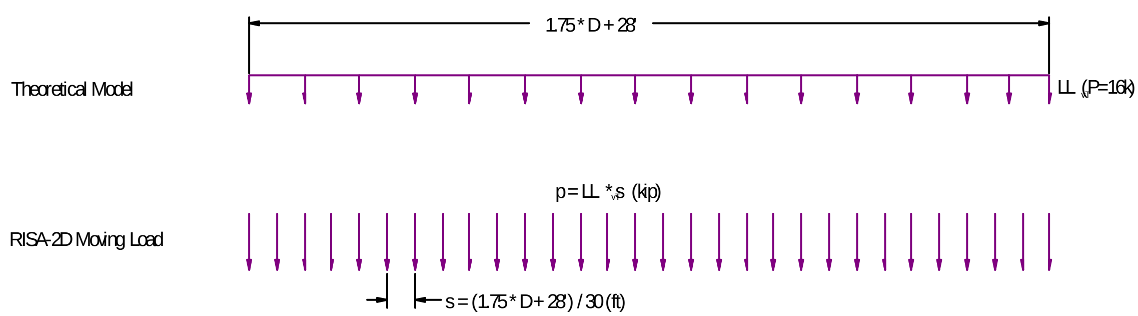

FIGURE VI.18. RISA-2D MOVING LOAD PATTERN FOR *D* > 8' FOR TWO DIMENSIONAL STRUCTURAL FRAME ANALYSIS WITH SOIL SPRINGS.

RISA-2D Spring Step 5. Use the "split member" function to split the members and create the critical section nodes. Re-label and sort the critical members using a convention similar to the CULV-5 naming convention. See Figure VI-19.

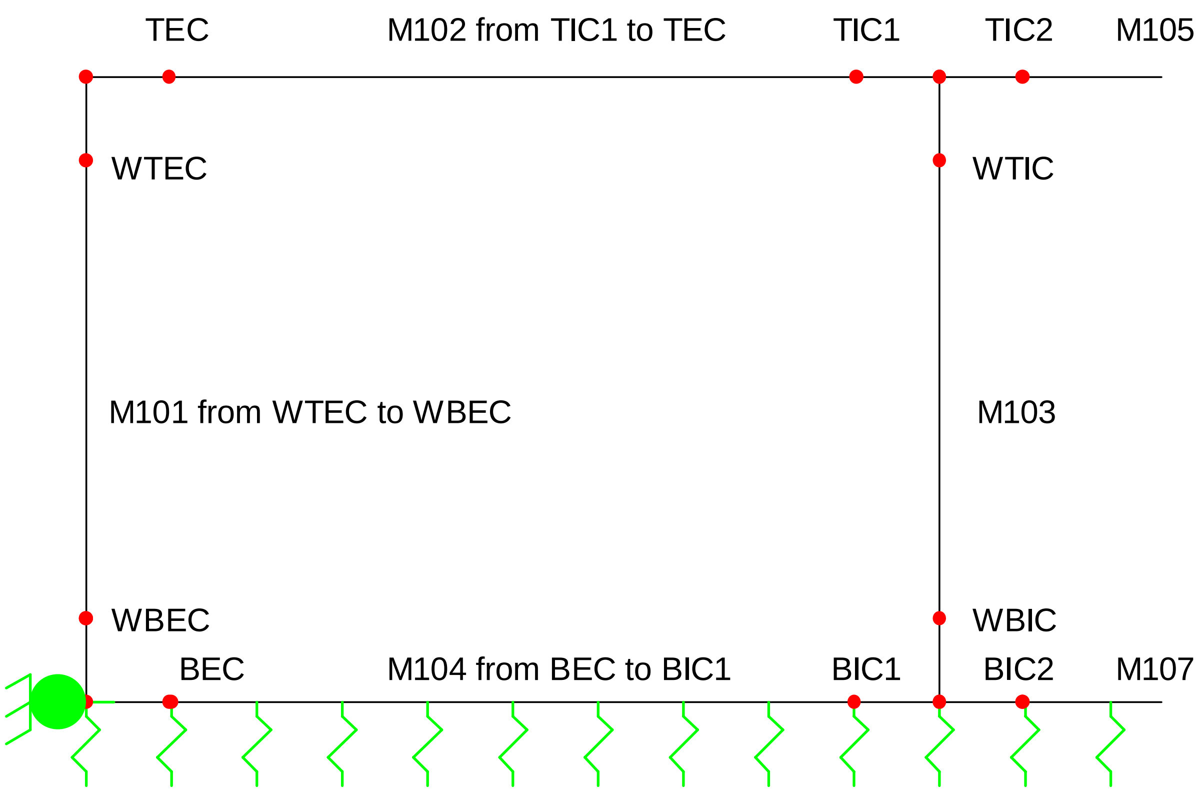

FIGURE VI.19. RISA-2D MEMBER NAMING CONVENTION.

RISA-2D Spring Step 6. Create four Load Combinations for dead and live demands.

a. Use the following Basic Load Case Factors for the Total Load Case dead load demands
   - DL_v factor of 1.0
   - DL_h factor of 1.0
b. Use the following Basic Load Case Factors for the Total Load Case live load demands:
   - LL_v factor of 1.0
   - LL_h factor of 1.0
c. Use the following Basic Load Case Factors for the Reduced Lateral Load Case dead load demands:
   - DL_v factor of 1.0
   - DL_h factor of 0.5
d. Use the following Basic Load Case Factors the Reduced Lateral Load Case live load demands:
   - LL_v factor of 1.0
   - LL_h factor of 0.0

RISA-2D Spring Step 7. Use RISA-2D to solve for moment, shear and axial demand, dead and live loads separately. This will require four separate computer runs, one for each load combination.

RISA-2D Spring Step 8. Record the dead load and the maximum and minimum live load demands for each critical section for both load cases from the member forces table.

RISA-2D Spring Step 9. Use Equation VI-1 to verify that actual thrust demand is lower than the incidental axial load assumed in the moment capacity equations.

RISA-2D Spring Step 10. This step goes beyond calculation of demand loads and has to do with calculating the culvert load rating. Per the culvert rating flow chart (Figure III-2) proceed to calculate Inventory and Operating rating factors for each critical section, for each demand type, for each load case per Equation II-1.

When calculating the rating factors, exercise extreme care regarding the signs for both demands and capacities.

a. Live load and capacity must be in the same sign and direction.
b. If the live load and dead load are in opposite directions or the calculated rating is negative, a check should be made to insure that the structure has adequate capacity to support the dead load. I.E. $C \geq 1.3D$

RISA-2D Spring Step 11. Select the controlling inventory and operating rating factors for each section.

RISA-2D Spring Step 12. Select the overall controlling rating factors for the culvert.

RISA-2D Spring Step 13. If shear controls the load rating, the load rater should perform a less-conservative analysis of the shear failure mode based on shear critical sections as per the provisions in Section VI.C.

RISA-2D Spring Step 14. Calculate the Inventory and Operating Ratings per Equation II-2.

Appendix D of this Guide presents an example Level 2 culvert load rating calculation based on RISA-2D with springs.

F. Level 3 Analysis: Two Dimensional Finite Element Soil-Structure Interaction Model.

The Level 3 analysis is based on a two-dimensional finite element model of the soil-structure system, and AASHTO vehicle loading parameters. The significant benefit of this model is that it evaluates the *interaction* between the culvert structure and the surrounding soil. Soil is no longer just a load applied to the structural frame (culvert), but instead is an integral aspect of the load resistance portion of the model. Because traffic loads are applied directly to the soil and are transmitted through the soil elements to impact the culvert, this finite element approach obviates the need to use AASHTO assumptions for soil pressure distributions or live load distributions in the direction of traffic.

The defining feature of the Level 3 analysis is that it assumes both the soil and the culvert slab elements behave as *isotropic, linear-elastic materials*. That is, the dominant property for expressing the engineering behavior of these materials is their elastic modulus. This is obviously a simplified view for such complex materials, and other, more sophisticated constitutive models for both the culvert structure and the soil exist. However, for basic load rating analyses where actual material properties are usually not known, the uncertainty introduced by using a simplified linear elastic model for the culvert and soil is consistent with other uncertainties in the modeling process.

1. Assumptions

The assumptions associated with a two-dimensional finite element model are similar to the two dimensional frame analysis with extensions and modifications for using finite elements to model soil behavior and loading.

- AASHTO vehicle load distributions are applied in the transverse direction.
- Body weight of soil elements accurately model soil dead loads.
- A one foot (b = 1 ft) section of the culvert may be analyzed.
- No hydrostatic pressure (water) inside the culvert.
- Supporting soils are fully drained, i.e. no hydrostatic pressure outside the culvert.
- Moments resulting in tension on the inside face of the culvert are positive.
- Moments resulting in tension on the outside face of the culvert are negative.

2. Model Dimensions

The culvert model dimensions for Level 3 are exactly the same as those for Level 2. Refer to Section VI.D.2 and Figure VI-2 for details. If the load rater has already developed a Level 2 culvert model, this can be directly appropriated into the Level 3 analysis.

In addition to the culvert structure, Level 3 requires modeling the subsurface regime; that is, the soil surrounding the culvert. Figure VI-20 illustrates the extent of the soil-structure model. The overall limits of the soil model relative to the culvert are D above, 1.5H below, and 2S on either side of the culvert.

When creating a submesh of the soil elements, it is necessary to define at least 10 soil elements along *each span* of the culvert. Any decrease in the number of soil elements adjacent to the culvert structure will result in significant error relative to how soil loads are transmitted to the culvert model. Soil elements should be approximately square.

3. Boundary Conditions

Boundary conditions for the Level 3 model must mimic continuous soil surrounding the culvert. This means that the outside edges of the model space (soil) will be restrained in the global X direction, while the bottom edge of the model space (soil) will be restrained in the global Y direction. The culvert portion of the model is not restrained by boundary conditions for the Level 3 model.

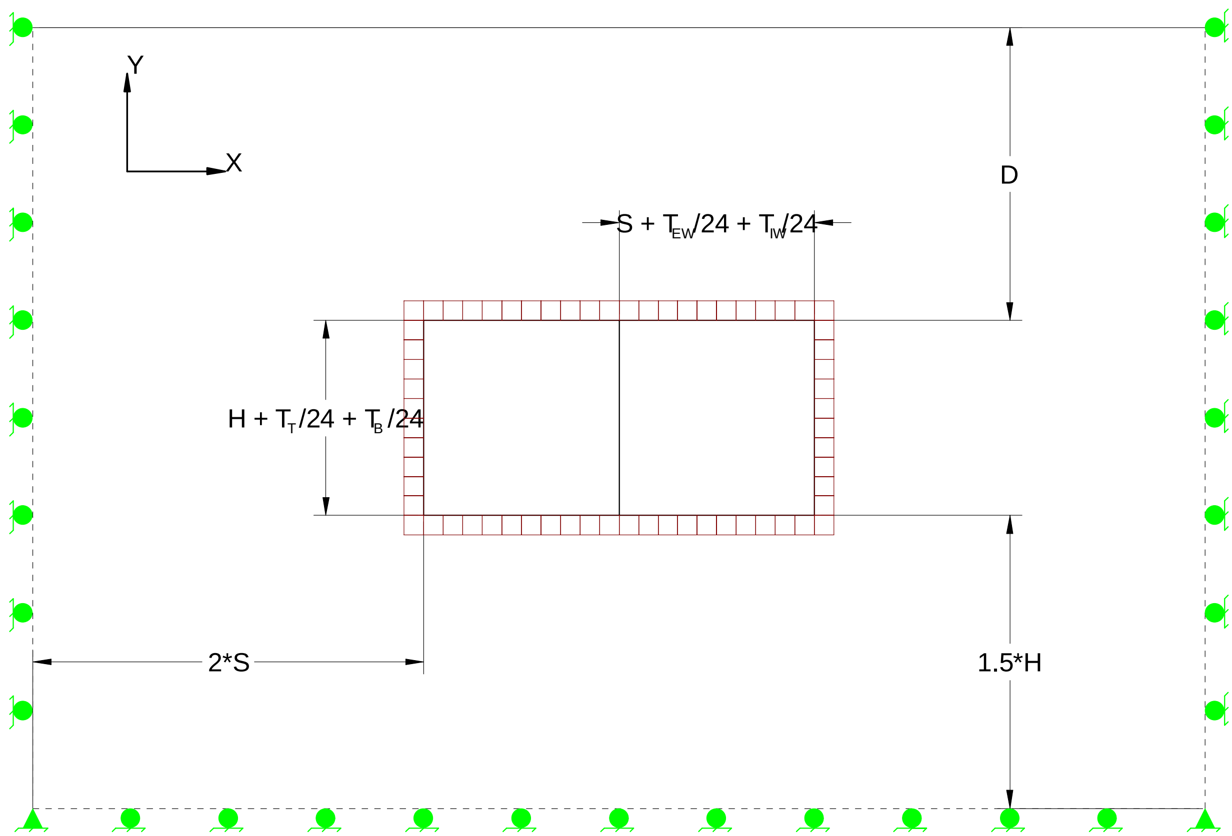

FIGURE VI.20. SOIL STRUCTURE INTERACTION MODEL LAYOUT.

4. Loads

Loads for the Level 3 model differ from the Level 1 and Level 2 models because the soil is part of the modeled system and is no longer another load applied to the culvert frame.

This means that all dead loads, both for soil and for the culvert, should be modeled by the body force (gravity) on the respective finite elements.

Live loads must still be established using AASHTO traffic loading guidelines. However, for Level 3, the only traffic load is the vertical live load, $\boldsymbol{LL_{VT}}$. This traffic load must be converted to point loads which are applied to the "soil" surface. This is conceptually different from the $\boldsymbol{LL_{VT}}$ calculations for Levels 1 and 2.

Previously, $\boldsymbol{LL_{VT}}$ represented a distributed load applied to the *culvert model*. The magnitude of $\boldsymbol{LL_{VT}}$ corresponded to an attenuated traffic pressure calculated based on the depth of fill, the wheel load, the culvert span, the impact factor, the number of lanes, and the number of trucks. In other words, the live load pressure acting on the culvert surface was much reduced (attenuated) from the tire contact pressure due to the distance between the point of application (where the rubber hits the road) and the culvert top slab (located some distance below ground surface). The attenuation calculations account for prismatic spreading of the load with depth, both in-plane (parallel to culvert cross section) and out-of-plane (perpendicular to culvert cross section).

For Level 3, the in-plane traffic pressure (wheel load) is modeled directly, since it can be directly applied to the soil surface. The modeling challenge, therefore, is to define the magnitude of this pressure ($\boldsymbol{LL_{VT}}$) such that it reasonably accounts for out-of-plane attenuation at the culvert surface. To accomplish this, the traffic load is distributed over a certain distance in the out-of-plane direction to establish the distributed load applied to the culvert top slab. This attenuated load, which is a modified type of $\boldsymbol{LL_{VT}}$ specifically for a Level 3 model, is applied as a point load to the surface soil elements.

For this Guide, the vertical live load has been expressed in terms of 10 distinct equations derived from AASHTO (AASHTO, 2007). These 10 equations are collectively designated as Equation VI-12.

As with the Level 1 and Level 2 models, for a given culvert, the load rater must select one of the 10 equations to determine the magnitude of the vertical live load. Two variables govern selection of the appropriate live load equation. The first is the number of lanes passing over the culvert. The second variable is the depth of fill, D. This fill depth will yield the proper load configuration as per the AASHTO stress distribution. Taken together, the number of lanes and the fill depth establish the controlling number of trucks and identify the proper equation to use for LLvT.

The modified point loads are applied to the top of the soil as *moving loads* as per the HS-20 load pattern moving from right to left and left to right. Figure VI-21 illustrates the center-to-center spacing for the modified point loads. Just to be clear, the modified point loads are applied to the "soil" surface as point loads, not as tire contact pressures.

EQUATION VI-12. VERTICAL LIVE LOAD APPLIED TO THE TOP SOIL MASS FOR LEVEL 3 ANALYSIS, LL_{vT} (KLF)

Number of Traffic Lanes	*Depth of Fill D (ft)*	*Magnitude LL_{vT} (klf)*	*Controlling No. of Trucks*
1	0 < D < 2'	$LL_{vT} = \frac{(1+IM)*P}{4+0.06*S}$	1 truck
1	2' < D < 3.8'	$LL_{vT} = \frac{(1+IM)*P}{1.15*\mathrm{D}+1.67'}$	1 truck
1	3.8' < D	$LL_{vT} = \frac{2*P}{1.15*\mathrm{D}+7.67'}$	1 truck
2	0 < D < 2'	$\mathrm{LL_{vT}} = \frac{(1+\mathrm{IM})*\mathrm{P}}{4+0.06*\mathrm{S}}$	1 truck
2	2' < D < 3.8'	$\mathrm{LL_{vT}} = \frac{(1+\mathrm{IM})*2*\mathrm{P}}{1.15*\mathrm{D}+5.67'}$	2 truck
2	3.8' < D	$\mathrm{LL_{vT}} = \frac{4*\mathrm{P}}{1.15*\mathrm{D}+17.67'}$	2 trucks
3+	0 < D < 2'	$LL_{vT} = \frac{(1+IM)*P}{4+0.06*S}$	1 truck
3+	2' < D < 3.8'	$\mathrm{LL_{vT}} = \frac{(1+\mathrm{IM})*2*\mathrm{P}}{1.15*\mathrm{D}+5.67'}$	2 truck
3+	3.8' < D < 9.4'	$\mathrm{LL_{vT}} = \frac{4*\mathrm{P}}{1.15*\mathrm{D}+17.67'}$	2 trucks
3+	9.4' < D	$\mathrm{LL_{vT}} = \frac{.9*6*\mathrm{P}}{1.15*\mathrm{D}+27.67'}$	3 trucks

where: $\boldsymbol{LL_{vT}}$ = the vertical live load on the top slab (ksf)
IM = the impact factor from Table II-1
S = the clear span of a single box (ft)
P = either 4 or 16 kips as indicated in Figure VI-5 through Figure VI-7
D = cover soil depth (ft)

5. Load Cases

The Level 3 approach directly models the culvert-soil interaction, so there are no externally-applied lateral loads. This means there is no need for the "total" and "reduced lateral" load cases as per the Level 1 and Level 2 analyses. The demand at corner critical sections and midspans for the Level 3 analysis is what it is.

6. Demand Load Calculations

Having created the analytical model, defined the boundary conditions, and determined the magnitude and extent of loads, the next step is to calculate the moment, shear and thrust demands. This requires application of an appropriate structural analysis software package, as discussed in the following section of the Guide.

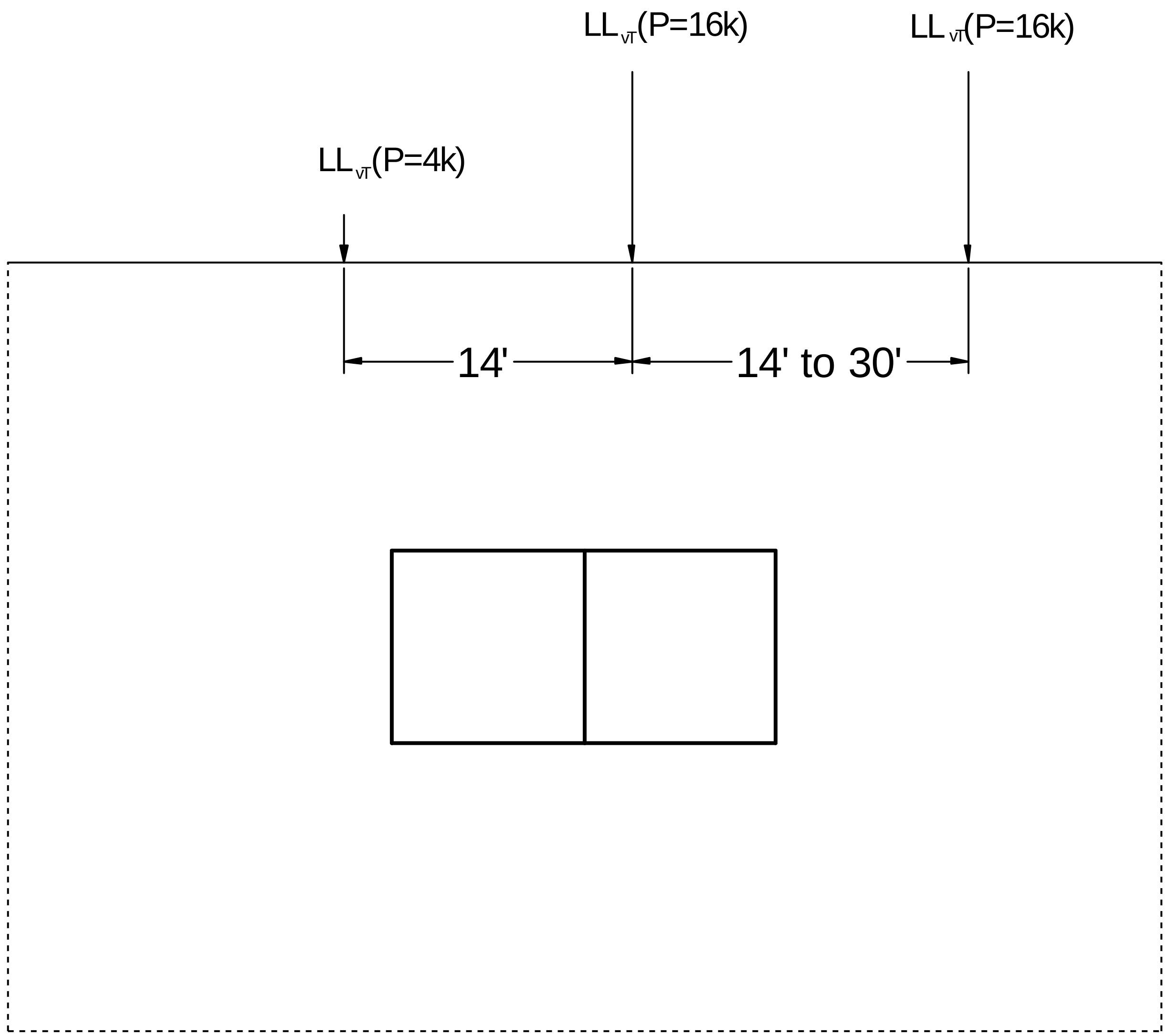

FIGURE VI.21. LIVE LOAD DISTRIBUTION FOR TWO DIMENSIONAL FINITE ELEMENT MODEL WITH SOIL STRUCTURE INTERACTION.

7. Analytical Program – RISA-2D with Linear Elastic Finite Elements (LEFE)

a. Overview

Numerous computer programs are available to perform two-dimensional, finite-element modeling of the culvert-soil system. Some have their origins in structural modeling, such as RISA. Others have their origins in geotechnical modeling, such as PLAXIS. Still others have been specifically designed with culverts/buried pipes in mind and feature both complex structural response systems and multiple soil models, such as CANDE. Still more powerful finite element and finite difference programs, such as ABACUS and FLAC, are available which can model complex structure geometries and represent nonlinear variation of soil stiffness with strain and anisotropy.

Notwithstanding the technological pull of increasingly complex programs available for modeling the culvert-soil system, certain features of the *culvert rating problem* suggest that for many applications, flexibility and ease-of-use are preferable to computational sophistication. First, culvert load rating requires application of a moving load across the culvert-soil system, and this necessitates specification of a non-symmetrical analytical model. Default models for this application are not available, so they must be generated from the most basic input fields. Setting up models this way is tedious, time-consuming, and highly susceptible to user error.

Second, project-specific data for the culvert and soil engineering properties are rarely available. Most raters will use default parameters for concrete and steel (taken from the construction drawings), and they will assume basic strength parameters for soil. This practice is not unreasonable, and is the rule rather than the exception. However, the potential for error introduced by these typical practices will, in most situations, overshadow benefits that a more complex analytical model may bring to the solution. This means that much of the benefit – that is, more refined determination of the moment, shear and thrust demands – from the more advanced programs is rarely, if ever, realized.

A third factor is that culvert rating calculations do specify dead and live loads, but they do not presume to specify the extent to which the culvert can suitably support these loads. That is what the rating process is meant to determine. Thus, a “weak” or “flexible” culvert will not support the applied load – it will fail either by excess deflection or inadequate structural capacity. The more complex structure and soil models will certainly depict this failure. But while predicting failure for a weak culvert is a good thing, it does not yield a load rating. It simply shows a particular culvert will not work. This means that with the more complex analytical models, determination of the load rating for a weak culvert may require iterative reduction of traffic loads until the culvert does not fail. This manual convergence solution approach adds more work to the culvert rating process.

With these factors in mind, for the purposes of Level 3 analyses under this Guide, the decision has been made to recommend using RISA-2D with linear-elastic finite elements (LEFE) to model the soil structure interaction. RISA-2D seems to reasonably balance computational rigor against the unique requirements of the culvert rating problem.

b. RISA-2D Strengths and Limitations

The RISA-2D program has some notable strengths that make it an ideal choice for a Level 3 culvert load rating program. These are:

- Program inputs are graphically based
- Deflections, shear, thrust and moments can be represented and analyzed graphically
- The program models in-plane behavior of plates very well
- Generality allows for intermediate boundary conditions
- Critical section demands can be determined directly

One limitation to using RISA-2D is that the constitutive models for both concrete and soil are limited to the linear elastic model. However, as noted above, this approach allows for direct calculation of the rating factor.

c. RISA-2D/LEFE Step-by-Step Instructions

Culvert load rating for Level 3 using the RISA-2D program can be accomplished by following these steps. This sequence assumes the load-rater has already defined the input parameters and is prepared to create the RISA-2D analytical model.

RISA-2D LEFE Step 1. Modify the model created for the Level 2 analysis to match Figure VI-20 through the following steps:

a. Remove all Level 2 boundary conditions and loads.

b. Place new nodes at the outside corners of the soil area as well as at the edges directly above, below and outside the outside corners of the culvert according to Figure VI-20.

c. Connect the nodes from RISA-2D LEFE Step 1.b using the plate drawing tool to make eight large soil elements surrounding the culvert and filling the soil area.

   i. The elements should have the material properties from Table IV-3.

   ii. The elements should be 12 in. thick.

d. Use the "submesh" tool to automatically submesh the large plates. Be sure to specify a minimum of 10 elements along each culvert span.

e. Create a thin "soil" beam at the ground surface, running from the top left corner of the soil area to the top right corner. This is necessary to facilitate the application of the moving live load. It is required by a limitation in RISA-2D which requires moving loads to be applied to beams only.

f. Set the boundary conditions for the outside edge of soil mesh, as shown in Figure VI-20.

RISA-2D LEFE Step 2. Establish the RISA load cases for dead load and live load, as discussed in Section VI.F.4.

a. The dead load is simply a -1 gravity loading in the Global Y direction

b. The live load is a moving load of magnitude and spacing as illustrated in Figure VI-21 and calculated in Equation VI-12 along the soil "beam" created in step 2.e. Use the check box to run the load both directions along the beam.

RISA-2D LEFE Step 3. Use RISA-2D to solve for moment, shear and axial dead and live loads separately. This will require two separate computer runs, one for each load combination.

RISA-2D LEFE Step 4. Record the maximum and minimum demands at each critical section from the member forces table.

RISA-2D LEFE Step 5. Use Equation VI-1 to check that actual thrust demand is lower than the incidental axial load assumed in the moment capacity equations.

RISA-2D LEFE Step 6. This step goes beyond calculation of demand loads and has to do with calculating the culvert load rating. Per the culvert rating flow chart (Figure III-2) proceed to calculate Inventory and Operating rating factors for each critical section, for each demand type, for each load case per Equation II-1.

When calculating the rating factors, exercise extreme care regarding the signs for both demands and capacities.

a. Live load and capacity must be in the same sign and direction.

b. If the live load and dead load are in opposite directions or the calculated rating is negative, a check should be made to ensure that the structure has adequate capacity to support the dead load. I.E. $C \geq 1.3D$

RISA-2D LEFE Step 7. Select the controlling inventory and operating rating factors for each section.

RISA-2D LEFE Step 8. Select the overall controlling rating factors for the culvert.

RISA-2D LEFE Step 9. If shear controls the load rating, the load rater should perform a less-conservative analysis of the shear failure mode based on shear critical sections as per the provisions of Section VI.C.

RISA-2D LEFE Step 10. Calculate the Inventory and Operating Ratings per Equation II-2.

Appendix E contains a culvert rating example using RISA-2D with LEFE.

VII. The General Analytical Model for Culvert Load Rating

A. The Level 4 Analysis Defined

Level 4 analyses go beyond the computational sophistication of the Level 1, Level 2, and Level 3 analyses discussed in this Guide. The Level 4 analysis for culvert load rating means the engineer uses a more advanced modeling approach to determine culvert load demands, culvert capacity, or both. Level 4 analyses will, at a minimum, model soil-structure interaction effects. Level 4 analyses may use either two-dimensional or three-dimensional models for the culvert-soil system.

It is important to emphasize that Level 4 means a higher level of analytical sophistication and not better quality input. Each of the modeling approaches described in this Guide (Level 1, Level 2, or Level 3) allows the use of default input parameters such as may be obtained from policy, from construction drawings, or from this Guide. Results from these same modeling approaches may be *enhanced* by using project-specific input parameters, such as actual concrete compressive strength values, actual reinforcing steel tensile strength values, actual soil modulus values, and so on. However, for Level 4 modeling, it is assumed that project-specific values will always be used. The point is that a more sophisticated model warrants more refined project inputs. It is a waste of effort to create a Level 4 culvert model but populate it with default or handbook material parameters.

It is also important to emphasize that the goal of a Level 4 analysis is a more accurate assessment of the live load capacity of the culvert structure; i.e., a *better* load rating. Usually, but not always, this translates to a higher load rating than would be obtained from one of the lesser analyses. This is because Level 4 models demand loads (moments, shears, and axial thrusts) in a more refined way, and when the demands are more correctly modeled, they are generally less conservative. But nothing in the load-rating process requires that the inventory and operating ratings from a Level 4 analysis be higher than for a Level 3 analysis.

B. When to use a Level 4 Analysis

As discussed in Chapter III, the level of analysis chosen is a trade-off between sophistication of analysis and required work effort. The simpler methods are frequently selected as a first choice due to the need to analyze many structures with limited resources.

When a lower-level analysis yields satisfactory results, there is no need to use a more sophisticated model. Satisfactory results would be the establishment of safe load carrying capacity that does not require posting the structure and does not unduly restrict the flow of permitted overweight trucks. A more sophisticated analysis is justified to avoid posting the structure or to ease restrictions on the flow of permitted overweight trucks.

Typically, then, a Level 4 analysis may be justified when a Level 3 analysis (performed using project-specific input parameters) indicates that a culvert must be load-posted, even when in the judgment of the engineer-inspector, load-posting is not necessary. Economics also enters into the decision-making process. The engineer must evaluate the cost and effort associated with conducting a Level 4 analysis against alternative courses of action.

Level 4 analysis will be required if the culvert is anything other than a reinforced concrete box culvert. Reinforced concrete box culverts are the most common type of culvert used by TxDOT, and Level 1, Level 2, and Level 3 analyses assume that the structure *is* a reinforced concrete box culvert. However, if the culvert is manufactured from other material such as aluminum, plastic, or steel, or if the culvert shape is other than rectangular box such as an arch or a pipe, the Level 4 analysis will be required.

Research-oriented studies are another potential application for Level 4 analyses. For example, interpretation of load test data for a culvert structure might require comparison of member stresses obtained from the load test with predicted stresses obtained from culvert modeling. In this case, the use of more sophisticated models is probably warranted.

The important thing to keep in mind is that the Level 4 analysis represents the most general modeling approach, but requires the most specific project input parameters. This means that considerable cost, effort and time will be required both to create the Level 4 analytical model, and to obtain the input parameters from which the model will yield meaningful results.

C. Comments on Two-Dimensional vs. Three-Dimensional Models

Both two-dimensional and three-dimensional finite-element models can be used for a Level 4 analysis of the culvert-soil system. Numerous computer programs are available to create these models. Comments are as follows.

1. Level 4 Analysis with a Two-Dimensional Model

If a two-dimensional model will be used for a Level 4 culvert load-rating analysis, the recommended computer program for determining demand moments, shears and thrusts is CANDE (**C**ulvert **AN**alysis and **DE**sign). First introduced in 1976 under the sponsorship of FHWA, CANDE is a special-purpose, public-domain finite element program that is used worldwide for the structural design and analysis of buried culverts. CANDE is viewed as highly trustworthy, having been carefully documented and validated through more than 30 years of engineering research and consulting applications. It is computational rigor that makes CANDE superior to the Level 3 model discussed in this Guide (RISA-2D with LEFE).

CANDE was recently upgraded under NCHRP Project 15-28 to create CANDE-2007 and features complex structural response systems and multiple soil models (Mlynarski M. M., 2008). CANDE-2007 provides an elastic solution (CANDE Level 1), automated finite element mesh generation for common configurations (CANDE Level 2), and a user-defined finite element mesh (CANDE Level 3), all in two dimensions. Enhancements over earlier versions of CANDE include an updated finite element analysis engine and graphical tools for interpreting the CANDE output. Documentation for CANDE-2007 includes the *User Manual and Guideline*, *Solution Method and Formulations*, and *Tutorial of Applications*. These are installed with the program.

Relative to culvert load rating, CANDE's primary benefits are: (a) an advanced reinforced concrete constitutive model featuring a tri-linear curve in compression and an abrupt tension rupture at initial tension cracking, (b) five alternative soil models to choose from including isotropic elastic, orthotropic elastic, overburden dependent, Duncan and Duncan/Selig, and extended Hardin, (c) the ability to model culvert construction in increments, and (d) calculation of culvert performance in terms of stress-dependent demand-to-capacity ratios. CANDE also includes subroutines to directly facilitate analysis of culvert types other than reinforced concrete boxes.

Notwithstanding its superior computational rigor, CANDE was not specifically designed for culvert load rating and thus is *not* very user-friendly for load rating applications. To load-rate a culvert using CANDE-2007, the user must rely on a *CANDE Level 3 analysis* (the most general level for CANDE). Even when the user is very familiar with the CANDE program, creation of the user-defined finite element mesh and application of moving loads are highly tedious and very time-consuming. Whereas a structural engineer familiar with RISA can likely perform a Level 3 analysis (as discussed in this Guide, using RISA with LEFE) in a few hours, load rating the same culvert using CANDE could take days.

2. Level 4 Analysis with a Three-Dimensional Model

When investigating the performance of culverts with shallow cover subjected to live loads, it has been recognized that three-dimensional attenuation of the live load takes place both through the soil and the structure. In most cases evaluating live load effects in two dimensions leads to conservative designs, as the longitudinal distribution of load is underestimated. Thus, three-dimensional analysis of the load-rating problem should lead to better results.

However, several modeling issues must be suitably addressed to solve the three-dimensional culvert rating problem. These include but are not limited to specification of the structural model, modeling the vehicle (live load) geometry, selection of the soil and reinforced concrete constitutive models, inclusion of soil shear failure and stiffness variation with depth, modeling of longitudinal bedding to support the culvert structure, and modeling of culvert joint effects. To this end, a Level 4 analysis based on three-dimensional modeling of the culvert-soil system represents the most advanced approach to culvert load rating.

Several software programs can be used for three-dimensional culvert load rating applications, some of the more prominent examples being ABAQUS, ANSYS, FLAC3D, and PLAXIS. Among commercially-available software programs suitable for three-dimensional modeling, ABAQUS stands out. ABAQUS is a general-purpose finite element analysis code, but it has been successfully programmed to provide realistic simulations that allow accurate predictions of soil deformations and soil-structure interactions. ABAQUS features a well-developed graphical interface and compatibility with various CAD programs which enhance its usability.

D. Practical Considerations for Level 4 Culvert Load Rating Analyses

This Guide recognizes a hierarchy of analysis for performing the demand calculations, with Level 4 being the most general and the most sophisticated modeling approach. Level 4 analyses are warranted only for specialized applications. Typically, a Level 4 culvert load rating analysis would be done only if a Level 3 analysis based on project-specific input data fails to yield satisfactory results.

Level 4 analyses require high-quality, project-specific input data for the culvert structural properties, soil properties, and vehicle loads. Because Level 4 analyses are the most general, they are also the most complex and difficult to create. Numerous engineering decisions must be made to fully specify the model.

Successful modeling at Level 4 presumes that the load rater has a strong background in structural modeling in general and the culvert load rating process in particular. Even under these conditions, it should be expected that Level 4 analyses will be highly complex, time-consuming, and costly to perform.

VIII. LIMITATIONS

This *Culvert Rating Guide* has been developed in order to present a clear, repeatable and valid procedure for TxDOT engineers and their consultants to use when load rating culverts in the TxDOT roadway system. Through TxDOT research project 0-5849, the analytical approaches described herein were validated by load rating a representative sample of TxDOT culvert designs from each of TxDOT's culvert design eras, by performing a parametric study, and through comparative analyses using data from a very limited field instrumentation and load test program consisting of three reinforced concrete box culverts bedded in drained, low-to-medium quality backfill soil under low depth of fill. While the principles outlined in this Guide are applicable to other applications, because of inherent diversity of many aspects of the culvert load rating process including culvert type, soil backfill conditions, drainage, analytical modeling tools, and others, certain limitations must be noted.

A. CULVERT TYPE

This Guide has been developed for load rating cast-in-place reinforced concrete box culverts, which are the most common type of culvert used by TxDOT. The load rating methods presented herein have not been explicitly evaluated for other culvert materials such as plastic, steel, or aluminum. Precast concrete box culverts may be modeled using techniques similar to those outlined in this Guide; however, the capacity equations for precast concrete box culverts presented in AASHTO's *SSHB* and *MCEB* are slightly different from those outlined here for reinforced concrete box culverts. Round pipes, arch, and other culvert shapes behave very differently from box culverts and this Guide does not present tools and procedures to model these other culvert shapes.

B. FILL DEPTH

TxDOT's culvert designs model fill depths ranging from zero (direct traffic) to deep fill (in excess of 20 feet). Parametric studies indicate that the deeper the fill, the more soil-structure interaction influences culvert behavior. However, the field instrumentation program for research project 0-5849 only evaluated culverts having four feet or less of fill. While the procedures outlined in the Guide can be used to load rate deep fill culverts, it will be especially important to validate the soil parameters used for demand modeling.

C. BACKFILL DRAINAGE

Submerged culverts and culverts in undrained, saturated soils exist in various parts of the state, in particular, along the Texas Gulf Coast and in high-rainfall areas of East Texas. Research project 0-5849 did not explore or consider the effects of water on the structural component of culvert behavior.

D. SOIL PARAMETERS

The sample load rating and parametric studies performed for research project 0-5849 indicate that the inventory rating factor is sensitive to certain soil parameters and conditions, in particular, the soil modulus value used for Level 3 analyses. Both published literature and the field instrumentation test program for research project 0-5849 suggest that soil modulus values for a given soil may vary by as much as one to two orders of magnitude depending on overburden stress, drainage, the method used to determine modulus, and other factors. While the Guide does offer handbook values for the soil parameters, these values should be used with caution and engineering judgment.

E. ANALYTICAL MODEL

This Guide has given preference to analytical models and structural analysis software suitable for production load rating of culverts. This is as opposed to models/software primarily intended for culvert design and analysis, or for research. Structural analysis models and software which incorporate more sophisticated constitutive models for the culvert and the soil exist, and these may be used for load rating as per the discussion in Chapter VII on Level 4 culvert load rating analyses.

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Appendix A. Example Culvert Details

MC10-3: 3-span, 10’x7’ with 6’ fill

1. Overview

This appendix introduces an example culvert – designated MC10-3, 3-span, 10'x7' with 6' fill – which will be used to illustrate load rating calculations for Level 1, Level 2, and Level 3 analyses. More specifically, this appendix explains how to obtain the culvert dimensional and structural properties necessary for a load rating analysis.

Typically, load rating is performed as part of the culvert inspection process, so various kinds of design information might be available for the culvert structure. However, for this example, it is assumed that the only information available is the culvert plan sheet. TxDOT's culvert designs appear on plan sheets such as the one shown in Figure IX-1. This particular plan sheet includes designs for 25 different culverts, so details pertaining to the specific culvert in question (highlighted in yellow) must be identified. Rating variables are determined through "take-offs" from the culvert plan sheet.

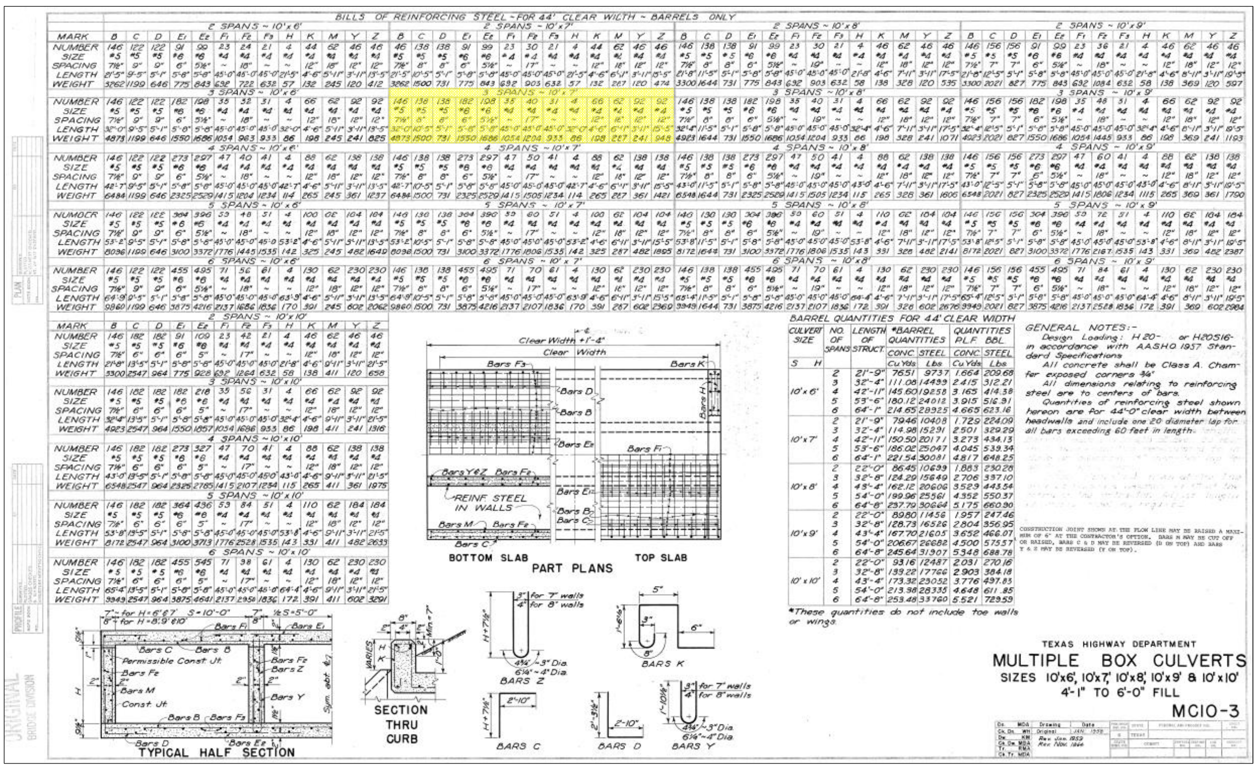

FIGURE IX.1. EXAMPLE CULVERT DESIGN SHEET.

2. General Culvert Information

Figure IX-2 shows the title block from the culvert plan sheet, identifying the design year and related design information. The design year is important because it is used to determine the steel and concrete grades as per AASHTO policy, as discussed in Section II.C of this Guide. Table IX-1 summarizes general information about the culvert.

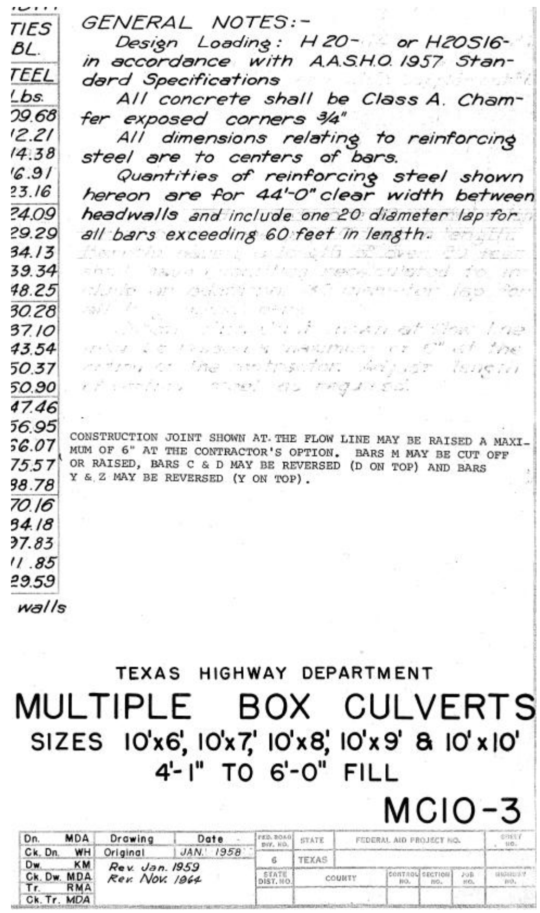

FIGURE IX.2. EXAMPLE CULVERT, TITLE BLOCK INFORMATION FROM PLAN SHEET.

TABLE IX-1. EXAMPLE CULVERT, GENERAL INFORMATION.

sheet #	MC10-3
culvert ID	MC10-3 3 10x7w6
year	1958
concrete class	A
steel grade	NA
installation type	NA
road width (ft)	44

3. Dimensions

Figure IX-3 enlarges the "typical half section" from the culvert plan sheet to facilitate determining the culvert dimensions. Section IV.C of this Guide discusses the dimension variables needed for culvert load rating. Table IX-2 summarizes the dimensional data for the example culvert. This information is used to create the culvert model, a cross-sectional sketch of which is shown in Figure IX-4, including identification of members, nodes, centerline dimensions, and most importantly, location of the culvert critical sections. This culvert does not contain haunches, so this affects designation of the culvert critical sections shown in Figure IX-4.

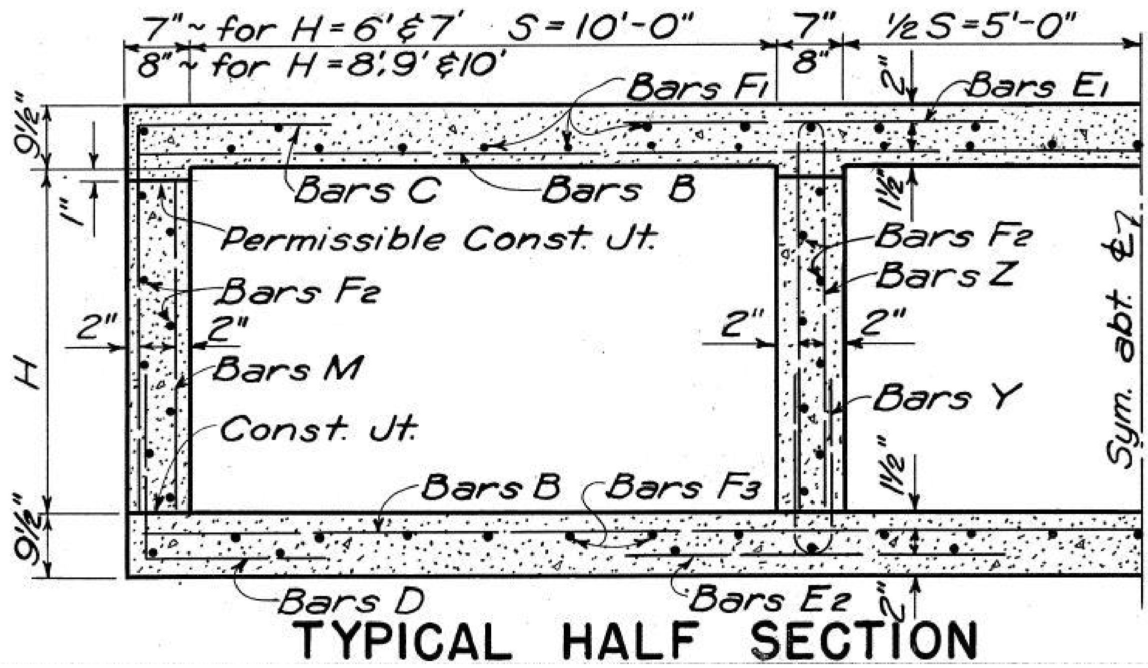

FIGURE IX.3. EXAMPLE CULVERT, DIMENSIONS FROM PLAN SHEET.

TABLE IX-2. EXAMPLE CULVERT, REQUIRED DIMENSIONS.

Dimension	*Abbr.*	*Value*	*Units*
number of spans	N	3.0	
cover soil depth	D	6.0	ft
clear span	S	10.0	ft
clear height	H	7.0	ft
exterior wall thickness	T_{EW}	7.0	in.
interior wall thickness	T_{IW}	7.0	in.
top slab thickness	T_T	9.5	in.
bottom slab thickness	T_B	9.5	in.
top haunch dimension	F_T	0.0	in.
bottom haunch dimension	F_B	0.0	in.

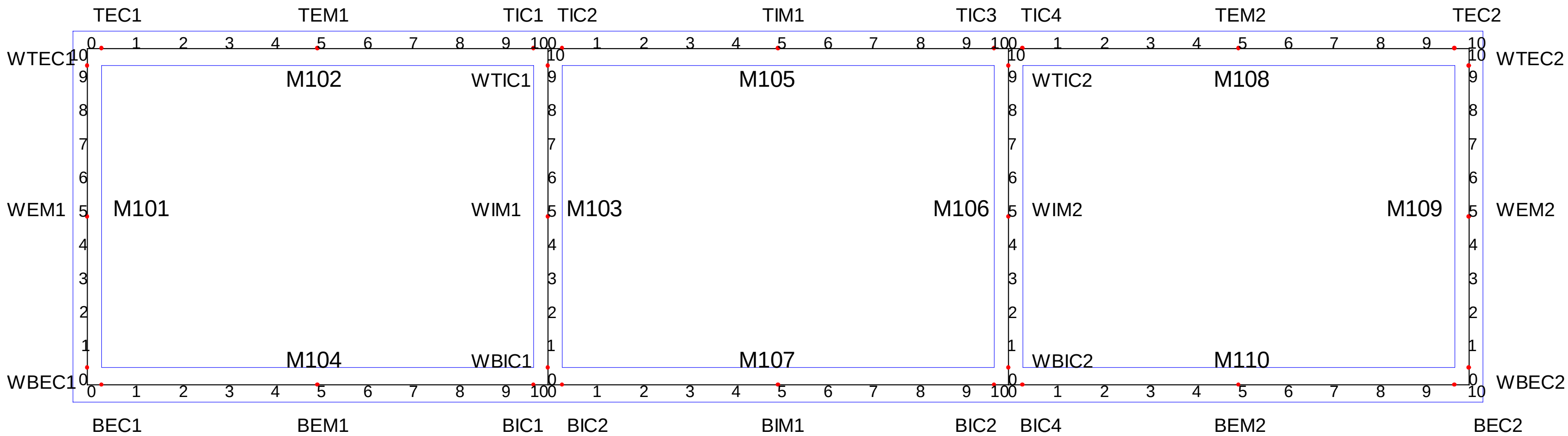

FIGURE IX.4. CROSS-SECTION SKETCH OF THE EXAMPLE CULVERT FOR THE ANALYTICAL MODEL.

Commentary: The 10th point numbering scheme applies only for the CULV-5 output. The sign convention used throughout the Guide is consistent with this layout in CULV-5. However, in order to maintain the sign convention in the higher level models (e.g. RISA-2D), the members must be oriented as per Figure VI-15.

4. Material (Structural) Properties

Design information including the concrete strength, reinforcing steel grade and other data must be obtained from the plan sheet and other construction documents as available. When the design year is known, structural material properties for the concrete and steel can be established based on AASHTO policy as per Section II.C of this Guide. Table IX-3 summarizes the structural material properties for the example culvert.

TABLE IX-3. EXAMPLE CULVERT, STRUCTURAL MATERIAL PROPERTIES.

Material Properties	*Abbr.*	*Value*	*Units*
comp. strength of concrete	f'_c	3000	psi
yield stress of steel	F_y	36000	psi
modulus of elasticity for steel	E_s	29000	ksi
modulus of elasticity for conc.	E_c	3122	ksi
modular ratio	n	9	
Whitney's stress block	β	0.85	

5. Soil Parameters

Site-specific details for the soils around this culvert are not available. Therefore, “medium” soil properties will be assumed for the analysis, as defined in Section IV.E of this Guide. Table IX-4 summarizes the soil properties.

TABLE IX-4. EXAMPLE CULVERT SOIL PARAMETERS.

Type	*Abbr.*	*Value*	*Units*
soil unit weight	γ	120	pcf
modulus of subgrade reaction	k	150	pci
modulus of elasticity	E	20000	psi
Poisson's ratio	ν	0.3	

6. Reinforcing Steel Schedule

The culvert plan sheet details must be carefully reviewed to determine the size and spacing of reinforcing bars for the tensile and compressive zones in the culvert. See Figure IX-3, Figure IX-5 and Figure IX-6. The area of steel can then be determined using Equation IX-1.

EQUATION IX-1. AREA OF REINFORCING STEEL.

$$A_s = \sum \left(\frac{\Phi_R}{8}\right)^2 \frac{\pi\, b}{4\, s}$$

where: A_s = area of the tension reinforcement (in.^{2})
$\Phi_R =$ the reinforcing bar size (1/8")
s = the reinforcing spacing (in.)
b = width of the compression face member (typically 12 inches)

REINFORCING STEEL ~ FOR 44' CLEAR WIDTH ~ BAR													
MARK	B	C	D	E1	E2	F1	F2	F3	H	K	M	Y	Z
	3 SPANS ~ 10' x 7'												
NUMBER	146	138	138	182	198	35	40	31	4	66	62	92	92
SIZE	#5	#5	#5	#6	#6	#4	#4	#4	#4	#4	#4	#4	#4
SPACING	7½"	8"	8"	6"	5½"	~	17"	~	~	12"	18"	12"	12"
LENGTH	32'-0"	10'-5"	5'-1"	5'-8"	5'-8"	45'-0"	45'-0"	45'-0"	32'-0"	4'-6"	6'-11"	3'-11"	15'-5"
WEIGHT	4873	1500	731	1550	1686	1054	1204	933	86	198	287	241	948

FIGURE IX.5. EXAMPLE CULVERT, REINFORCING SCHEDULE FROM PLAN SHEET.

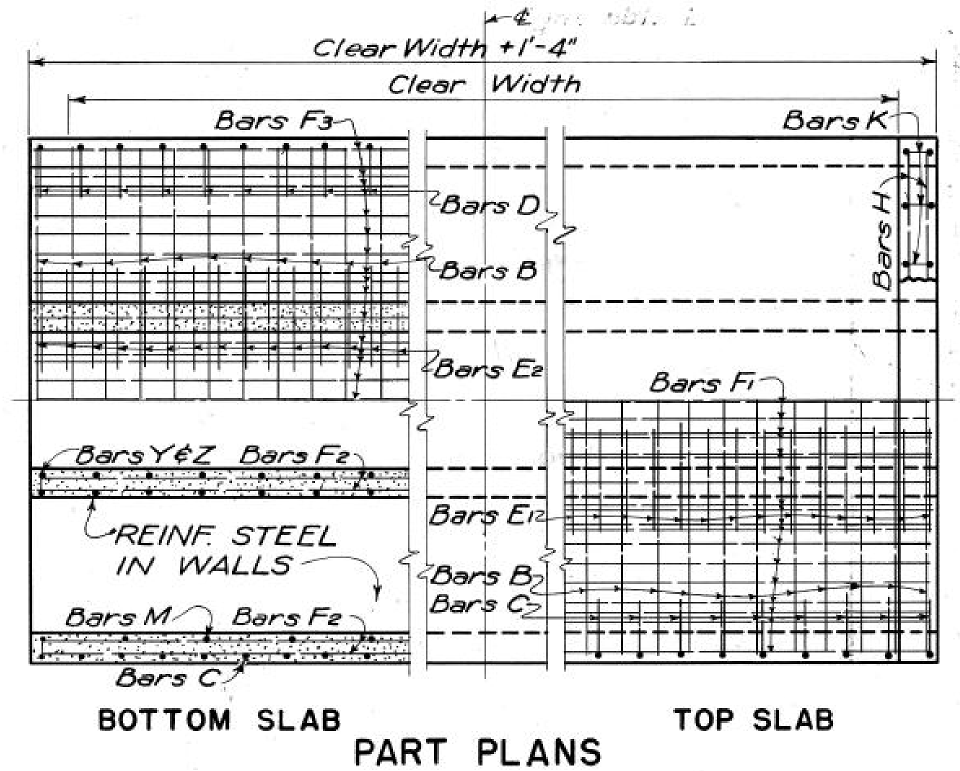

FIGURE IX.6. EXAMPLE CULVERT, DESIGNATION OF REINFORCING STEEL AS PER PLAN SHEET.

Ultimately, the goal of the reinforcing steel quantity take-off is to establish the area of reinforcing steel at each critical section, for both the tension and compression faces, for each member of the culvert. This means that it is necessary to define the area of steel at each critical section identified on Figure IX-4. Table IX-5 summarizes the reinforcing steel parameters for each critical section of the example culvert.

It should be noted that, due to symmetry, it is necessary to specify only half the culvert. Since the example culvert is a three span culvert shown in Figure IX-4, Member 106 and Member 103 are the same and will yield identical results. The same is true for members 101 and 109, 102 and 108, and 104 and 110. Therefore, it is necessary to determine the reinforcing steel only for Members 101, 102, 103, 104, 105 and 107 (1, 2, 3, 4, 5 and 7 in CULV-5 output).

Based on this fact, to save work, some load raters will note symmetry conditions up front and only specify critical sections for the unique structural members. This Guide follows this practice.

TABLE IX-5. EXAMPLE CULVERT, REINFORCING STEEL SCHEDULE.

			Inside Layer Reinforcing Schedule					Outside Layer Reinforcing Schedule				
Member	*Sections*		*Mark*	*Bar Dia. (1/8")*	*Spacing (in.)*	*A_s (in.^{2}/ft)*	*d (in.)*	*Mark*	*Bar Dia. (1/8")*	*Spacing (in.)*	*A_s (in.^{2}/ft)*	*d (in.)*
M101	1	WBEC	M	4	18	0.0000	5	D	5	8	0.4602	5
	2	WEM				0.1309		C	5	8	0.4602	5
	3	WTEC				0.0000		C	5	8	0.4602	5
M102	4	TEC	B	5	7.5	0.4909	8	C	5	8	0.4602	7.55
	5	TEM	B	5	7.5	0.4909	8				0.0000	7.5
	6	TIC1	B	5	7.5	0.4909	8	E1	6	6	0.8836	7.5
M103	7	WBIC1	Y	4	12	0.1963	5	Y	4	12	0.1963	5
	8	WIM1	Z	4	12	0.1963	5	Z	4	12	0.1963	5
	9	WTIC1	Z	4	12	0.1963	5	Z	4	12	0.1963	5
M104	10	BEC	B	5	7.5	0.4909	8	D	5	8	0.4602	7.5
	11	BEM	B	5	7.5	0.4909	8				0.0000	7.5
	12	BIC1	B	5	7.5	0.4909	8	E2	6	5.5	0.9639	7.5
M105	13	TIC2	B	5	7.5	0.4909	8	E1	6	6	0.8836	7.5
	14	TIM1	B	5	7.5	0.4909	8				0.0000	7.5
	15	TIC3	B	5	7.5	0.4909	8	E1	6	6	0.8836	7.5
M107	19	BIC2	B	5	7.5	0.4909	8	E2	6	5.5	0.9639	7.5
	20	BIM1	B	5	7.5	0.4909	8				0.0000	7.5
	21	BIC3	B	5	7.5	0.4909	8	E2	6	5.5	0.9639	7.5

7. Culvert Installation Method

The culvert installation method is unknown. However, since the culvert was designed and installed in the late 1950s, it is reasonable to assume that residual stresses which might have existed for an embankment or trench installation will have dissipated. Thus it will be acceptable to ignore how the culvert was installed.

Appendix B. EXAMPLE CULVERT CAPACITY CALCULATIONS

When the model dimensions, material properties, soil properties, and reinforcing steel parameters are defined, the moment, shear and thrust capacities for each critical section of the culvert must be determined.

Because the reinforcing steel layout for culverts is not fully symmetrical, capacity is a directional property. That is, the culvert slab slices under analysis will have different capacities depending on the direction of bending. The sign convention used throughout this Guide is that bending which produces tension on the inside face of the culvert is positive, while bending which produces tension on the outside face of the culvert is negative. The sign convention is defined in terms of bending, with shear and axial thrust following on. There is no separate, independent sign convention for shear or for axial thrust.

Ultimately then, the capacity must be determined for each critical section, for each type of stress (moment, shear and thrust), for both positive and negative bending. Table IX-6 summarizes capacity calculation results for each critical section for the positive bending case. The capacity values are obtained using the step-by-step procedure presented in Chapter V of this Guide. The critical sections are defined in Figure IX-4 of Appendix A, and the areas of reinforcing steel associated with each critical section are shown in Table IX-5.

TABLE IX-6. EXAMPLE CULVERT SECTION PROPERTIES.

			Tensile Face Inside (Positive Bending)						
			Capacity Step 1.	**Capacity Step 2.**	**Capacity Step 3.**	**Capacity Step 4.**	**Capacity Step 5.**	**Capacity Step 6.**	**Capacity Step 7.**
Member	*Sections*		*c (in.)*	*F'_s (psi)*	*F'_b (psi)*	*ρ_b*	*ρ check*	*φM_n (k-ft/ft)*	*φV_n (klf)*
M101	1	WBEC	1.16	0	0	NA	NA	2.4	8.4
	2	WEM	1.22	0	0	0.0426	OK	1.7	8.4
	3	WTEC	1.16	0	0	NA	NA	2.4	8.4
M102	4	TEC	1.39	0	0	0.0426	OK	10.2	13.4
	5	TEM	0.68	0	0	0.0426	OK	10.2	13.4
	6	TIC1	1.57	0	0	0.0426	OK	10.2	13.4
M103	7	WBIC1	0.98	0	0	0.0426	OK	2.6	8.4
	8	WIM1	0.98	0	0	0.0426	OK	2.6	8.4
	9	WTIC1	0.98	0	0	0.0426	OK	2.6	8.4
M104	10	BEC	1.39	0	0	0.0426	OK	10.2	13.4
	11	BEM	0.68	0	0	0.0426	OK	10.2	13.4
	12	BIC1	1.59	0	0	0.0426	OK	10.2	13.4
M105	13	TIC2	1.57	0	0	0.0426	OK	10.2	13.4
	14	TIM1	0.68	0	0	0.0426	OK	10.2	13.4
	15	TIC3	1.57	0	0	0.0426	OK	10.2	13.4
M107	19	BIC2	1.59	0	0	0.0426	OK	10.2	13.4
	20	BIM1	0.68	0	0	0.0426	OK	10.2	13.4
	21	BIC3	1.59	0	0	0.0426	OK	10.2	13.4

Table VIII-7 summarizes capacity calculation results for each critical section for the negative bending case. Again, the capacity values are obtained using the step-by-step procedure presented in Chapter V of this Guide. Note that the thrust capacity need only be determined once. Because thrust is always negative (compression), it is only presented in Table IX-7.

TABLE IX-7. EXAMPLE CULVERT SECTION PROPERTIES CONT.

			Tensile Face Outside (Negative Bending)							Thrust
			Capacity Step 1.	**Capacity Step 8.**	**Capacity Step 3.**	**Capacity Step 4.**	**Capacity Step 5.**	**Capacity Step 6.**	**Capacity Step 7.**	**Capacity Step 8.**
Member	*Sections*		*c (in.)*	*F's (psi)*	*φPn (klf)*	*ρb*	*ρ check*	*φMn (k-ft/ft)*	*φVn (klf)*	*φPn (klf)*
M101	1	WBEC	0.64	0	0	0.0426	OK	-5.9	-8.4	-206.6
	2	WEM	1.05	0	0	0.0426	OK	-5.9	-8.4	-210.6
	3	WTEC	0.64	0	0	0.0426	OK	-5.9	-8.4	-206.6
M102	4	TEC	1.16	0	0	0.0426	OK	-9.0	-12.6	-290.3
	5	TEM	0.96	0	0	0.0426	OK	-4.4	-12.6	-276.4
	6	TIC1	1.39	0	0	0.0426	OK	-16.7	-12.6	-303.0
M103	7	WBIC1	0.98	0	0	0.0426	OK	-2.6	-8.4	-204.6
	8	WIM1	0.98	0	0	0.0426	OK	-2.6	-8.4	-204.6
	9	WTIC1	0.98	0	0	0.0426	OK	-2.6	-8.4	-204.6
M104	10	BEC	1.16	0	0	0.0426	OK	-9.0	-12.6	-290.3
	11	BEM	0.96	0	0	0.0426	OK	-4.4	-12.6	-276.4
	12	BIC1	1.44	0	0	0.0426	OK	-18.0	-12.6	-305.4
M105	13	TIC2	1.39	0	0	0.0426	OK	-16.7	-12.6	-303.0
	14	TIM1	0.96	0	0	0.0426	OK	-4.4	-12.6	-276.4
	15	TIC3	1.39	0	0	0.0426	OK	-16.7	-12.6	-303.0
M107	19	BIC2	1.44	0	0	0.0426	OK	-18.0	-12.6	-305.4
	20	BIM1	0.96	0	0	0.0426	OK	-4.4	-12.6	-276.4
	21	BIC3	1.44	0	0	0.0426	OK	-18.0	-12.6	-305.4

It must be emphasized that the culvert load rating process is one component of the culvert inspection process, and the typical case is that a culvert which is being load-rated will have had a visual inspection. Section 6.5.4 of the *MCEB* specifically addresses the relationship between field inspection and the load rating and notes that "the condition and extent of deterioration of structural components of the bridge [culvert] should be considered in the computation of… capacity when force or moment is chosen for use in the basic rating equation." This means that any discrepancies from plan, or excessive distress such as thin sections, spalling, cracking, deflection, exposed reinforcing steel, and other items which may affect structural capacity, should be considered when establishing actual section capacities. For this example, no adjustments to plan values have been made.

Appendix C. LEVEL 1: CULV-5 EXAMPLE PROBLEM

CULV5 Step 1. Using data obtained for the example culvert identified in Appendix A, write the CULV-5 input file in a basic text editor (eg, Notepad) according to the form in Figure VI-8. Alternatively, the load rater may use the "Culv5 Input" program developed by TechMRT and hosted on the TxDOT Bridge Division website to create the input file. The input file for this example is as follows:

```
                   TAW     6-25-2008
Culvert Rating Guide VIII.C CULV-5 Example
PROB MC10-3 3 10x7w6
SPECE12      1 0   120.
CULV 310.007.006.00            09.5 09.5 07.0 07.0 2.060.30.   0  0  0         1
```

CULV5 Step 2. Run the CULV-5 program using the input file created in step one. The following is a summary of the CULV-5 output on seven pages. This includes:

- **CULVERT, SPEC DATA**. Page 1 presents a restatement of input values. The load rater should verify these are correct.
- **SUMMARY OF MAXIMUM FACTORED MOMENTS, SHEARS AND AXIAL FORCES.** This information, presented on output pages 2 and 3, is *not* used for culvert load rating. However, the load rater should note that because the culvert structure is symmetrical about its centerline, even though the culvert is actually modeled using 10 members, results are only presented for 6 members. These are the 6 members representing the middle and left side of the model. Demands for the four members forming the right portion of the model (members 6, 8, 9, and 10) are the same as for the three members forming the left side of the model (members 1, 2, 3 and 4) and are thus omitted from further consideration.
- **SUMMARY OF INDIVIDUAL UNFACTORED MOMENTS AND SHEARS.** This section of output (pages 4 and 5) contains demands for moments and shears at 10th points for each member. This will be used in subsequent steps for load rating. Shaded rows represent critical sections. These are either mid-span sections or 10th-point nodes for interpolation of corner critical sections.
- **SUMMARY OF INDIVIDUAL UNFACTORED AXIAL FORCES.** This section of output (pages 6 and 7) contains demands for axial thrusts at 10th points for each member. This will be used in subsequent steps for load rating. Shaded rows represent critical sections. These are either mid-span sections or 10th-point nodes for interpolation of corner critical sections.

```
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Culvert Rating Guide VIII.C CULV-5 Example

PROB MC10-3 3 10x7w6


-- CULVERT, SPEC DATA (* DENOTES DEFAULT) --

SERVICE LOAD DESIGN                         ANALYSIS PROBLEM
LIVE LOADING               =   HS20     OMIT LIVE LOAD AS PER SPECS =     NO
GAMMA FACTOR               =   1.00 *   AXLE WT FOR OVERLOAD (LB)   =    .00 *
BETA FACTOR FOR DL         =   1.00 *   PRINT 10TH PT MOMTS & SHRS  =    YES *
BETA FACTOR FOR LL         =   1.00 *   PRINT INFLUENCE LINES       =
SOIL WEIGHT (PCF)          = 120.00     CONCRETE WEIGHT (PCF)       = 150.00 *
IMPACT FACTOR              =    .00
NUMBER OF BARRELS          =      3     FLOOR SPPORT                =   SNGL
CLEAR SPAN (FT)            =  10.00     CLEAR HEIGHT (FT)           =   7.00
TOP SLAB THICKNESS (IN)    =   9.50     BOTTOM SLAB THICKNESS (IN)  =   9.50
EXT WALL THICKNESS (IN)    =   7.00     INT WALL THICKNESS (IN)     =   7.00
DEPTH OF FILL (FT)         =   6.00     LIVE LOAD SURCHARGE (FT)    =   2.00
MAX LAT SOIL PRESSURE (PCF)=  60.00     MIN LAT SOIL PRESSURE (PCF) =  30.00
TOP HAUNCH WIDTH (IN)      =    .00     BOTTOM HAUNCH WIDTH (IN)    =    .00


-- EXTRA DEAD LOAD AND SPECIAL LIVE LOAD --

NONE
```

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-- **SUMMARY OF MAXIMUM FACTORED MOMENTS, SHEARS AND AXIAL FORCES** --

BM	10TH	--- MOMENTS (KFT) ----			----- SHEARS (K) ------			-- @AXIAL FORCES (K) --		
NO	PT	LDNG #1	LDNG #2	LDNG #3	LDNG #1	LDNG #2	LDNG #3	LDNG #1	LDNG #2	LDNG #3
1 -	0	-3.256	-5.744	-5.098	1.436	3.217	3.247	-3.769	-5.282	-4.056
1 -	1	-2.288	-3.509	-2.859	1.113	2.478	2.508	-3.769	-5.282	-4.056
1 -	2	-1.564	-1.837	-1.180	.809	1.776	1.806	-3.760	-5.282	-4.056
1 -	3	-1.067	-.697	-.035	.523	1.110	1.140	-3.760	-5.282	-4.056
1 -	4	-.786	-.061	.606	.255	.481	.511	-3.760	-5.282	-4.056
1 -	5	-.707	.098	.771	.005	-.112	-.082	-3.760	-5.282	-4.056
1 -	6	-.815	-.191	.488	-.226	-.668	-.638	-3.760	-5.282	-4.056
1 -	7	-1.096	-.898	-.214	-.439	-1.188	-1.158	-3.760	-5.282	-4.056
1 -	8	-1.537	-1.997	-1.307	-.634	-1.672	-1.642	-3.760	-5.282	-4.056
1 -	9	-2.122	-3.459	-2.763	-.811	-2.119	-2.089	-3.760	-5.282	-4.056
1 -	10	-2.819	-5.260	-4.553	-.970	-2.530	-2.500	-3.762	-5.276	-4.056
2 -	0	-2.819	-5.262	-4.553	5.094	5.501	4.056	-.983	-2.517	-2.500
2 -	1	1.130	-.735	-.730	4.006	4.345	3.168	-.999	-2.505	-2.500
2 -	2	4.671	2.046	2.153	2.850	3.189	2.280	-1.003	-2.504	-2.500
2 -	3	7.004	3.880	4.096	1.708	2.101	1.393	-1.003	-2.500	-2.500
2 -	4	8.114	4.770	5.100	.642	1.039	.505	-1.003	-2.500	-2.500
2 -	5	8.000	4.720	5.165	-.383	-.768	-.383	-1.003	-2.500	-2.500
2 -	6	6.663	3.731	4.290	-1.783	-1.805	-1.270	-1.003	-2.500	-2.500
2 -	7	4.139	1.803	2.475	-2.849	-2.867	-2.158	-1.003	-2.500	-2.500
2 -	8	.469	-1.110	-.278	-3.991	-3.954	-3.046	-1.003	-2.511	-2.500
2 -	9	-4.170	-5.468	-3.972	-5.147	-5.110	-3.934	-.996	-2.523	-2.500
2 -	10	-9.002	-11.193	-8.604	-6.236	-6.266	-4.821	-.996	-2.500	-2.500
3 -	0	.974	.026	.342	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	1	.967	.011	.333	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	2	.961	-.004	.324	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	3	.954	-.019	.315	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	4	.947	-.034	.306	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	5	.941	-.049	.297	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	6	.934	-.064	.288	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	7	.927	-.079	.279	.025	-.047	-.012	-10.731	-10.550	-9.260
3 -	8	.924	-.093	.270	.025	-.047	-.012	-10.732	-10.550	-9.260
3 -	9	.920	-.119	.261	.025	-.047	-.012	-10.732	-10.557	-9.260
3 -	10	.918	-.144	.252	.025	-.047	-.012	-10.758	-10.769	-9.260
4 -	0	-3.256	-5.744	-5.098	5.451	5.934	4.473	-1.414	-3.254	-3.247
4 -	1	1.076	-.882	-.882	4.210	4.694	3.494	-1.416	-3.247	-3.247
4 -	2	4.875	2.221	2.298	2.972	3.454	2.516	-1.416	-3.248	-3.247
4 -	3	7.362	4.256	4.443	1.735	2.213	1.537	-1.416	-3.248	-3.247
4 -	4	8.542	5.253	5.552	.513	.976	.559	-1.417	-3.248	-3.247
4 -	5	8.414	5.210	5.626	-.622	-.604	-.420	-1.417	-3.248	-3.247

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PROB MC10-3 3 10x7w6

-- SUMMARY OF MAXIMUM FACTORED MOMENTS, SHEARS AND AXIAL FORCES -- (CONT'D)

BM	10TH	--- MOMENTS (KFT) ----			----- SHEARS (K) ------			-- @AXIAL FORCES (K) --		
NO	PT	LDNG #1	LDNG #2	LDNG #3	LDNG #1	LDNG #2	LDNG #3	LDNG #1	LDNG #2	LDNG #3
4 -	6	6.978	4.129	4.663	-1.757	-1.816	-1.399	-1.417	-3.248	-3.247
4 -	7	4.277	2.009	2.665	-2.979	-3.053	-2.377	-1.417	-3.248	-3.247
4 -	8	.334	-1.150	-.368	-4.216	-4.293	-3.356	-1.417	-3.248	-3.247
4 -	9	-4.647	-5.692	-4.437	-5.454	-5.534	-4.334	-1.426	-3.264	-3.247
4 -	10	-9.977	-11.857	-9.542	-6.695	-6.774	-5.313	-1.410	-3.249	-3.247
5 -	0	-8.614	-10.872	-8.353	5.928	5.686	4.438	-1.010	-2.508	-2.511
5 -	1	-4.386	-5.580	-4.125	4.772	4.581	3.551	-1.010	-2.512	-2.511
5 -	2	-.485	-1.908	-.837	3.616	3.482	2.663	-1.029	-2.511	-2.511
5 -	3	2.465	.582	1.512	2.482	2.395	1.775	-1.018	-2.512	-2.511
5 -	4	4.231	1.993	2.921	1.400	1.319	.888	-1.018	-2.512	-2.511
5 -	5	4.891	2.465	3.391	.339	-.273	.000	-1.000	-2.512	-2.511
5 -	6	4.231	1.993	2.921	-1.400	-1.319	-.888	-1.018	-2.512	-2.511
5 -	7	2.465	.582	1.512	-2.482	-2.395	-1.775	-1.018	-2.512	-2.511
5 -	8	-.485	-1.908	-.837	-3.616	-3.482	-2.663	-1.029	-2.511	-2.511
5 -	9	-4.386	-5.580	-4.125	-4.772	-4.581	-3.551	-1.010	-2.512	-2.511
5 -	10	-8.614	-10.872	-8.353	-5.928	-5.686	-4.438	-1.010	-2.508	-2.511
7 -	0	-9.478	-11.481	-9.200	6.276	6.177	4.893	-1.396	-3.253	-3.235
7 -	1	-4.818	-5.908	-4.540	5.032	4.963	3.914	-1.396	-3.230	-3.235
7 -	2	-.585	-1.974	-.915	3.788	3.749	2.936	-1.414	-3.235	-3.235
7 -	3	2.570	.781	1.674	2.545	2.535	1.957	-1.404	-3.236	-3.235
7 -	4	4.476	2.344	3.228	1.320	1.321	.979	-1.385	-3.236	-3.235
7 -	5	5.195	2.869	3.746	.178	-.110	.000	-1.385	-3.236	-3.235
7 -	6	4.476	2.344	3.228	-1.320	-1.321	-.979	-1.385	-3.236	-3.235
7 -	7	2.570	.781	1.674	-2.545	-2.535	-1.957	-1.404	-3.236	-3.235
7 -	8	-.585	-1.974	-.915	-3.788	-3.749	-2.936	-1.414	-3.235	-3.235
7 -	9	-4.818	-5.908	-4.540	-5.032	-4.963	-3.914	-1.396	-3.230	-3.235
7 -	10	-9.478	-11.481	-9.200	-6.276	-6.177	-4.893	-1.396	-3.253	-3.235

NOTE: LDNG #1 = 100%(VERT DL) + 100%(+VERT LL) + 50%(LAT DL)
LDNG #2 = 100%(VERT DL) + 100%(-VERT LL) + 100%(LAT DL) + 100%(LAT LL)
LDNG #3 = 100%(VERT DL) + 100%(LAT DL) + 100%(LAT LL)
@ = AXIAL FORCES CORRESPONDING WITH MAXIMUM MOMENTS AT SAME SECTION

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-- **SUMMARY OF INDIVIDUAL UNFACTORED MOMENTS AND SHEARS** --

BM	10TH	--------- MOMENTS (KFT)	---------				----------- SHEARS (K)	-----------			
NO	PT	VDL	LDL	+VLL	-VLL	LLL	VDL	LDL	+VLL	-VLL	LLL
1 -	0	-2.150	-2.486	.137	-.645	-.462	.040	2.739	.026	-.030	.468
1 -	1	-2.119	-.606	.134	-.651	-.134	.040	2.094	.026	-.030	.374
1 -	2	-2.087	.786	.131	-.657	.122	.040	1.485	.026	-.030	.281
1 -	3	-2.056	1.717	.130	-.662	.304	.040	.913	.026	-.030	.187
1 -	4	-2.024	2.217	.129	-.668	.413	.040	.377	.026	-.030	.094
1 -	5	-1.993	2.314	.129	-.673	.450	.040	-.122	.026	-.030	.000
1 -	6	-1.961	2.036	.128	-.679	.413	.040	-.585	.026	-.030	-.093
1 -	7	-1.930	1.411	.127	-.685	.304	.040	-1.012	.026	-.030	-.187
1 -	8	-1.898	.469	.127	-.690	.122	.040	-1.402	.026	-.030	-.280
1 -	9	-1.866	-.764	.126	-.696	-.133	.040	-1.756	.026	-.030	-.374
1 -	10	-1.835	-2.258	.145	-.707	-.460	.040	-2.073	.026	-.030	-.467
2 -	0	-1.835	-2.258	.145	-.709	-.460	3.735	.267	1.227	1.445	.054
2 -	1	1.648	-1.976	.470	-.005	-.403	2.847	.267	1.025	1.177	.054
2 -	2	4.191	-1.693	1.326	-.107	-.345	1.959	.267	.757	.908	.054
2 -	3	5.795	-1.411	1.915	-.217	-.288	1.072	.267	.503	.709	.054
2 -	4	6.459	-1.129	2.219	-.331	-.230	.184	.267	.325	.534	.054
2 -	5	6.184	-.847	2.240	-.445	-.172	-.704	.267	.188	-.385	.054
2 -	6	4.970	-.565	1.976	-.559	-.115	-1.591	.267	-.325	-.534	.054
2 -	7	2.816	-.283	1.465	-.673	-.057	-2.479	.267	-.503	-.709	.054
2 -	8	-.278	-.001	.748	-.831	.001	-3.367	.267	-.757	-.908	.054
2 -	9	-4.311	.281	.000	-1.497	.058	-4.254	.267	-1.025	-1.177	.054
2 -	10	-9.283	.563	.000	-2.589	.116	-5.142	.267	-1.227	-1.445	.054
3 -	0	.614	-.231	.475	-.316	-.041	-.017	.005	.040	-.035	.000
3 -	1	.601	-.227	.479	-.322	-.041	-.017	.005	.040	-.035	.000
3 -	2	.588	-.223	.484	-.328	-.041	-.017	.005	.040	-.035	.000
3 -	3	.575	-.219	.488	-.334	-.041	-.017	.005	.040	-.035	.000
3 -	4	.562	-.215	.493	-.340	-.041	-.017	.005	.040	-.035	.000
3 -	5	.549	-.211	.497	-.346	-.041	-.017	.005	.040	-.035	.000
3 -	6	.536	-.207	.501	-.352	-.041	-.017	.005	.040	-.035	.000
3 -	7	.523	-.203	.506	-.358	-.041	-.017	.005	.040	-.035	.000
3 -	8	.510	-.199	.513	-.363	-.041	-.017	.005	.040	-.035	.000
3 -	9	.497	-.195	.521	-.380	-.041	-.017	.005	.040	-.035	.000
3 -	10	.484	-.191	.530	-.396	-.041	-.017	.005	.040	-.035	.000
4 -	0	-2.150	-2.486	.137	-.645	-.462	4.123	.295	1.180	1.461	.055
4 -	1	1.696	-2.174	.467	.000	-.404	3.145	.295	.918	1.199	.055
4 -	2	4.506	-1.862	1.300	-.077	-.346	2.166	.295	.658	.938	.055
4 -	3	6.281	-1.550	1.857	-.187	-.288	1.188	.295	.400	.676	.055
4 -	4	7.020	-1.237	2.141	-.299	-.230	.209	.295	.156	.418	.055
4 -	5	6.723	-.925	2.154	-.415	-.172	-.770	.295	.000	-.184	.055

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-- SUMMARY OF INDIVIDUAL UNFACTORED MOMENTS AND SHEARS -- (CONT'D)

BM 10TH  --------- MOMENTS (KFT) ---------  ----------- SHEARS (K) -----------
NO  PT    VDL    LDL   +VLL   -VLL    LLL     VDL    LDL   +VLL   -VLL    LLL

4 -  6   5.391  -.613  1.894  -.534  -.114  -1.748   .295  -.156  -.418   .055
4 -  7   3.023  -.301  1.405  -.656  -.056  -2.727   .295  -.400  -.676   .055
4 -  8   -.381   .011   .710  -.782   .001  -3.705   .295  -.658  -.938   .055
4 -  9  -4.820   .323   .011 -1.255   .059  -4.684   .295  -.918 -1.199   .055
4 - 10 -10.295   .636   .000 -2.315   .117  -5.662   .295 -1.180 -1.461   .055

5 -  0  -8.800   .372   .000 -2.520   .075   4.438   .000  1.490  1.247   .000
5 -  1  -4.572   .372   .000 -1.455   .075   3.551   .000  1.221  1.030   .000
5 -  2  -1.284   .372   .613 -1.072   .075   2.663   .000   .953   .819   .000
5 -  3   1.065   .372  1.214  -.930   .075   1.775   .000   .706   .619   .000
5 -  4   2.474   .372  1.571  -.928   .075    .888   .000   .512   .431   .000
5 -  5   2.944   .372  1.761  -.925   .075    .000   .000   .339  -.273   .000
5 -  6   2.474   .372  1.571  -.928   .075   -.888   .000  -.512  -.431   .000
5 -  7   1.065   .372  1.214  -.930   .075  -1.775   .000  -.706  -.619   .000
5 -  8  -1.284   .372   .613 -1.072   .075  -2.663   .000  -.953  -.819   .000
5 -  9  -4.572   .372   .000 -1.455   .075  -3.551   .000 -1.221 -1.030   .000
5 - 10  -8.800   .372   .000 -2.520   .075  -4.438   .000 -1.490 -1.247   .000

7 -  0  -9.681   .405   .000 -2.281   .076   4.893   .000  1.383  1.284   .000
7 -  1  -5.020   .405   .000 -1.369   .076   3.914   .000  1.118  1.048   .000
7 -  2  -1.395   .405   .608 -1.059   .076   2.936   .000   .853   .813   .000
7 -  3   1.194   .405  1.174  -.894   .076   1.957   .000   .588   .578   .000
7 -  4   2.747   .405  1.526  -.883   .076    .979   .000   .342   .342   .000
7 -  5   3.265   .405  1.727  -.876   .076    .000   .000   .178  -.110   .000
7 -  6   2.747   .405  1.526  -.883   .076   -.979   .000  -.342  -.342   .000
7 -  7   1.194   .405  1.174  -.894   .076  -1.957   .000  -.588  -.578   .000
7 -  8  -1.395   .405   .608 -1.059   .076  -2.936   .000  -.853  -.813   .000
7 -  9  -5.020   .405   .000 -1.369   .076  -3.914   .000 -1.118 -1.048   .000
7 - 10  -9.681   .405   .000 -2.281   .076  -4.893   .000 -1.383 -1.284   .000
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-- SUMMARY OF INDIVIDUAL UNFACTORED AXIAL FORCES --

BM NO	10TH PT	VDL	LDL	+VLL	-VLL	LLL
1 -	0	-3.735	-.267	.099	-1.227	-.054
1 -	1	-3.735	-.267	.099	-1.227	-.054
1 -	2	-3.735	-.267	.108	-1.227	-.054
1 -	3	-3.735	-.267	.108	-1.227	-.054
1 -	4	-3.735	-.267	.108	-1.227	-.054
1 -	5	-3.735	-.267	.108	-1.227	-.054
1 -	6	-3.735	-.267	.108	-1.227	-.054
1 -	7	-3.735	-.267	.108	-1.227	-.054
1 -	8	-3.735	-.267	.108	-1.227	-.054
1 -	9	-3.735	-.267	.108	-1.227	-.054
1 -	10	-3.735	-.267	.105	-1.220	-.054
2 -	0	.041	-2.073	.013	-.018	-.467
2 -	1	.041	-2.073	-.003	-.005	-.467
2 -	2	.041	-2.073	-.007	-.004	-.467
2 -	3	.041	-2.073	-.007	-.001	-.467
2 -	4	.041	-2.073	-.007	-.001	-.467
2 -	5	.041	-2.073	-.007	-.001	-.467
2 -	6	.041	-2.073	-.007	-.001	-.467
2 -	7	.041	-2.073	-.007	-.001	-.467
2 -	8	.041	-2.073	-.007	-.011	-.467
2 -	9	.041	-2.073	.000	-.023	-.467
2 -	10	.041	-2.073	.000	-.001	-.467
3 -	0	-9.581	.267	-1.284	-1.290	.054
3 -	1	-9.581	.267	-1.284	-1.290	.054
3 -	2	-9.581	.267	-1.284	-1.290	.054
3 -	3	-9.581	.267	-1.284	-1.290	.054
3 -	4	-9.581	.267	-1.284	-1.290	.054
3 -	5	-9.581	.267	-1.284	-1.290	.054
3 -	6	-9.581	.267	-1.284	-1.290	.054
3 -	7	-9.581	.267	-1.284	-1.290	.054
3 -	8	-9.581	.267	-1.285	-1.290	.054
3 -	9	-9.581	.267	-1.285	-1.297	.054
3 -	10	-9.581	.267	-1.310	-1.510	.054
4 -	0	-.040	-2.739	-.004	-.007	-.468
4 -	1	-.040	-2.739	-.006	.000	-.468
4 -	2	-.040	-2.739	-.006	-.001	-.468
4 -	3	-.040	-2.739	-.006	-.001	-.468
4 -	4	-.040	-2.739	-.007	-.001	-.468
4 -	5	-.040	-2.739	-.007	-.001	-.468

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-- SUMMARY OF INDIVIDUAL UNFACTORED AXIAL FORCES -- (CONT'D)

BM 10TH  -------- AXIAL FORCES (K) --------
NO  PT    VDL    LDL    +VLL   -VLL    LLL

4 -  6   -.040 -2.739  -.007  -.001  -.468
4 -  7   -.040 -2.739  -.007  -.001  -.468
4 -  8   -.040 -2.739  -.007  -.001  -.468
4 -  9   -.040 -2.739  -.017  -.017  -.468
4 - 10   -.040 -2.739   .000  -.003  -.468

5 -  0    .024 -2.068   .000   .003  -.467
5 -  1    .024 -2.068   .000  -.001  -.467
5 -  2    .024 -2.068  -.019   .000  -.467
5 -  3    .024 -2.068  -.008  -.001  -.467
5 -  4    .024 -2.068  -.008  -.001  -.467
5 -  5    .024 -2.068   .011  -.001  -.467
5 -  6    .024 -2.068  -.008  -.001  -.467
5 -  7    .024 -2.068  -.008  -.001  -.467
5 -  8    .024 -2.068  -.019   .000  -.467
5 -  9    .024 -2.068   .000  -.001  -.467
5 - 10    .024 -2.068   .000   .003  -.467

7 -  0   -.024 -2.744   .000  -.018  -.468
7 -  1   -.024 -2.744   .000   .005  -.468
7 -  2   -.024 -2.744  -.019   .000  -.468
7 -  3   -.024 -2.744  -.008  -.001  -.468
7 -  4   -.024 -2.744   .011  -.001  -.468
7 -  5   -.024 -2.744   .011  -.001  -.468
7 -  6   -.024 -2.744   .011  -.001  -.468
7 -  7   -.024 -2.744  -.008  -.001  -.468
7 -  8   -.024 -2.744  -.019   .000  -.468
7 -  9   -.024 -2.744   .000   .005  -.468
7 - 10   -.024 -2.744   .000  -.018  -.468


NOTE:  VDL = VERT DL                  LDL = LAT DL
      +VLL = MAX POSITIVE VERT LL   -VLL = MAX NEGATIVE VERT LL   LLL = LAT LL
             INCLUDING IMPACT FACT.        INCLUDING IMPACT FACT.
```

CULV5 Step 3. Interpretation of the Culv-5 output requires establishing both corner and mid-span critical sections. Using the **SUMMARY OF INDIVIDUAL UNFACTORED MOMENTS AND SHEARS**, and the location of the critical sections as per Figure IX-4, select the 10^{th} points needed to set up the linear interpolation associated with moment, shear and axial thrust demands for the corner critical sections.

Calculation of the demand loads, by interpolation, requires a clear understanding of the overall sign convention and the way in which CULV-5 lays out the members and 10th points. Table IX-8 illustrates how to calculate demand moments for the corner critical sections. This same approach would be used for shear and axial thrust values.

Recall that mid-span demands are modeled as being located at mid-span. This means that the mid-span demands occur at node 5.

TABLE IX-8. CULV-5 CRITICAL SECTIONS FOR DEMANDS.

Member	*Sections*		*10th Points*		*Interpolation*
M101	1	WBEC	0	1	$X_{critical} = X_0 + \frac{10 * (X_1 - X_0)}{S_M}(L_{WBEC})$
	2	WEM	5		$X_{critical} = X_5$
	3	WTEC	9	10	$X_{critical} = X_{10} + \frac{10 * (X_9 - X_{10})}{S_M}(L_{WTEC})$
M102	4	TEC	0	1	$X_{critical} = X_0 + \frac{10 * (X_1 - X_0)}{S_M}(L_{TEC})$
	5	TEM	5		$X_{critical} = X_5$
	6	TIC1	9	10	$X_{critical} = X_{10} + \frac{10 * (X_9 - X_{10})}{S_M}(L_{TIC})$
M103	7	WBIC1	0	1	$X_{critical} = X_0 + \frac{10 * (X_1 - X_0)}{S_M}(L_{WBIC})$
	8	WIM1	5		$X_{critical} = X_5$
	9	WTIC1	9	10	$X_{critical} = X_{10} + \frac{10 * (X_9 - X_{10})}{S_M}(L_{WTIC})$
M104	10	BEC	0	1	$X_{critical} = X_0 + \frac{10 * (X_1 - X_0)}{S_M}(L_{BEC})$
	11	BEM	5		$X_{critical} = X_5$
	12	BIC1	9	10	$X_{critical} = X_{10} + \frac{10 * (X_9 - X_{10})}{S_M}(L_{BIC})$
M105	13	TIC2	0	1	$X_{critical} = X_0 + \frac{10 * (X_1 - X_0)}{S_M}(L_{TIC})$
	14	TIM1	5		$X_{critical} = X_5$
	15	TIC3	9	10	$X_{critical} = X_{10} + \frac{10 * (X_9 - X_{10})}{S_M}(L_{TIC})$
M107	19	BIC2	0	1	$X_{critical} = X_0 + \frac{10 * (X_1 - X_0)}{S_M}(L_{BIC})$
	20	BIM1	5		$X_{critical} = X_5$
	21	BIC3	9	10	$X_{critical} = X_{10} + \frac{10 * (X_9 - X_{10})}{S_M}(L_{BIC})$

where: $X_{critical}$ = the demand at the critical section
X_N = the demand at the Nth 10th point
S_M = the model span length (ft)
L_N = the length from the closest corner node to critical section, N (ft)

CULV5 Step 4. From the CULV-5 output file, **SUMMARY OF INDIVIDUAL UNFACTORED MOMENTS, SHEAR AND AXIAL FORCES** tables, and based on the critical sections established in Step 3, record the "Raw Demands" for vertical dead load (VDL), lateral dead load (LDL), maximum vertical live load (+VLL), minimum vertical live load (-VLL), and lateral live load (LLL) demands at each critical section, both corners and mid-spans. See yellow highlighting in the CULV-5 output and the summary in Table IX-9.

TABLE IX-9. CULV-5 RAW DEMANDS AT CRITICAL SECTIONS.

Member	*Sections*		*M VDL*	*M LDL*	*M +VLL*	*M -VLL*	*M LLL*	*V VDL*	*V LDL*	*V +VLL*	*V -VLL*	*V LLL*	*P VDL*	*P LDL*	*P +VLL*	*P -VLL*	*P LLL*
M101	1	WBEC	-2.134	-1.531	0.135	-0.648	-0.295	0.040	2.411	0.026	-0.030	0.420	-3.735	-0.267	0.099	-1.227	-0.054
	2	WEM	-1.993	2.314	0.129	-0.673	0.450	0.040	-0.122	0.026	-0.030	0.000	-3.735	-0.267	0.108	-1.227	-0.054
	3	WTEC	-1.851	-1.499	0.135	-0.701	-0.294	0.040	-1.912	0.026	-0.030	-0.420	-3.735	-0.267	0.107	-1.224	-0.054
M102	4	TEC	-0.747	-2.170	0.247	-0.489	-0.442	3.458	0.267	1.164	1.361	0.054	0.041	-2.073	0.008	-0.014	-0.467
	5	TEM	6.184	-0.847	2.240	-0.445	-0.172	-0.704	0.267	0.188	-0.385	0.054	0.041	-2.073	-0.007	-0.001	-0.467
	6	TIC1	-7.729	0.475	0.000	-2.248	0.098	-4.865	0.267	-1.164	-1.361	0.054	0.041	-2.073	0.000	-0.008	-0.467
M103	7	WBIC1	0.621	-0.233	0.473	-0.313	-0.041	-0.017	0.005	0.040	-0.035	0.000	-9.581	0.267	-1.284	-1.290	0.054
	8	WIM1	0.549	-0.211	0.497	-0.346	-0.041	-0.017	0.005	0.040	-0.035	0.000	-9.581	0.267	-1.284	-1.290	0.054
	9	WTIC1	0.491	-0.193	0.525	-0.388	-0.041	-0.017	0.005	0.040	-0.035	0.000	-9.581	0.267	-1.297	-1.402	0.054
M104	10	BEC	-0.948	-2.389	0.240	-0.443	-0.444	3.817	0.295	1.098	1.379	0.055	-0.040	-2.739	-0.005	-0.005	-0.468
	11	BEM	6.723	-0.925	2.154	-0.415	-0.172	-0.770	0.295	0.000	-0.184	0.055	-0.040	-2.739	-0.007	-0.001	-0.468
	12	BIC1	-8.584	0.538	0.003	-1.984	0.099	-5.356	0.295	-1.098	-1.379	0.055	-0.040	-2.739	-0.005	-0.007	-0.468
M105	13	TIC2	-7.479	0.372	0.000	-2.187	0.075	4.161	0.000	1.406	1.179	0.000	0.024	-2.068	0.000	0.002	-0.467
	14	TIM1	2.944	0.372	1.761	-0.925	0.075	0.000	0.000	0.339	-0.273	0.000	0.024	-2.068	0.011	-0.001	-0.467
	15	TIC3	-7.479	0.372	0.000	-2.187	0.075	-4.161	0.000	-1.406	-1.179	0.000	0.024	-2.068	0.000	0.002	-0.467
M107	19	BIC2	-8.224	0.405	0.000	-1.996	0.076	4.587	0.000	1.300	1.210	0.000	-0.024	-2.744	0.000	-0.011	-0.468
	20	BIM1	3.265	0.405	1.727	-0.876	0.076	0.000	0.000	0.178	-0.110	0.000	-0.024	-2.744	0.011	-0.001	-0.468
	21	BIC3	-8.224	0.405	0.000	-1.996	0.076	-4.587	0.000	-1.300	-1.210	0.000	-0.024	-2.744	0.000	-0.011	-0.468

CULV5 Step 5. Calculate the dead and live load demand for each demand type (moment, shear and axial), for each load case at each critical section using Equation VI-9 and Equation VI-10. Note that the live load demands have a maximum and minimum calculation. To maintain a systematic approach, typical practice is to determine both the maximum and minimum live loads for each type of demand and select the minimum (controlling) value when calculating rating factors. Calculate the dead and live load demand at each section using Equation VI-9 and Equation VI-10 where *D* is the dead load demand and *L* is the live load demand required for rating in Equation II-1. See Table IX-10 and Table IX-11.

CULV5 Step 6. After determining the demands, use Equation VI-1 to check that actual thrust demand is lower than the incidental axial load assumed in the moment capacity equations. This check is performed in the last column of Table IX-10 and Table IX-11.

TABLE IX-10. CULV-5 DEMANDS FOR TOTAL LOAD CASE.

Member	*Sections*		*M_D (k-ft/ft)*	*M_L (max) (k-ft/ft)*	*M_L (min) (k-ft/ft)*	*V_D (klf)*	*V_L (max) (klf)*	*V_L (min) (klf)*	*P_D (klf)*	*P_L (max) (klf)*	*P_L (min) (klf)*	*Thrust Check*
M101	1	WBEC	-3.665	-0.160	-0.943	2.451	0.446	0.390	-4.002	0.045	-1.281	OK
	2	WEM	0.321	0.579	-0.223	-0.082	0.026	-0.030	-4.002	0.054	-1.281	OK
	3	WTEC	-3.350	-0.159	-0.995	-1.872	-0.394	-0.450	-4.002	0.053	-1.278	OK
M102	4	TEC	-2.916	-0.196	-0.931	3.725	1.218	1.415	-2.032	-0.459	-0.481	OK
	5	TEM	5.337	2.068	-0.617	-0.437	0.242	-0.331	-2.032	-0.474	-0.468	OK
	6	TIC1	-7.254	0.098	-2.150	-4.598	-1.110	-1.307	-2.032	-0.467	-0.475	OK
M103	7	WBIC1	0.388	0.432	-0.354	-0.012	0.040	-0.035	-9.314	-1.230	-1.236	OK
	8	WIM1	0.338	0.456	-0.387	-0.012	0.040	-0.035	-9.314	-1.230	-1.236	OK
	9	WTIC1	0.298	0.484	-0.429	-0.012	0.040	-0.035	-9.314	-1.243	-1.348	OK
M104	10	BEC	-3.337	-0.204	-0.887	4.112	1.153	1.434	-2.779	-0.473	-0.473	OK
	11	BEM	5.798	1.982	-0.587	-0.475	0.055	-0.129	-2.779	-0.475	-0.469	OK
	12	BIC1	-8.046	0.102	-1.885	-5.061	-1.043	-1.324	-2.779	-0.473	-0.475	OK
M105	13	TIC2	-7.107	0.075	-2.112	4.161	1.406	1.179	-2.044	-0.467	-0.465	OK
	14	TIM1	3.316	1.836	-0.850	0.000	0.339	-0.273	-2.044	-0.456	-0.468	OK
	15	TIC3	-7.107	0.075	-2.112	-4.161	-1.406	-1.179	-2.044	-0.467	-0.465	OK
M107	19	BIC2	-7.819	0.076	-1.920	4.587	1.300	1.210	-2.768	-0.468	-0.479	OK
	20	BIM1	3.670	1.803	-0.800	0.000	0.178	-0.110	-2.768	-0.457	-0.469	OK
	21	BIC3	-7.819	0.076	-1.920	-4.587	-1.300	-1.210	-2.768	-0.468	-0.479	OK

TABLE IX-11. CULV5 DEMAND FOR THE REDUCED LATERAL LOAD CASE.

Member	*Sections*		*M_D (k-ft/ft)*	*M_L (max) (k-ft/ft)*	*M_L (min) (k-ft/ft)*	*V_D (klf)*	*V_L (max) (klf)*	*V_L (min) (klf)*	*P_D (klf)*	*P_L (max) (klf)*	*P_L (min) (klf)*	*Thrust Check*
M101	1	WBEC	-2.900	0.135	-0.648	1.246	0.026	-0.030	-3.869	0.099	-1.227	OK
	2	WEM	-0.836	0.129	-0.673	-0.021	0.026	-0.030	-3.869	0.108	-1.227	OK
	3	WTEC	-2.600	0.135	-0.701	-0.916	0.026	-0.030	-3.869	0.107	-1.224	OK
M102	4	TEC	-1.832	0.247	-0.489	3.591	1.164	1.361	-0.996	0.008	-0.014	OK
	5	TEM	5.761	2.240	-0.445	-0.571	0.188	-0.385	-0.996	-0.007	-0.001	OK
	6	TIC1	-7.492	0.000	-2.248	-4.731	-1.164	-1.361	-0.996	0.000	-0.008	OK
M103	7	WBIC1	0.504	0.473	-0.313	-0.015	0.040	-0.035	-9.448	-1.284	-1.290	OK
	8	WIM1	0.444	0.497	-0.346	-0.015	0.040	-0.035	-9.448	-1.284	-1.290	OK
	9	WTIC1	0.394	0.525	-0.388	-0.015	0.040	-0.035	-9.448	-1.297	-1.402	OK
M104	10	BEC	-2.142	0.240	-0.443	3.965	1.098	1.379	-1.410	-0.005	-0.005	OK
	11	BEM	6.261	2.154	-0.415	-0.623	0.000	-0.184	-1.410	-0.007	-0.001	OK
	12	BIC1	-8.315	0.003	-1.984	-5.209	-1.098	-1.379	-1.410	-0.005	-0.007	OK
M105	13	TIC2	-7.293	0.000	-2.187	4.161	1.406	1.179	-1.010	0.000	0.002	OK
	14	TIM1	3.130	1.761	-0.925	0.000	0.339	-0.273	-1.010	0.011	-0.001	OK
	15	TIC3	-7.293	0.000	-2.187	-4.161	-1.406	-1.179	-1.010	0.000	0.002	OK
M107	19	BIC2	-8.022	0.000	-1.996	4.587	1.300	1.210	-1.396	0.000	-0.011	OK
	20	BIM1	3.468	1.727	-0.876	0.000	0.178	-0.110	-1.396	0.011	-0.001	OK
	21	BIC3	-8.022	0.000	-1.996	-4.587	-1.300	-1.210	-1.396	0.000	-0.011	OK

CULV5 Step 7. This step goes beyond calculation of demand loads and has to do with calculating the culvert load rating factors.

Per the culvert rating flow chart (Figure III-2) calculate Inventory and Operating rating factors for each critical section, for each demand type, for each load case based on Equation II-1. This calculation uses the capacity values for each critical section as determined in Appendix B.

When calculating the rating factors, exercise extreme care regarding the signs for both demands and capacities.

- Live load and capacity must be in the same sign and direction.
- If the live load and dead load are in opposite directions or the calculated rating is negative, a check should be made to insure that the structure has adequate capacity to support the dead load. I.E. $C \geq 1.3D$

Table IX-12 and Table IX-13 summarize these calculations.

CULV5 Step 8. Select the controlling (minimum) Rating Factors for Inventory and Operating Levels for each critical section. These appear in the two right columns of Table IX-12 and Table IX-13.

CULV5 Step 9. Select the controlling (minimum) Rating Factors for Inventory and Operating Levels for the entire culvert. These appear at the bottom of Table IX-13.

CULV5 Step 10. If shear controls the load rating, the load rater should perform a less-conservative analysis of the shear failure mode based on shear critical sections as per the provisions in Section VI.C. In this example, the controlling failure mode is moment, so additional shear analysis is not required.

CULV5 Step 11. Calculate the Inventory Rating and Operating Rating for the culvert. Multiply controlling load rating factor by the truck tractor tonnage (W= 20 tons) according to Equation II-2.

Summary: Based on a Level 1 analysis using CULV-5, the Inventory Rating is HS-9 while the Operating Rating is HS-15. If the culvert condition is fair this requires posting at the Inventory Level or posting at the Operating Level with an inspection frequency of 24 months. If the condition is poor, the culvert should be posted at the Inventory Level and inspected more frequently than every 24 months.

TABLE IX-12. CULV5 TOTAL LOAD CASE RATING FACTOR CALCULATIONS.

			(Max) Rating Factors						(Min) Rating Factors						Controlling RF	
Member	*Sections*		IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_C	ORF_C
M101	1	WBEC	3.20	5.35	5.36	8.95	2062.79	3443.27	0.54	0.91	6.13	10.24	72.46	120.96	0.54	0.91
	2	WEM	1.05	1.76	150.42	251.09	1752.62	2925.53	13.01	21.71	127.09	212.14	73.88	123.32	1.05	1.76
	3	WTEC	4.42	7.38	6.96	11.62	1767.30	2950.03	0.70	1.18	6.09	10.17	72.66	121.28	0.70	1.18
M102	4	TEC	11.01	18.38	3.21	5.35	289.14	482.64	2.41	4.02	2.76	4.60	275.32	459.57	2.41	4.02
	5	TEM	0.73	1.22	26.61	44.43	266.16	444.28	8.50	14.20	16.71	27.89	269.57	449.98	0.73	1.22
	6	TIC1	91.58	152.87	2.70	4.51	296.40	494.76	1.47	2.46	2.29	3.83	291.98	487.38	1.47	2.46
M103	7	WBIC1	2.22	3.71	96.73	161.46	72.12	120.38	4.03	6.72	110.13	183.84	71.77	119.80	2.22	3.71
	8	WIM1	2.17	3.63	96.73	161.46	72.12	120.38	3.61	6.02	110.13	183.84	71.77	119.80	2.17	3.63
	9	WTIC1	2.10	3.50	96.73	161.46	71.35	119.10	3.20	5.34	110.13	183.84	65.82	109.86	2.10	3.50
M104	10	BEC	9.39	15.68	3.18	5.30	279.54	466.61	2.24	3.74	2.56	4.27	279.23	466.10	2.24	3.74
	11	BEM	0.62	1.04	117.52	196.16	264.66	441.77	9.41	15.71	42.70	71.27	268.04	447.43	0.62	1.04
	12	BIC1	92.69	154.72	2.60	4.34	294.24	491.16	1.76	2.94	2.05	3.43	292.90	488.91	1.76	2.94
M105	13	TIC2	120.81	201.66	2.59	4.32	296.38	494.73	1.54	2.58	3.09	5.16	297.59	496.75	1.54	2.58
	14	TIM1	1.48	2.48	18.23	30.42	276.65	461.79	4.75	7.93	21.22	35.42	269.56	449.95	1.48	2.48
	15	TIC3	120.81	201.66	2.32	3.87	296.38	494.73	1.54	2.58	2.76	4.61	297.59	496.75	1.54	2.58
M107	19	BIC2	124.96	208.59	2.60	4.34	297.20	496.10	1.81	3.01	2.80	4.67	289.98	484.04	1.81	3.01
	20	BIM1	1.39	2.32	34.71	57.94	275.10	459.20	5.31	8.87	52.66	87.90	268.06	447.45	1.39	2.32
	21	BIC3	124.96	208.59	2.31	3.85	297.20	496.10	1.81	3.01	2.48	4.14	289.98	484.04	1.81	3.01

TABLE IX-13. CULV5 REDUCED LATERAL LOAD CASE RATING FACTORS.

			(Max) Rating Factors						(Min) Rating Factors						Controlling RF	
Member	*Sections*		IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_C	ORF_C
M101	1	WBEC	21.04	35.12	119.83	200.02	938.44	1566.47	1.50	2.50	153.60	256.40	75.72	126.39	1.50	2.50
	2	WEM	10.10	16.86	149.02	248.74	877.05	1464.00	3.28	5.47	128.31	214.18	77.20	128.86	3.28	5.47
	3	WTEC	19.73	32.94	169.64	283.16	872.16	1455.83	1.64	2.74	110.44	184.34	75.93	126.75	1.64	2.74
M102	4	TEC	25.10	41.89	3.42	5.71	15501.32	25875.28	5.75	9.60	2.92	4.88	9236.45	15417.76	2.92	4.88
	5	TEM	0.56	0.94	34.68	57.90	18111.49	30232.26	12.36	20.64	14.16	23.63	126780.46	211625.85	0.56	0.94
	6	TIC1	NA	NA	2.51	4.19	NA	NA	1.35	2.25	2.14	3.58	19685.50	32859.64	1.35	2.25
M103	7	WBIC1	1.88	3.15	96.76	161.52	69.02	115.22	4.78	7.98	110.09	183.76	68.70	114.68	1.88	3.15
	8	WIM1	1.87	3.12	96.76	161.52	69.02	115.22	4.22	7.04	110.09	183.76	68.70	114.68	1.87	3.12
	9	WTIC1	1.82	3.04	96.76	161.52	68.32	114.04	3.69	6.15	110.09	183.76	63.22	105.54	1.82	3.04
M104	10	BEC	26.68	44.54	3.41	5.70	29204.91	48749.74	5.92	9.89	2.72	4.55	26211.86	43753.65	2.72	4.55
	11	BEM	0.45	0.74	NA	NA	18076.06	30173.12	13.98	23.33	29.46	49.17	126532.45	211211.85	0.45	0.74
	12	BIC1	3235.59	5400.95	2.39	3.99	29862.05	49846.65	1.59	2.65	1.91	3.19	20399.45	34051.38	1.59	2.65
M105	13	TIC2	NA	NA	2.59	4.32	NA	NA	1.44	2.41	3.09	5.16	73264.68	122295.66	1.44	2.41
	14	TIM1	1.61	2.69	18.23	30.42	11524.71	19237.40	4.24	7.08	21.22	35.42	126771.78	211611.35	1.61	2.69
	15	TIC3	NA	NA	2.32	3.87	NA	NA	1.44	2.41	2.76	4.61	73264.68	122295.66	1.44	2.41
M107	19	BIC2	NA	NA	2.60	4.34	NA	NA	1.68	2.80	2.80	4.67	11997.94	20027.33	1.68	2.80
	20	BIM1	1.52	2.54	34.71	57.94	11503.68	19202.30	4.71	7.86	52.66	87.90	126540.53	211225.35	1.52	2.54
	21	BIC3	NA	NA	2.31	3.85	NA	NA	1.68	2.80	2.48	4.14	11997.94	20027.33	1.68	2.80
									Controlling Rating Factor			BEM - M(max) - RLL			0.45	0.74
									Load Rating			*(HS equivalent)*			9	15

Appendix D. Level 2: RISA-2D with Springs Example

RISA-2D Spring Step 1. Calculate all loads using Equation VI-2, Equation VI-4, Equation VI-5, Equation VI-6 and Equation VI-7. The magnitudes of these loads are summarized in Table IX-14.

TABLE IX-14. RISA-2D LOADS.

Type	*Abbr.*	*Value*	*Units*
vertical dead load	DL_v	0.720	ksf
horizontal dead load at top	DL_{hT}	0.384	ksf
horizontal dead load at bottom	DL_{hB}	0.851	ksf
horizontal live load	LL_h	0.120	ksf
vertical live load on top	LL_{vT}	0.230	ksf

RISA-2D Spring Step 2. Create a model consistent with Figure VI-2 and Figure VI-10:

a. Disable cracked sections and shear deformations within the global parameters. Reduce output to three points per member. Screen shots showing this step are seen in Figure IX-7.

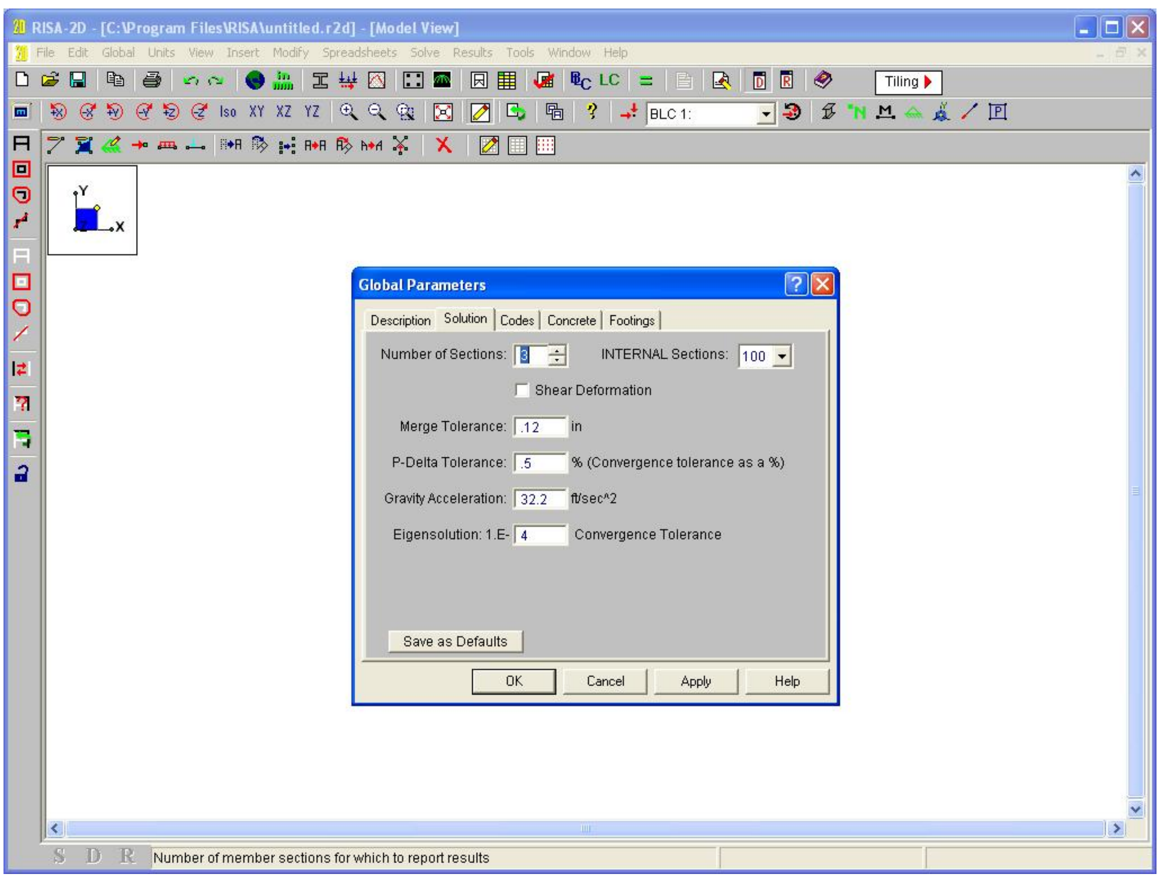


FIGURE IX.7. RISA-2D GLOBAL PARAMETERS.

b. Lay out corner nodes as seen in Figure IX-8.

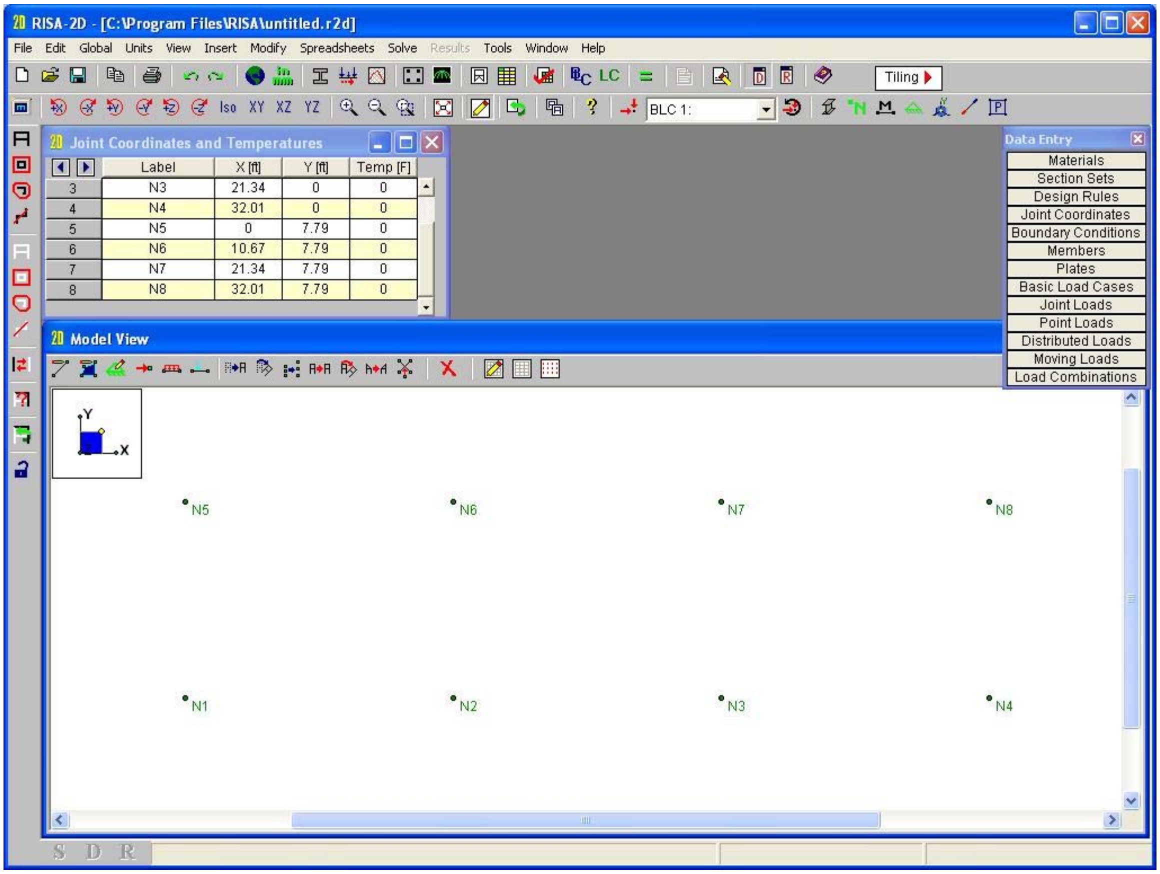


FIGURE IX.8. RISA-2D CORNER JOINT COORDINATES.

c. Connect nodes using members with rectangular cross sections and appropriate concrete properties according to Table II-1 and Table IV-1. Draw members counterclockwise around the center of the culvert to produce consistent moment sign conventions as per Figure VI-15. Figure IX-9 shows the draw member box used to create the members. Figure IX-10 illustrates a few ways to check that beam directions are defined properly.

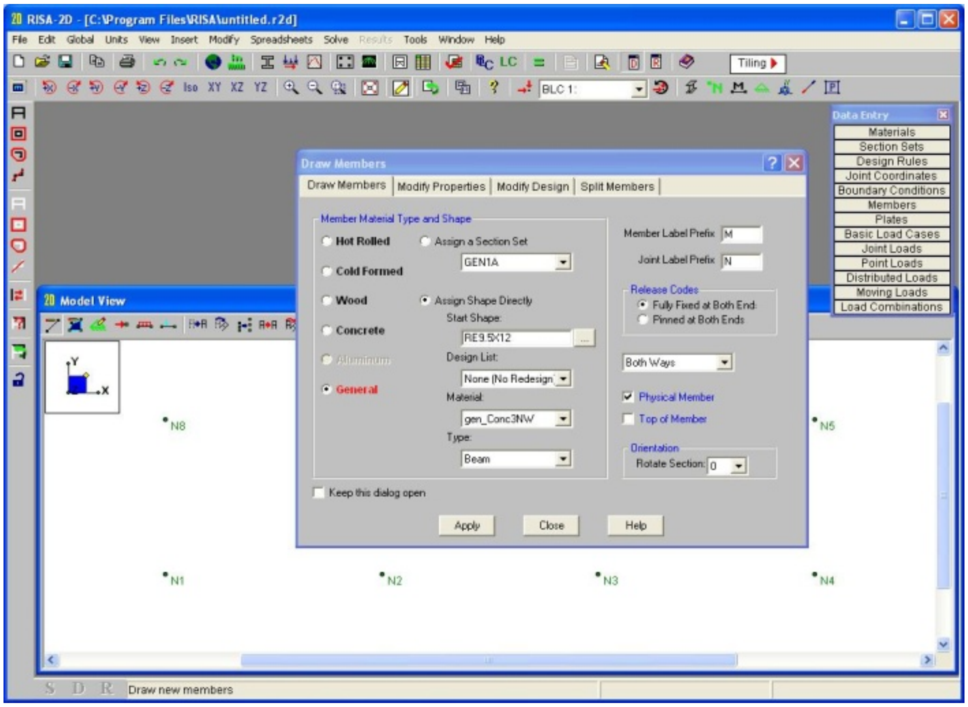


FIGURE IX.9. RISA-2D MEMBER CREATION.

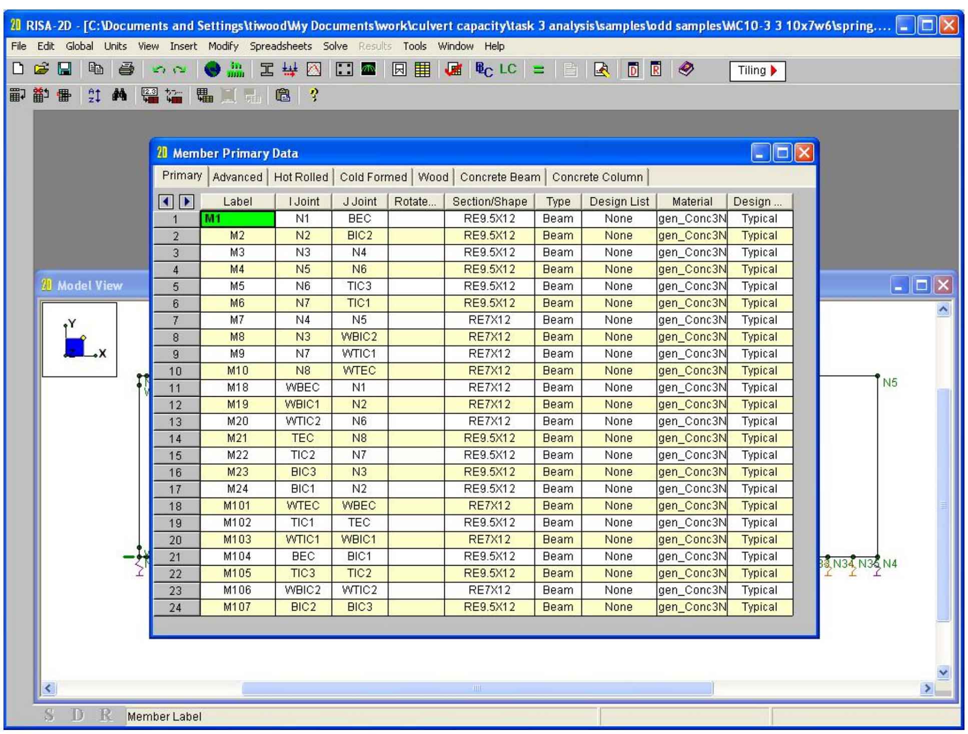


FIGURE IX.10. RISA-2D MODEL LAYOUT CHECK.

d. Using the "split member" function, add support nodes to the bottom members and set boundary conditions according to Figure VI-11 with spring constants from Equation VI-11. Table IX-15 summarizes the spring support calculations. Figure IX-11 and Figure IX-12 show how to use the split member function and the boundary condition controls to properly restrain the structure.

TABLE IX-15. SPRING SUPPORTS CALCULATIONS.

Type	*Abbr.*	*Value*	*Units*
spring spacing	s	12.8	in
modulus of subgrade reaction	k	150	pci
interior spring constant	κ	23.0	kli
exterior spring constant	κ	11.5	kli

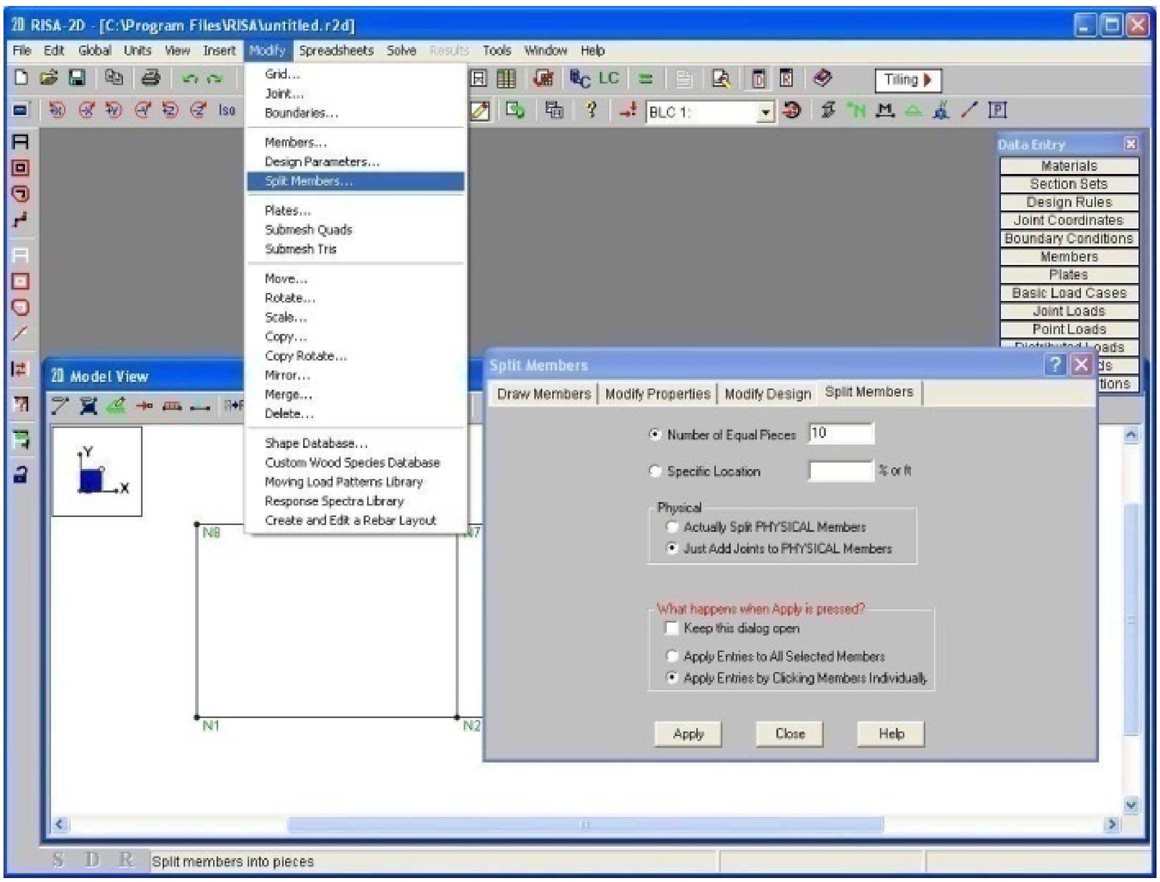


FIGURE IX.11. RISA-2D MODEL – ADDING THE NODES USING THE "JUST ADD" FUNCTION

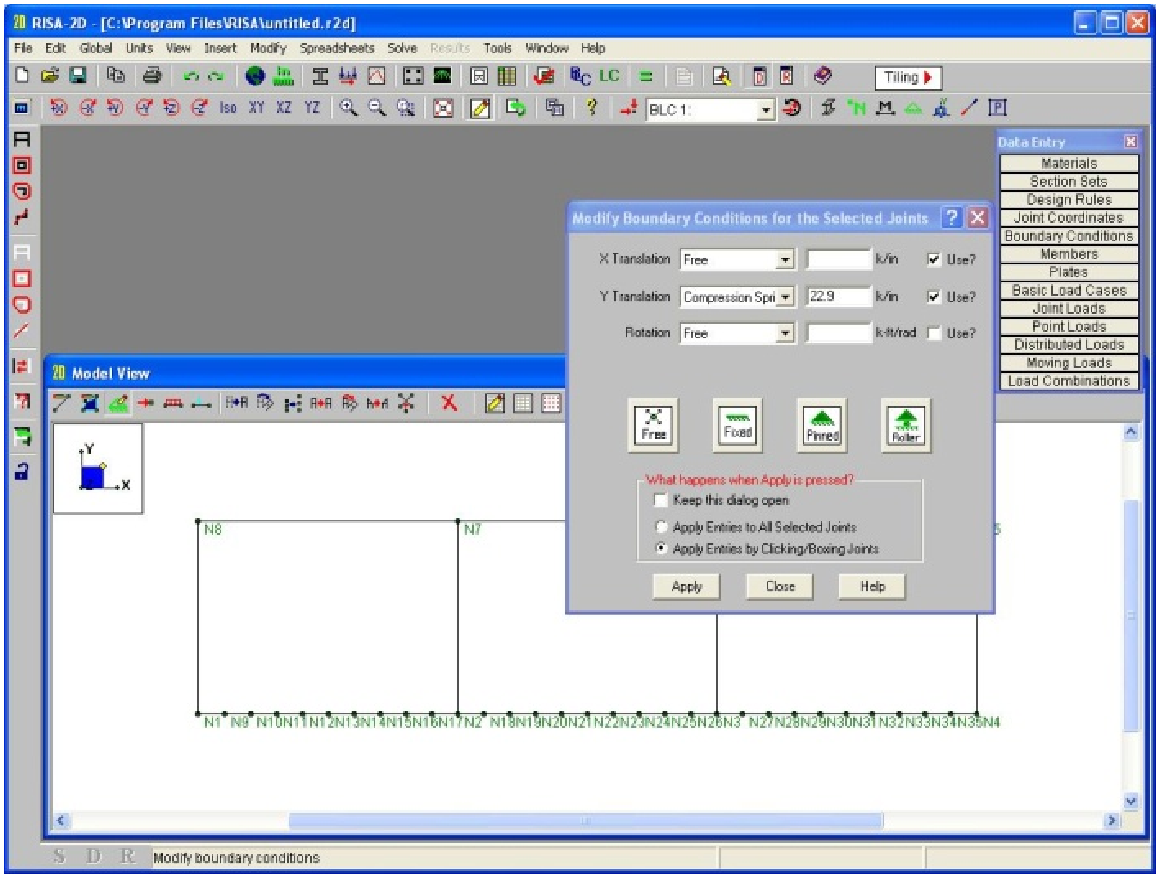


FIGURE IX.12. RISA-2D MODEL – DEFINING THE BOUNDARY CONDITIONS

RISA-2D Spring Step 3. Apply the loads according to Figure VI-10 in separate Basic Load Cases. Figure IX-13 depicts labeling the three static Basic Load Cases.

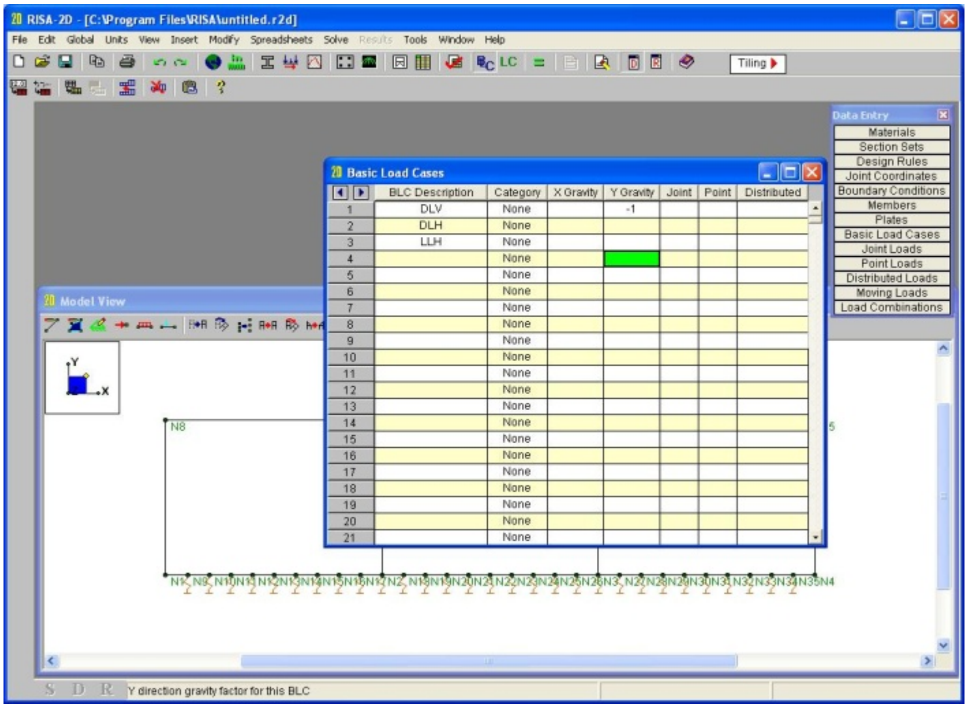


FIGURE IX.13. RISA-2D – IDENTIFICATION OF ALL LOAD GROUPS AND USING GRAVITY FEATURE

a. Vertical Dead Load, DL_v (Equation VI-2). Be sure to include the self-weight gravity loading by including a factor of (-1) in the Y-gravity direction. Figure IX-13 shows where gravity is activated. Figure IX-14 shows how to define the distributed dead load.

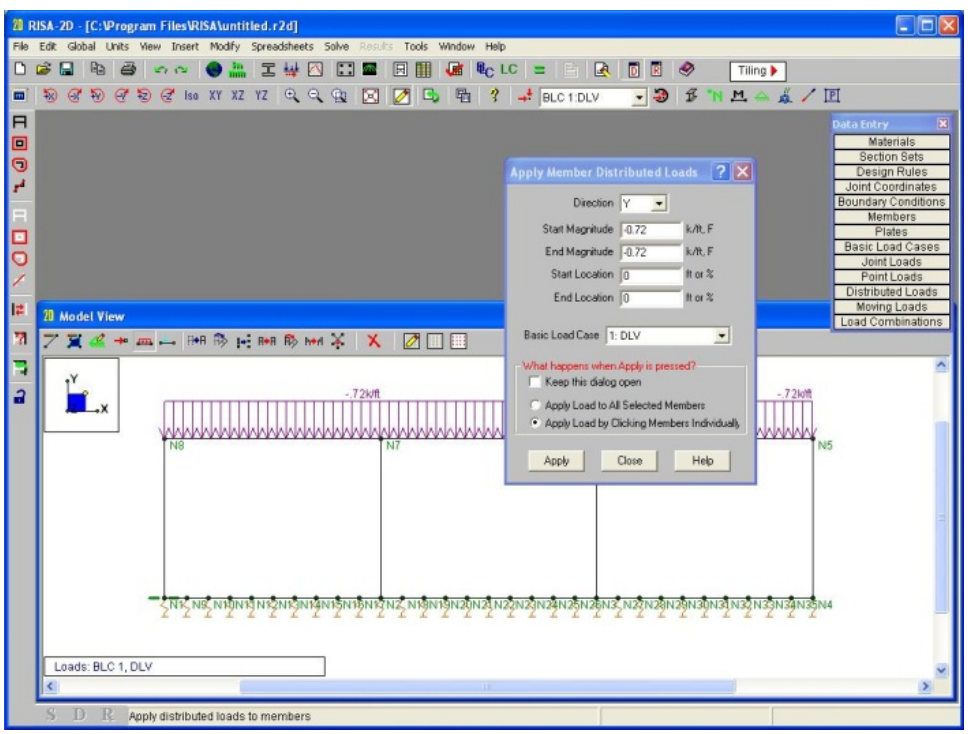


FIGURE IX.14. RISA-2D – APPLICATION OF VERTICAL DEAD LOAD (DLV)

b. Horizontal Dead Load, DL_h (Equation VI-4 and Equation VI-5). Figure IX-15 shows applying this load in RISA-2D.

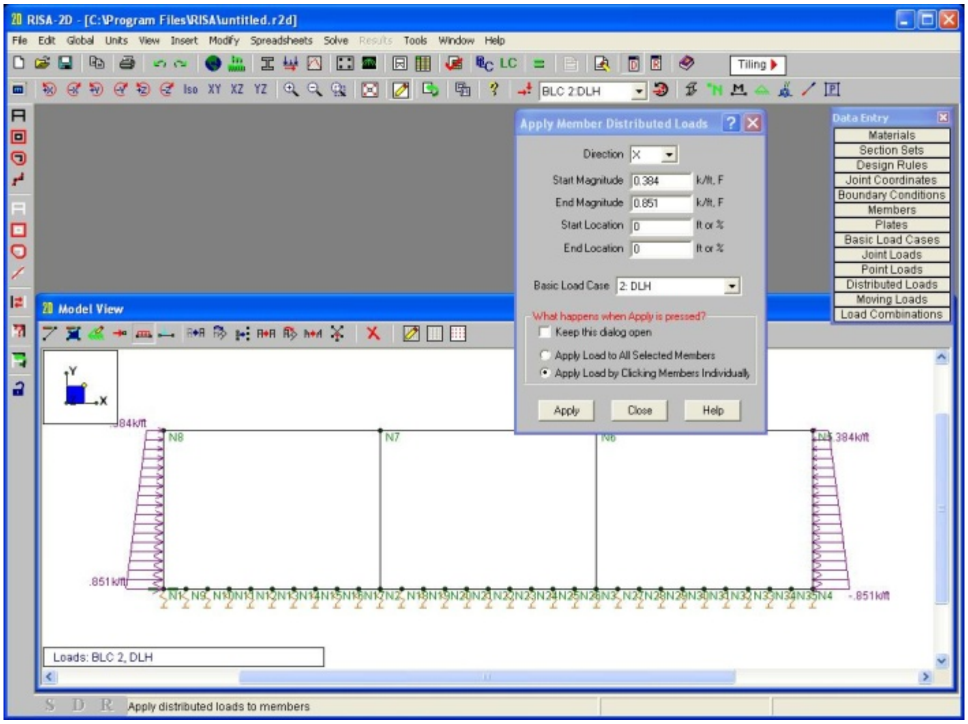


FIGURE IX.15. RISA-2D – APPLICATION OF HORIZONTAL DEAD LOAD (DLH)

c. Horizontal Live Load, LL_h (Equation VI-6). Figure IX-16 illustrates this load application.

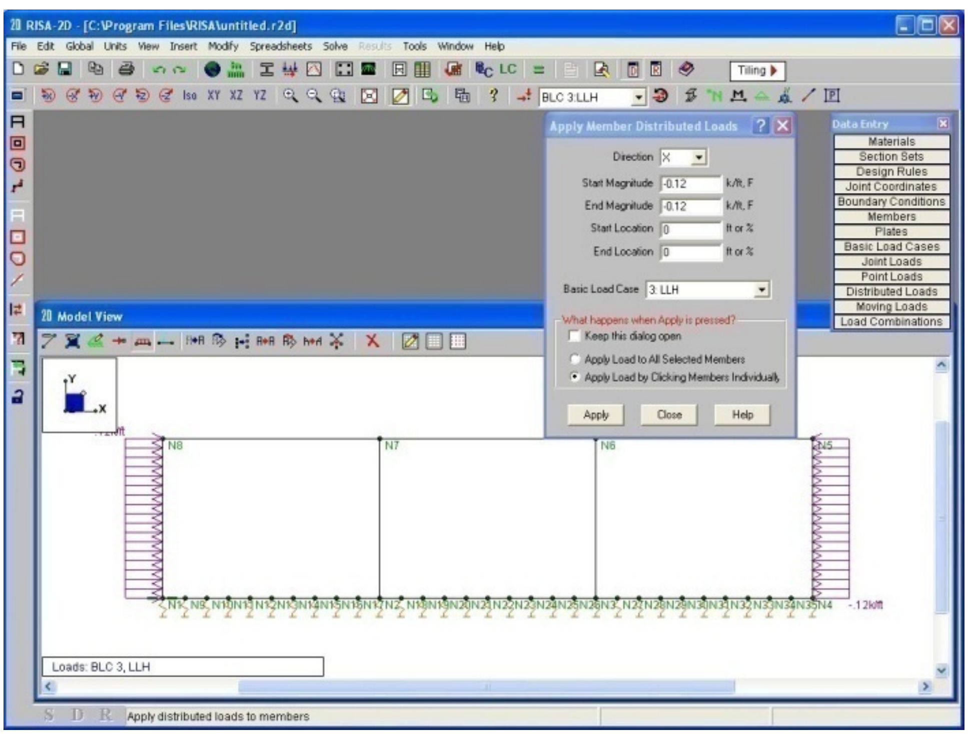


FIGURE IX.16. RISA-2D – APPLICATION OF HORIZONTAL LIVE LOAD (LLH)

RISA-2D Spring Step 4. Vertical Live Load, LL_{VT} (Equation VI-7) must be calculated and placed as a *moving* load as seen in Figure VI-12, Figure VI-13 and Figure VI-14. The moving load will be approximated by creating a *moving load pattern* of 10 equivalent, uniformly-spaced, point loads over the length of each load as seen in Figure VI-16, Figure VI-17 and Figure VI-18. These figures show the moving load discretized and grouped in terms of the 10 equivalent, uniformly-spaced, point loads. Check the "Both Ways" box to insure that the live load moves from left to right and right to left.

Figure IX-17 through Figure IX-20 presents a series of four images depicting this process. The first figure shows how to create a moving load in RISA. The next shows how to add a pattern and define the load case. The third image shows the application of the moving load to the structure. The fourth image is an animated graphic that facilitates checking to make sure the moving load is properly applied.

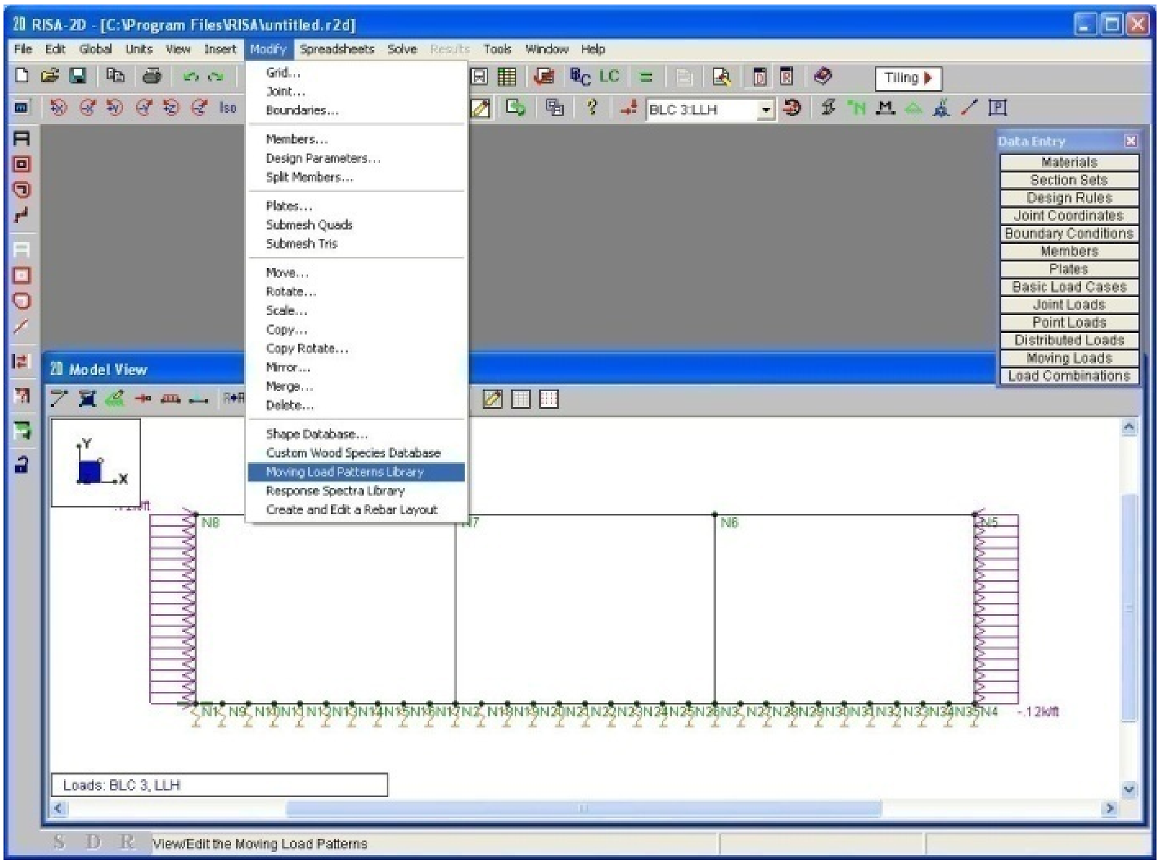


FIGURE IX.17. RISA-2D – HOW TO CREATE A MOVING LOAD

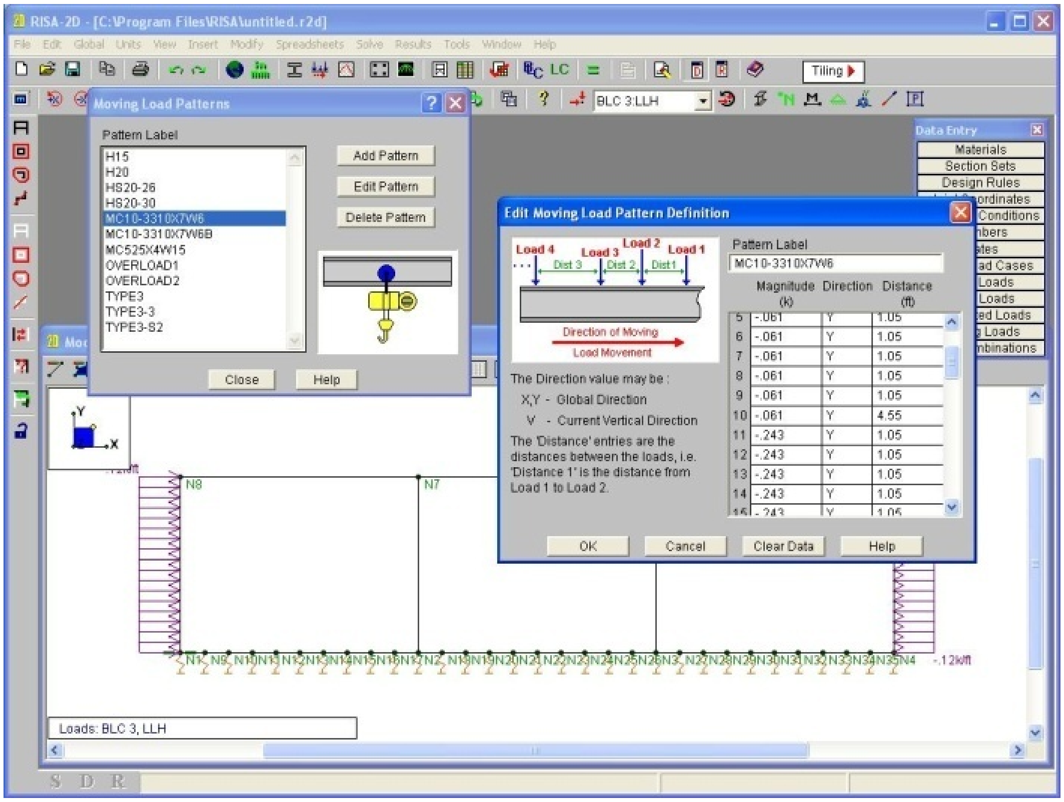


FIGURE IX.18. RISA-2D – ADDING A PATTERN AND DEFINING THE MOVING LOAD CASE

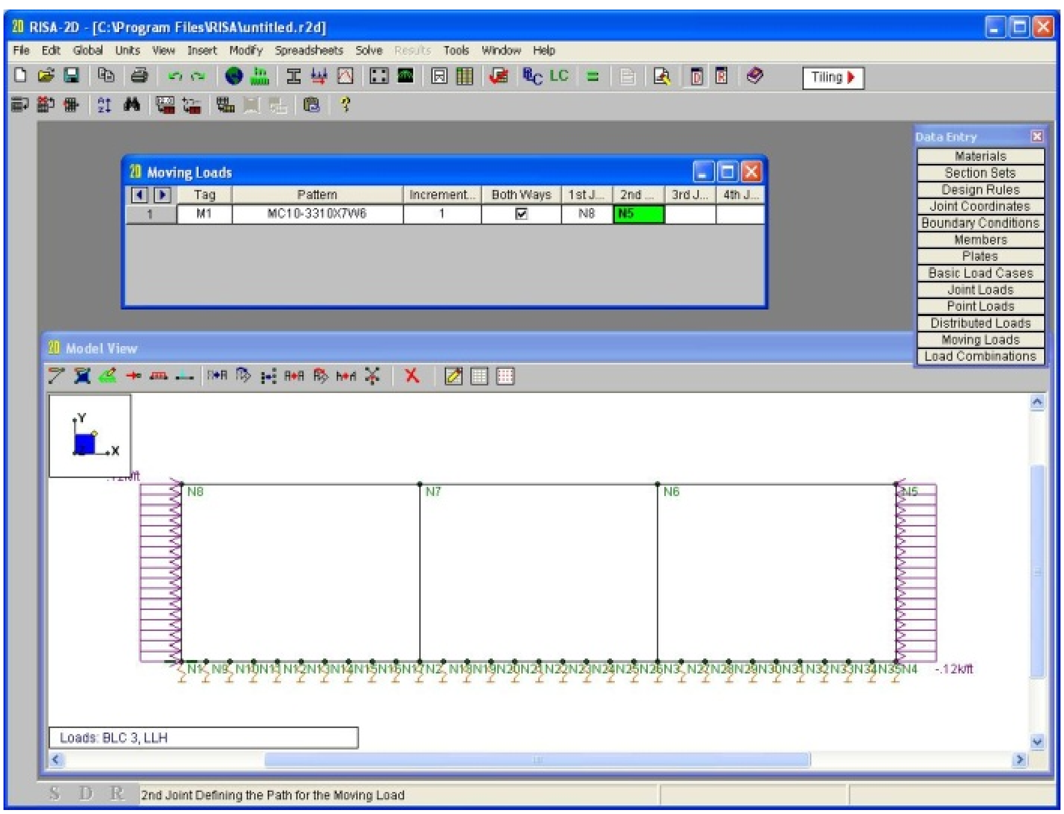


FIGURE IX.19. RISA-2D – APPLY THE MOVING LOAD TO THE STRUCTURE

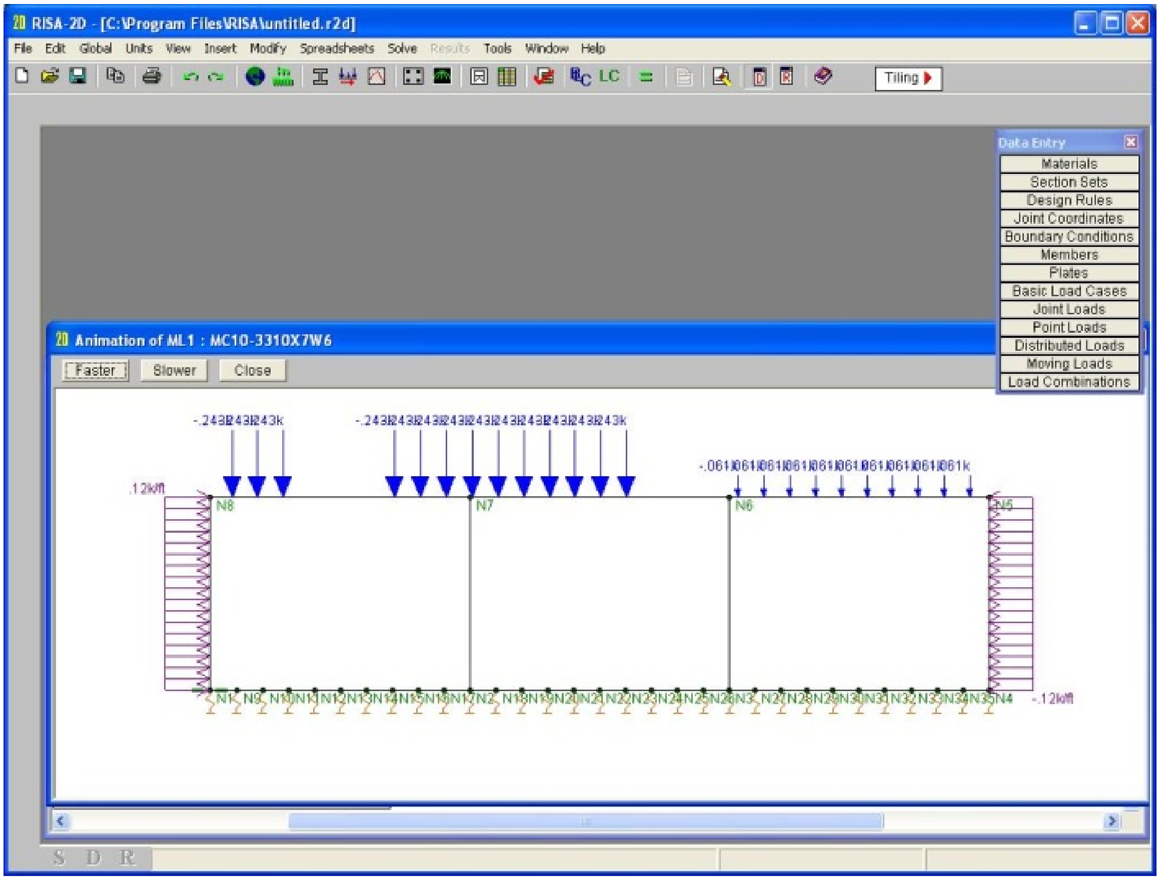


FIGURE IX.20. RISA-2D – ANIMATED GRAPHIC TO CHECK THE MOVING LOAD CASE

RISA-2D Spring Step 5. Use the "split member" function to split the members to create the critical section nodes. Re-label and sort the critical members using a convention similar to the CULV-5 naming convention. See Figure VI-19.

Figure IX-21 through Figure IX-23 presents a series of three images depicting the process. The first image shows how to create the critical sections. The second image illustrates the process of relabeling the nodes to represent the critical sections. The third image shows a check demonstrating that the relabeled beam elements are now correctly labeled and can be sorted by the new member name. This facilitates subsequent data analysis.

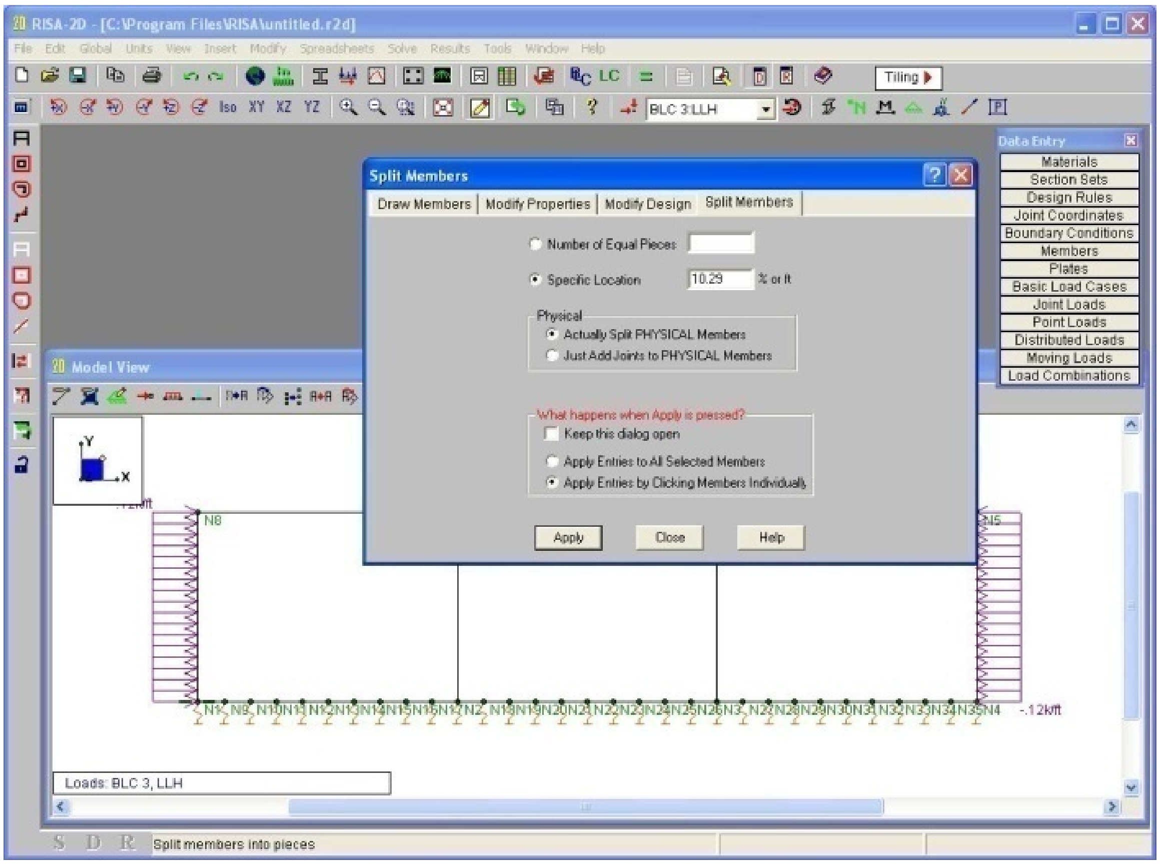


FIGURE IX.21. RISA-2D – SPLIT MEMBER TO CREATE CORNER CRITICAL SECTION NODES.

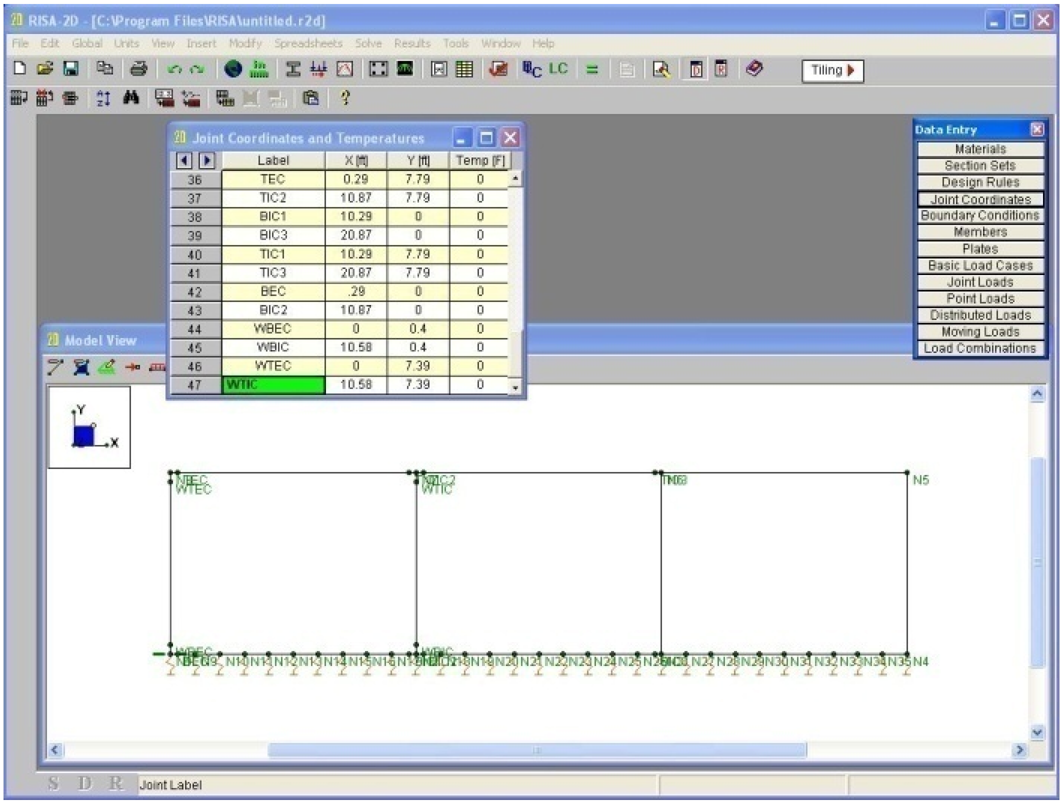


FIGURE IX.22. RISA-2D – RELABEL NODES TO REPRESENT CRITICAL SECTIONS.

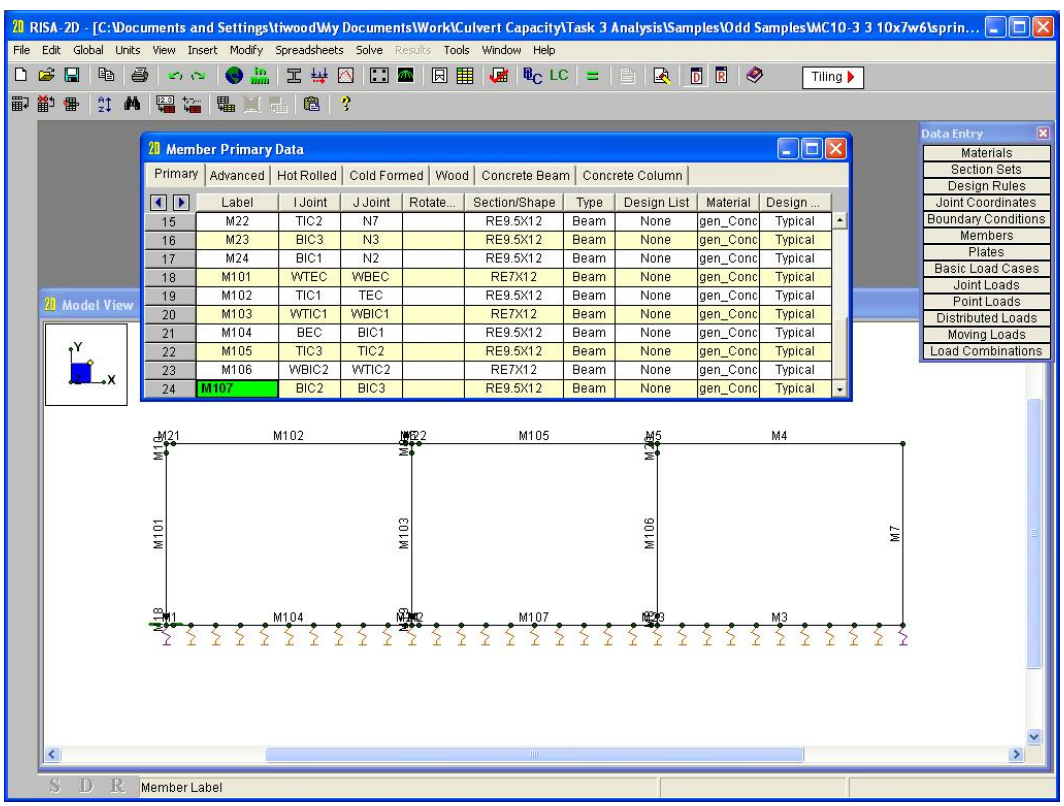


FIGURE IX.23. RISA-2D – RELABELED BEAM ELEMENTS TO FACILITATE SORT BY MEMBER NAMES.

RISA-2D Spring Step 6. Create four Load Combinations for dead and live demands.

a. Use the following Basic Load Case Factors for the Total Load Case dead load demands
   - DL_v factor of 1.0
   - DL_h factor of 1.0

b. Use the following Basic Load Case Factors for the Total Load Case live load demands:
   - LL_v factor of 1.0
   - LL_h factor of 1.0

c. Use the following Basic Load Case Factors for the Reduced Lateral Load Case dead load demands:
   - DL_v factor of 1.0
   - DL_h factor of 0.5

d. Use the following Basic Load Case Factors the Reduced Lateral Load Case live load demands:
   - LL_v factor of 1.0
   - LL_h factor of 0.0

Figure IX-24 shows how to define the four load combinations in RISA-2D

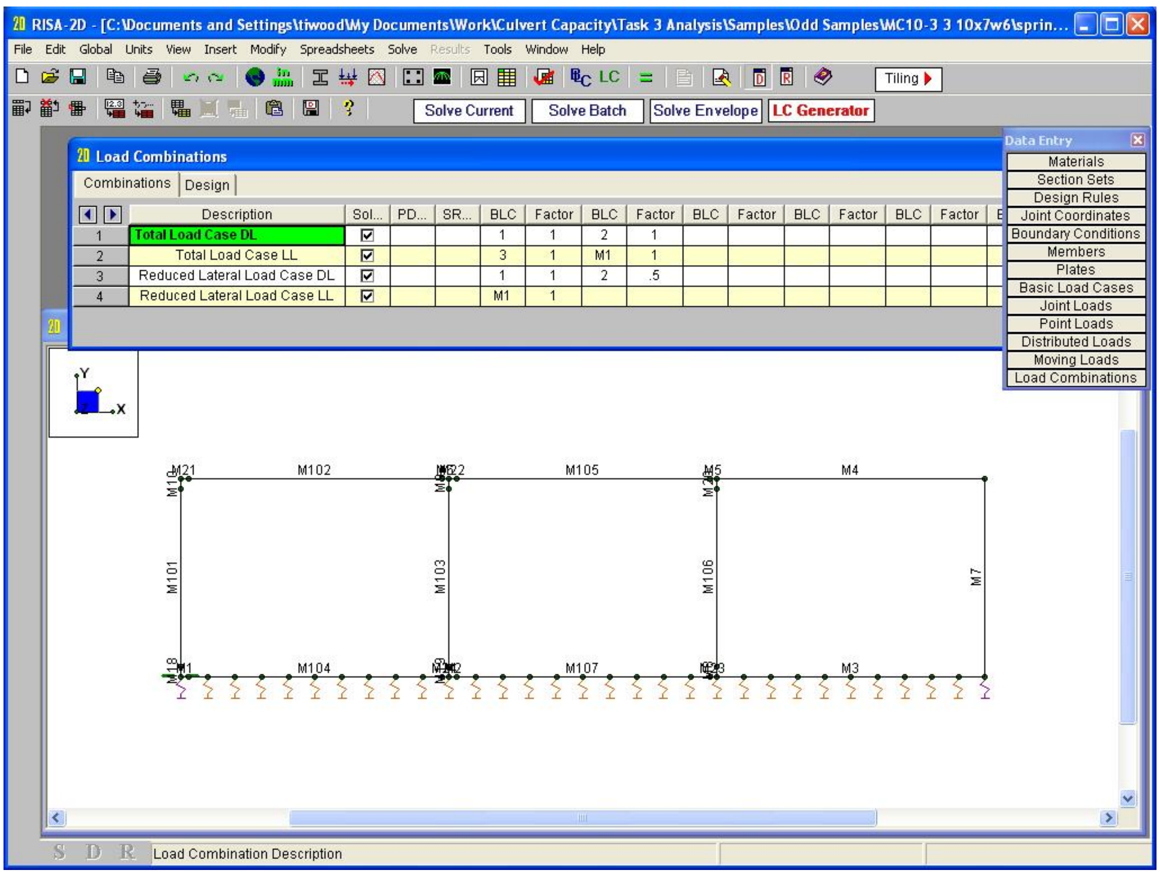


FIGURE IX.24. RISA-2D – CREATION OF LOAD COMBINATIONS

RISA-2D Spring Step 7. Use RISA-2D to solve for moment, shear and axial demand, dead and live loads separately. This will require four separate computer runs, one for each load combination.

Figure IX-25 shows the “solve” command to perform the demand calculations. This particular image applies to the Total Load Case for dead load. Three more computer runs will have to be made: Total Load Case live load, Reduced Lateral Load case dead load, and Reduced Lateral Load case live load.

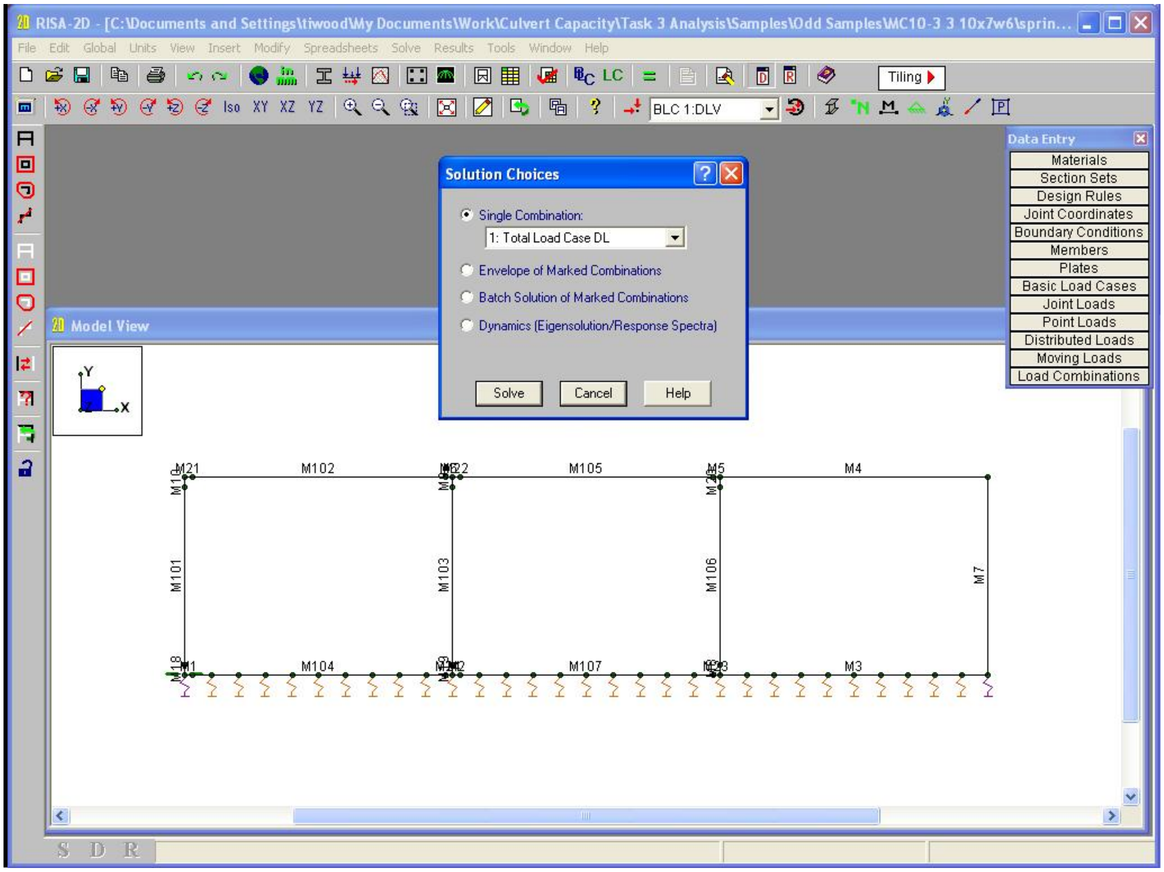


FIGURE IX.25. RISA-2D – SOLVING FOR DEAD LOAD DEMAND, TOTAL LOAD CASE.

Figure IX-26 illustrates the RISA output from these computer runs. This particular image shows RISA member output for Total Load Case live load calculations. Note that this output includes both maximum and minimum values at each section (node) of the model. All four load combinations will have to be performed. This level of output facilitates subsequent data analysis.

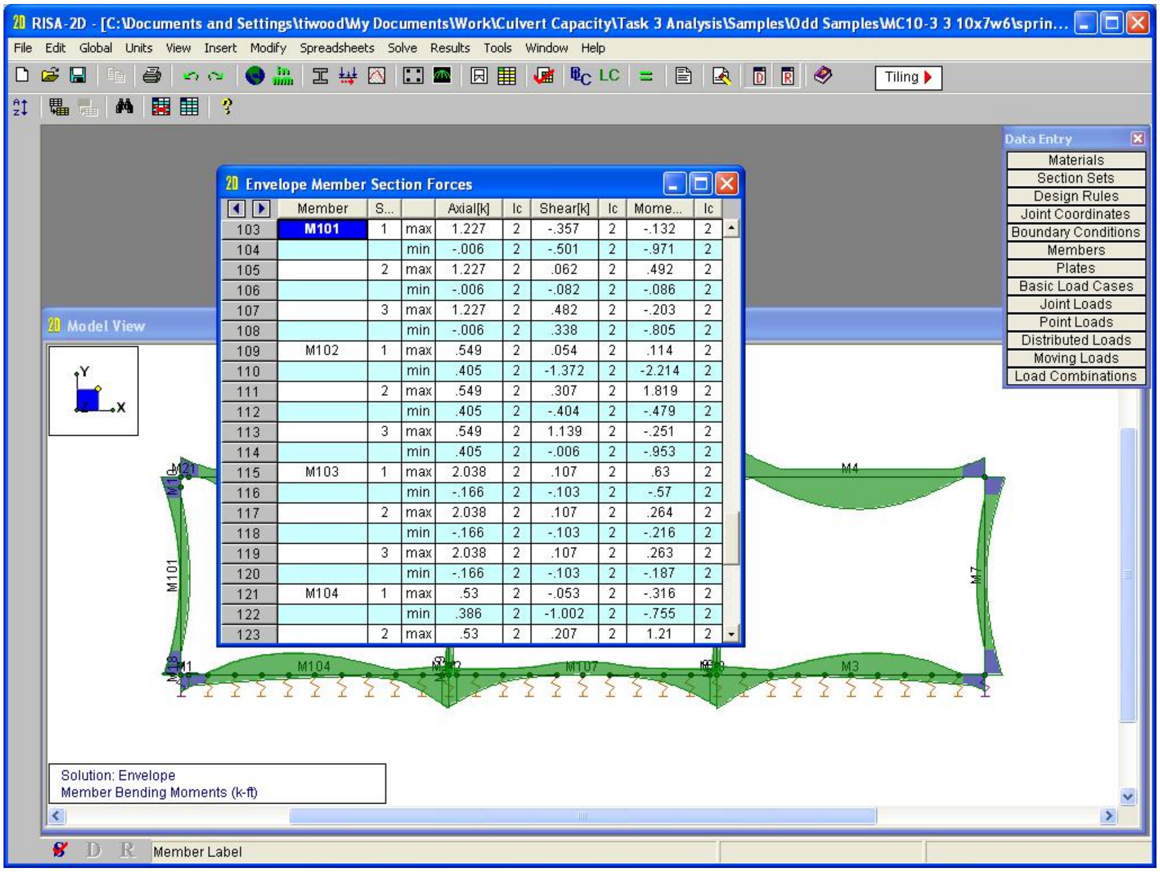


FIGURE IX.26. RISA-2D – OUTPUT SHOWING MEMBER SECTION FORCES FOR DETERMINING LIVE LOAD DEMAND.

RISA-2D Spring Step 8. Record the dead load and the maximum and minimum live load demands for each critical section for both load cases from the member forces table.

These data are obtained from the RISA member section force table illustrated in Figure IX-26. Table IX-16 and Table IX-17 summarize data for each critical section, for each load case (total and reduced lateral), for each demand type (moment, shear and thrust). Note that because the live load data are form envelope solutions, these include both maximum and minimum values.

RISA-2D Spring Step 9. After determining the demands, use Equation VI-1 to verify that actual thrust demand is lower than the incidental axial load assumed in the moment capacity equations. This is shown in the far right hand column of Table IX-16 and Table IX-17

TABLE IX-16. RISA-2D TOTAL LOAD CASE DEMANDS.

			Total Load Case Demands									
Member	*Sections*		*M_D (k-ft/ft)*	*M_L (max) (k-ft/ft)*	*M_L (min) (k-ft/ft)*	*V_D (klf)*	*V_L (max) (klf)*	*V_L (min) (klf)*	*P_D (klf)*	*P_L (max) (klf)*	*P_L (min) (klf)*	*Thrust Check*
M101	1	WBEC	-3.509	-0.203	-0.805	2.462	0.482	0.338	-4.546	0.006	-1.227	OK
	2	WEM	0.471	0.492	-0.086	-0.062	0.062	-0.082	-4.251	0.006	-1.227	OK
	3	WTEC	-3.092	-0.132	-0.971	-1.854	-0.357	-0.501	-3.955	0.006	-1.227	OK
M102	4	TEC	-2.764	-0.251	-0.953	3.679	1.139	-0.006	-2.013	-0.405	-0.549	OK
	5	TEM	5.198	1.819	-0.479	-0.495	0.307	-0.404	-2.013	-0.405	-0.549	OK
	6	TIC1	-7.711	0.114	-2.214	-4.669	0.054	-1.372	-2.013	-0.405	-0.549	OK
M103	7	WBIC1	-0.103	0.263	-0.187	0.099	0.107	-0.103	-9.952	0.166	-2.038	OK
	8	WIM1	0.242	0.264	-0.216	0.099	0.107	-0.103	-9.656	0.166	-2.038	OK
	9	WTIC1	0.586	0.630	-0.570	0.099	0.107	-0.103	-9.361	0.166	-2.038	OK
M104	10	BEC	-3.454	-0.316	-0.755	-3.832	-0.053	-1.002	-2.797	0.386	-0.530	OK
	11	BEM	4.716	1.210	-0.172	0.786	0.207	-0.111	-2.797	-0.386	-0.530	OK
	12	BIC1	-6.952	0.466	-1.696	4.388	0.972	-0.111	-2.797	-0.386	-0.530	OK
M105	13	TIC2	-7.228	0.244	-2.338	4.174	1.328	-0.130	-1.914	-0.374	-0.580	OK
	14	TIM1	3.206	1.549	-0.432	0.000	0.388	-0.388	-1.914	-0.374	-0.580	OK
	15	TIC3	-7.228	0.244	-2.338	-4.174	0.130	-1.328	-1.914	-0.374	-0.580	OK
M107	19	BIC2	-7.104	0.352	-1.608	-4.355	0.056	-0.889	-2.896	-0.355	-0.561	OK
	20	BIM1	3.780	0.751	-0.002	0.470	0.118	-0.118	-2.896	-0.355	-0.561	OK
	21	BIC3	-7.104	0.352	-1.608	4.355	0.889	-0.056	-2.896	-0.355	-0.561	OK

TABLE IX-17. RISA-2D REDUCED LATERAL LOAD CASE DEMANDS.

			Reduced Lateral Load Case Demands									
Member	*Sections*		*M_D (k-ft/ft)*	*M_L (max) (k-ft/ft)*	*M_L (min) (k-ft/ft)*	*V_D (klf)*	*V_L (max) (klf)*	*V_L (min) (klf)*	*P_D (klf)*	*P_L (max) (klf)*	*P_L (min) (klf)*	*Thrust Check*
M101	1	WBEC	-2.780	0.084	-0.516	1.269	0.061	-0.081	-4.397	0.065	-1.168	OK
	2	WEM	-0.658	0.052	-0.527	0.007	0.061	-0.081	-4.101	0.065	-1.168	OK
	3	WTEC	-2.307	0.167	-0.682	-0.889	0.061	-0.081	-3.806	0.065	-1.168	OK
M102	4	TEC	-1.620	0.191	-0.504	3.530	1.082	-0.065	-0.969	0.061	-0.081	OK
	5	TEM	5.595	1.987	-0.315	-0.644	0.248	-0.463	-0.969	0.061	-0.081	OK
	6	TIC1	-8.060	-0.003	-2.350	-4.818	0.000	-1.431	-0.969	0.061	-0.081	OK
M103	7	WBIC1	-0.128	0.254	-0.195	0.127	0.116	-0.093	-10.101	0.104	-2.098	OK
	8	WIM1	0.317	0.297	-0.186	0.127	0.116	-0.093	-9.806	0.104	-2.098	OK
	9	WTIC1	0.762	0.693	-0.503	0.127	0.116	-0.093	-9.510	0.104	-2.098	OK
M104	10	BEC	-2.268	0.133	-0.302	-3.648	0.016	-0.930	-1.437	0.081	-0.061	OK
	11	BEM	4.992	1.310	-0.045	0.902	0.250	-0.059	-1.437	0.081	-0.061	OK
	12	BIC1	-7.104	0.373	-1.757	4.459	1.000	-0.059	-1.437	0.081	-0.061	OK
M105	13	TIC2	-7.434	0.153	-2.418	4.174	1.328	-0.127	-0.841	0.101	-0.100	OK
	14	TIM1	3.001	1.468	-0.511	0.000	0.388	-0.388	-0.841	0.101	-0.100	OK
	15	TIC3	-7.434	0.153	-2.418	-4.174	0.127	-1.328	-0.841	0.101	-0.100	OK
M107	19	BIC2	-7.293	0.272	-1.679	-4.425	0.047	-0.917	-1.564	0.100	-0.101	OK
	20	BIM1	3.781	0.753	-0.043	0.479	0.118	-0.118	-1.564	0.100	-0.101	OK
	21	BIC3	-7.293	0.272	-1.679	4.425	0.917	-0.047	-1.564	0.100	-0.101	OK

RISA-2D Spring Step 10. This step goes beyond calculation of demand loads and has to do with calculating the culvert load rating. Per the culvert rating flow chart (Figure III-2) proceed to calculate Inventory and Operating rating factors for each critical section, for each demand type, for each load case per Equation II-1. This calculation uses the capacity values for each critical section as determined in Appendix B.

When calculating the rating factors, exercise extreme care regarding the signs for both demands and capacities.

- Live load and capacity must be in the same sign and direction.
- If the live load and dead load are in opposite directions or the calculated rating is negative, a check should be made to insure that the structure has adequate capacity to support the dead load. I.E. $C \geq 1.3D$

RISA-2D Spring Step 11. Select the controlling inventory and operating rating factors for each section. These appear in the two right columns of Table IX-18 and Table IX-19.

RISA-2D Spring Step 12. Select the overall controlling rating factors for the culvert. These appear at the bottom of Table IX-19.

RISA-2D Spring Step 13. If shear controls the load rating, the load rater should perform a less-conservative analysis of the shear failure mode based on shear critical sections as per the provisions in Section VI.C. In this example, the controlling failure mode is moment, so additional shear analysis is not required.

RISA-2D Spring Step 14. Calculate the Inventory and Operating Ratings per Equation II-2. Multiply controlling load rating factor by the truck tractor tonnage (W= 20 tons). These appear at the bottom of Table IX-19.

Summary: Based on a Level 2 analysis using RISA-2D with springs and medium soil, the Inventory Rating is HS-14 while the Operating Rating is HS-23. If the culvert condition is fair this requires no posting. If the condition is poor, the culvert should be posted at the Inventory Level and inspected more frequently than every 24 months.

TABLE IX-18. RISA-2D TOTAL LOAD CASE RATING FACTOR CALCULATIONS.

			(Max) Rating Factors						**(Min) Rating Factors**						**Controlling RF**	
Member	*Sections*		IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_C	ORF_C
M101	1	WBEC	2.98	4.98	4.95	8.27	15416.62	25733.89	0.75	1.26	7.06	11.79	75.39	125.84	0.75	1.26
	2	WEM	1.06	1.76	62.89	104.97	15748.74	26288.28	34.77	58.04	46.64	77.86	77.01	128.55	1.06	1.76
	3	WTEC	6.48	10.82	7.71	12.86	15475.63	25832.39	0.88	1.47	5.49	9.17	75.68	126.32	0.88	1.47
M102	4	TEC	9.89	16.52	3.49	5.83	327.30	546.33	2.61	4.35	1332.79	2224.73	241.45	403.03	2.61	4.35
	5	TEM	0.88	1.46	21.09	35.21	311.53	520.02	10.78	18.00	13.60	22.71	229.82	383.62	0.88	1.46
	6	TIC1	81.84	136.60	166.22	277.46	341.80	570.54	1.38	2.30	2.18	3.64	252.15	420.89	1.38	2.30
M103	7	WBIC1	4.77	7.97	35.54	59.32	532.08	888.16	6.05	10.10	38.07	63.55	43.34	72.34	4.77	7.97
	8	WIM1	3.97	6.63	35.54	59.32	533.14	889.94	6.20	10.34	38.07	63.55	43.43	72.49	3.97	6.63
	9	WTIC1	1.34	2.23	35.54	59.32	534.21	891.72	2.71	4.52	38.07	63.55	43.51	72.63	1.34	2.23
M104	10	BEC	6.55	10.94	65.98	110.14	342.19	571.20	2.74	4.58	3.49	5.83	249.22	416.00	2.74	4.58
	11	BEM	1.56	2.60	27.58	46.03	325.65	543.59	28.35	47.32	56.43	94.19	237.17	395.90	1.56	2.60
	12	BIC1	19.04	31.79	3.65	6.10	360.29	601.41	2.45	4.08	75.87	126.64	262.40	438.01	2.45	4.08
M105	13	TIC2	37.05	61.84	2.77	4.62	370.29	618.10	1.43	2.39	63.79	106.49	238.77	398.57	1.43	2.39
	14	TIM1	1.80	3.01	15.93	26.58	337.51	563.39	9.19	15.34	14.93	24.92	217.64	363.29	1.80	3.01
	15	TIC3	37.05	61.84	66.77	111.45	370.29	618.10	1.43	2.39	2.48	4.14	238.77	398.57	1.43	2.39
M107	19	BIC2	25.47	42.52	156.93	261.95	391.59	653.65	2.52	4.21	3.58	5.98	247.80	413.63	2.52	4.21
	20	BIM1	3.26	5.43	49.98	83.42	353.92	590.78	2157.34	3601.11	51.48	85.93	223.96	373.84	3.26	5.43
	21	BIC3	2.52	42.52	4.02	6.70	391.59	653.65	2.52	4.21	150.03	250.44	247.80	413.63	2.52	4.21

TABLE IX-19. RISA-2D REDUCED LATERAL LOAD CASE RATING FACTORS.

Member	*Sections*		IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_C	ORF_C
	1	WBEC	33.08	55.21	50.85	84.87	1424.45	2377.73	2.02	3.37	57.06	95.25	79.27	132.32	2.02	3.37
M101	2	WEM	23.00	38.39	63.24	105.56	1455.11	2428.92	4.39	7.33	47.73	79.67	80.98	135.17	4.39	7.33
	3	WTEC	14.94	24.94	72.04	120.25	1429.89	2386.82	1.94	3.25	41.10	68.61	79.57	132.83	1.94	3.25
	4	TEC	29.74	49.64	3.76	6.27	2183.29	3644.42	6.29	10.50	121.65	203.07	1644.21	2744.56	3.76	6.27
M102	5	TEM	0.68	1.14	26.47	44.19	2078.63	3469.71	17.15	28.63	11.68	19.49	1565.39	2612.99	0.68	1.14
	6	TIC1	948.45	1583.18	NA	NA	2279.58	3805.15	1.21	2.02	2.03	3.39	1716.72	2865.60	1.21	2.02
	7	WBIC1	5.00	8.35	32.64	54.48	848.42	1416.20	5.73	9.56	42.34	70.68	42.06	70.20	5.00	8.35
M103	8	WIM1	3.38	5.64	32.64	54.48	850.12	1419.04	7.44	12.41	42.34	70.68	42.14	70.34	3.38	5.64
	9	WTIC1	1.06	1.77	32.64	54.48	851.82	1421.89	3.28	5.47	42.34	70.68	42.23	70.48	1.06	1.77
	10	BEC	45.63	76.16	522.77	872.63	1640.75	2738.78	9.21	15.37	3.88	6.47	2178.69	3636.74	3.88	6.47
M104	11	BEM	1.31	2.19	22.55	37.65	1561.93	2607.21	112.02	186.98	107.34	179.18	2074.03	3462.04	1.31	2.19
	12	BIC1	24.04	40.12	3.51	5.86	1727.02	2882.79	2.31	3.86	143.46	239.46	2293.25	3827.97	2.31	3.86
	13	TIC2	59.89	99.97	2.77	4.62	1377.54	2299.42	1.33	2.22	65.30	109.00	1391.31	2322.42	1.33	2.22
M105	14	TIM1	1.98	3.31	15.93	26.58	1256.17	2096.84	7.53	12.57	14.93	24.92	1268.73	2117.80	1.98	3.31
	15	TIC3	59.89	99.97	68.34	114.08	1377.54	2299.42	1.33	2.22	2.48	4.14	1391.31	2322.42	1.33	2.22
	19	BIC2	33.38	55.72	187.87	313.60	1398.12	2333.79	2.35	3.92	3.43	5.72	1384.28	2310.68	2.35	3.92
M107	20	BIM1	3.25	5.42	49.93	83.35	1264.40	2110.57	100.36	167.52	51.52	86.00	1251.88	2089.68	3.25	5.42
	21	BIC3	33.38	55.72	3.85	6.42	1398.12	2333.79	2.35	3.92	179.65	299.88	1384.28	2310.68	2.35	3.92

Controlling Rating Factor	TEM - M(max) - RLL	0.68	1.14
Load Rating	*(HS equivalent)*	14	23

Appendix E. LEVEL 3: RISA-2D WITH LEFE EXAMPLE

RISA-2D LEFE Step 1. Modify the model created for the Level 2 analysis to match Figure VI-20 through the following steps:

a. Remove all Level 2 boundary conditions and loads. Figure IX-27 shows the culvert model without loads or boundary conditions.

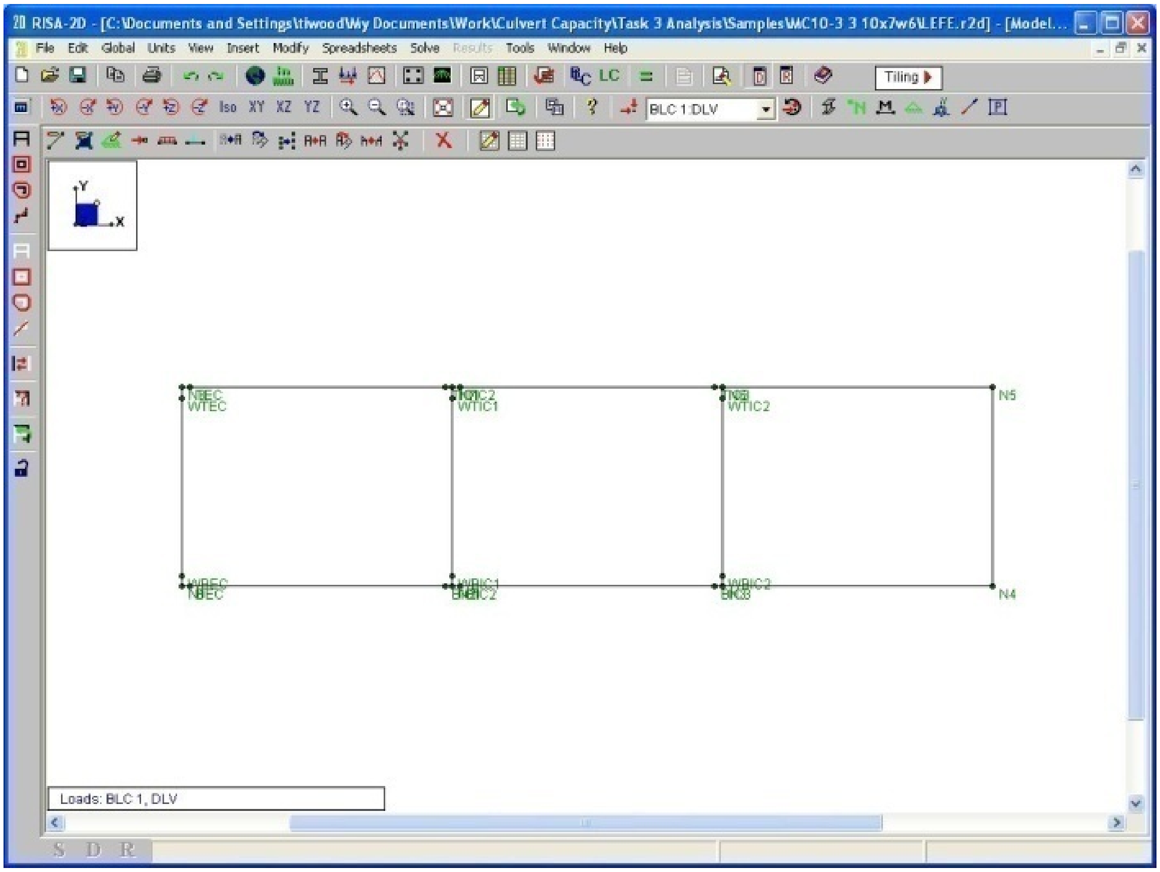


FIGURE IX.27. RISA-2D WITH LEFE REMOVE BC AND LOADS.

b. Place new nodes at the outside corners of the soil area as well as at the edges directly above, below and outside the outside corners of the culvert according to Figure VI-20. Figure IX-28 shows the new node locations

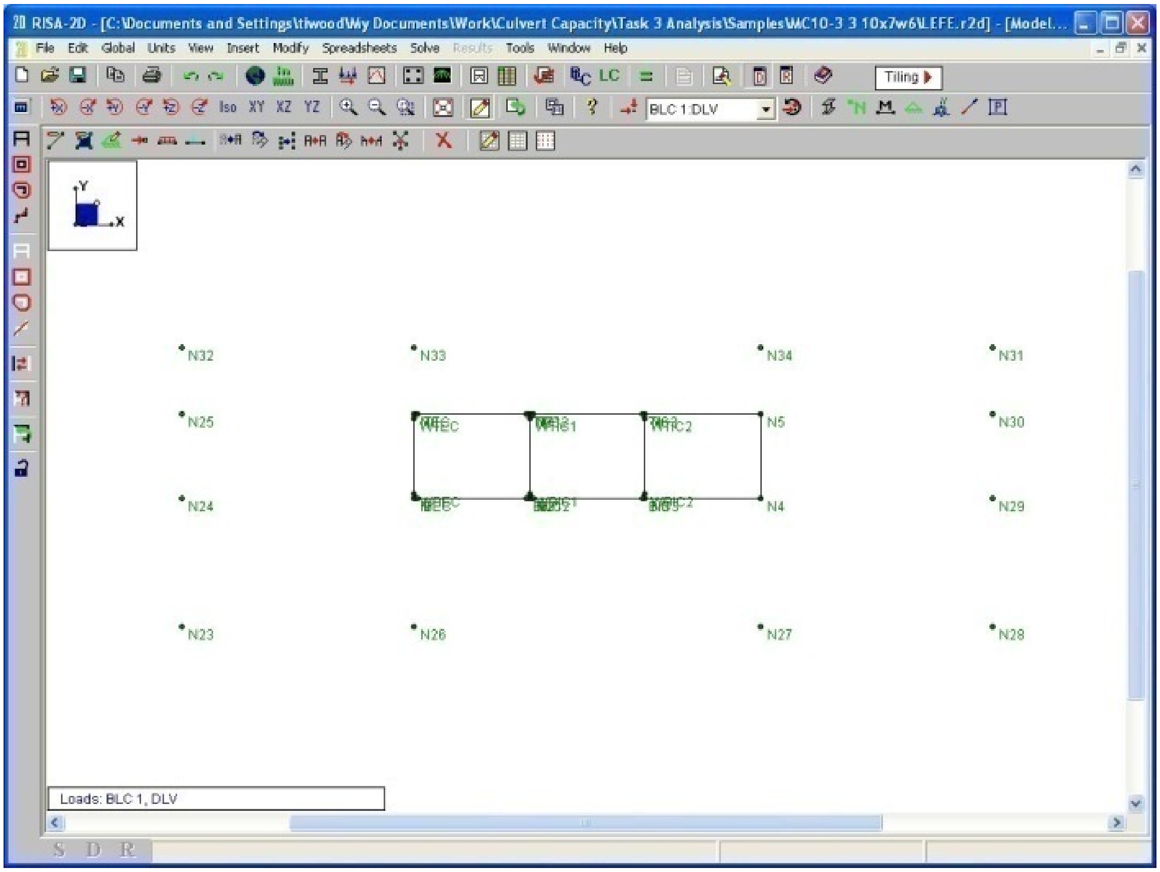


FIGURE IX.28. RISA-2D WITH LEFE NEW NODE LOCATIONS.

c. Connect the nodes from RISA-2D LEFE Step 1.b using the plate drawing tool to make eight large soil elements surrounding the culvert and filling the soil area.

i. The elements should have the material properties from Table IV-3. Figure IX-29 illustrates how to define a new material property.

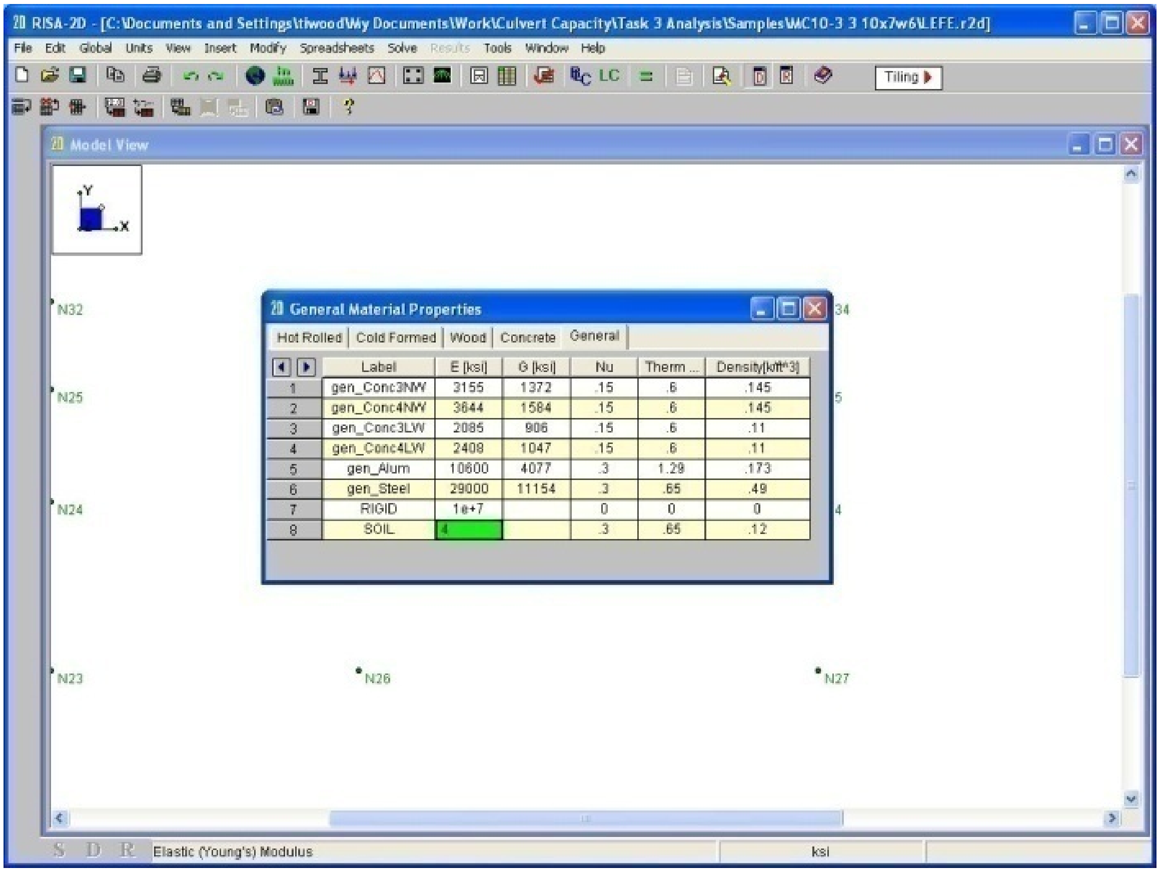


FIGURE IX.29. RISA-2D WITH LEFE SOIL MATERIAL PROPERTY DEFINITIONS.

ii. The elements should be 12 in. thick. Figure IX-30 shows where to define the plate properties. Figure IX-31 shows the culvert with the drawn plates.

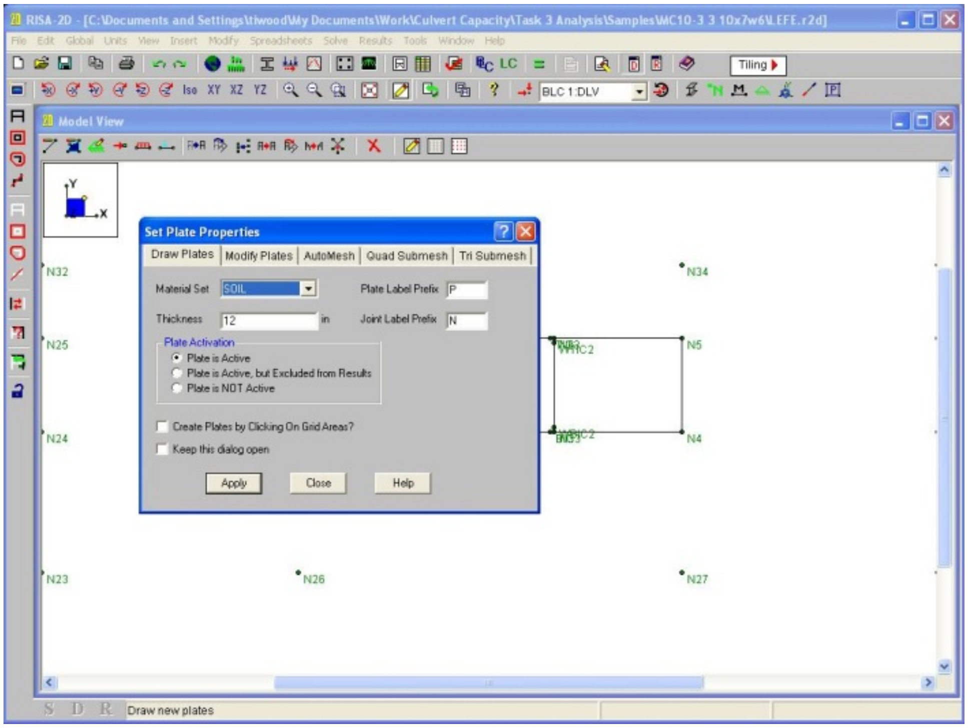


FIGURE IX.30. RISA-2D WITH LEFE DEFINE PLATE PROPERTIES.

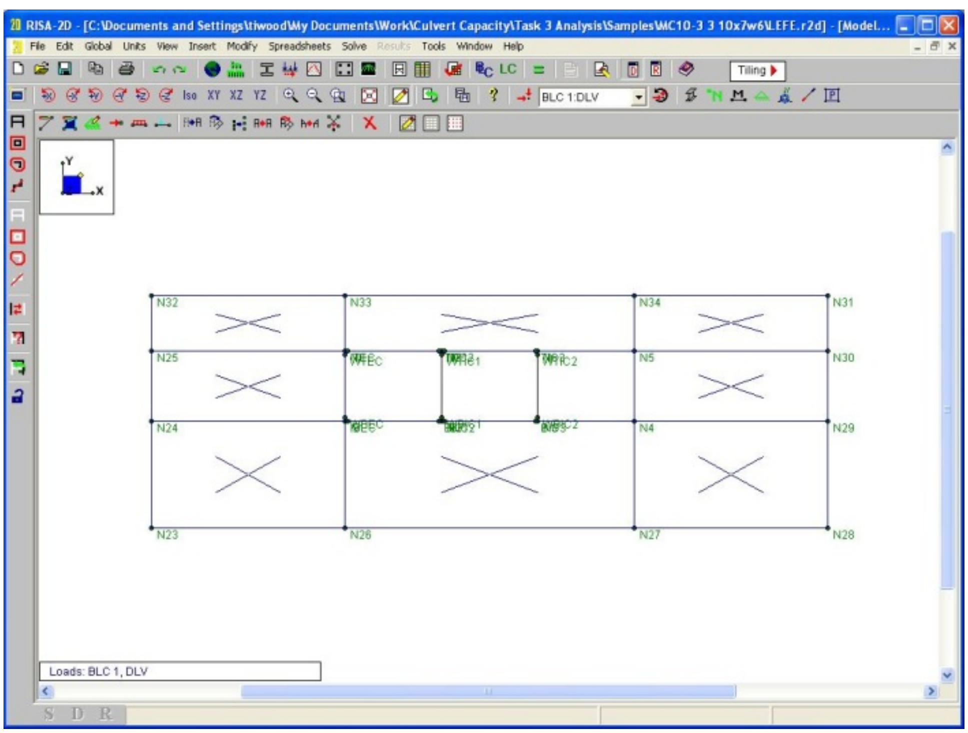


FIGURE IX.31. RISA-2D WITH LEFE DRAW PLATES.

d. Use the “submesh” tool to automatically submesh the large plates. Be sure to specify a minimum of 10 elements along each culvert span. Figure IX-32 shows where to define the extent of the submeshing. Figure IX-33 shows the submeshed soil-structure interaction model.

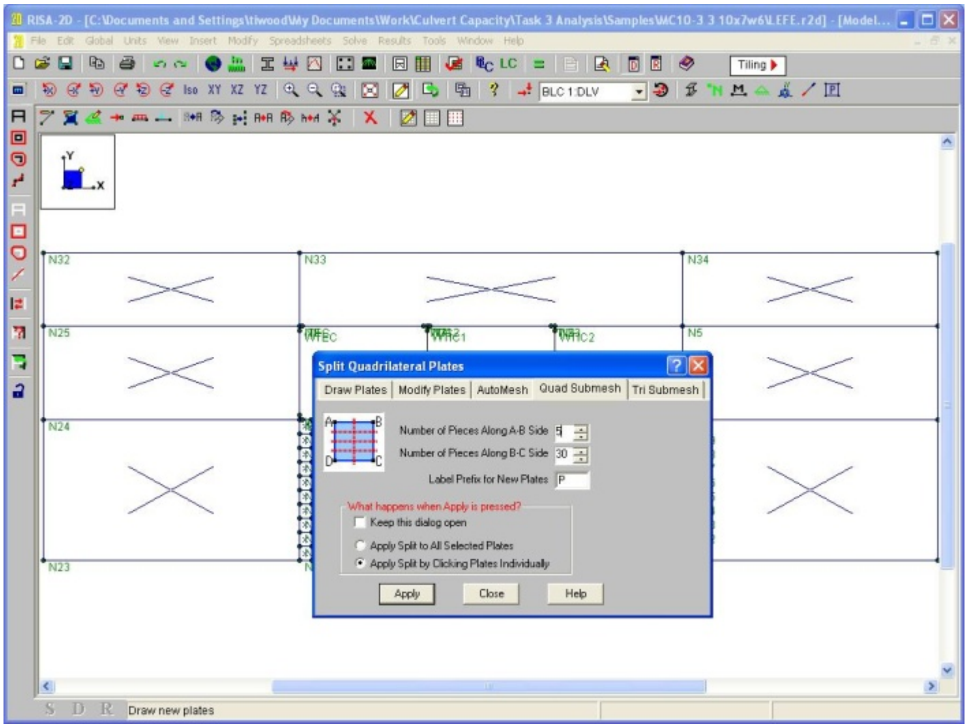


FIGURE IX.32. RISA-2D WITH LEFE PLATE SUBMESHING CONTROLS.

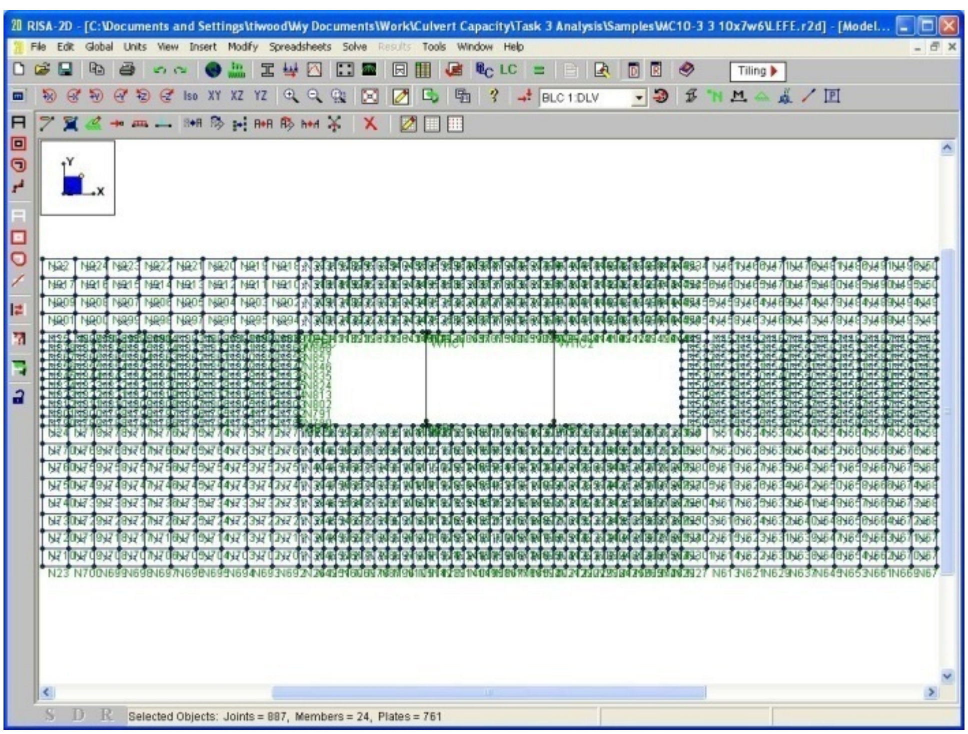
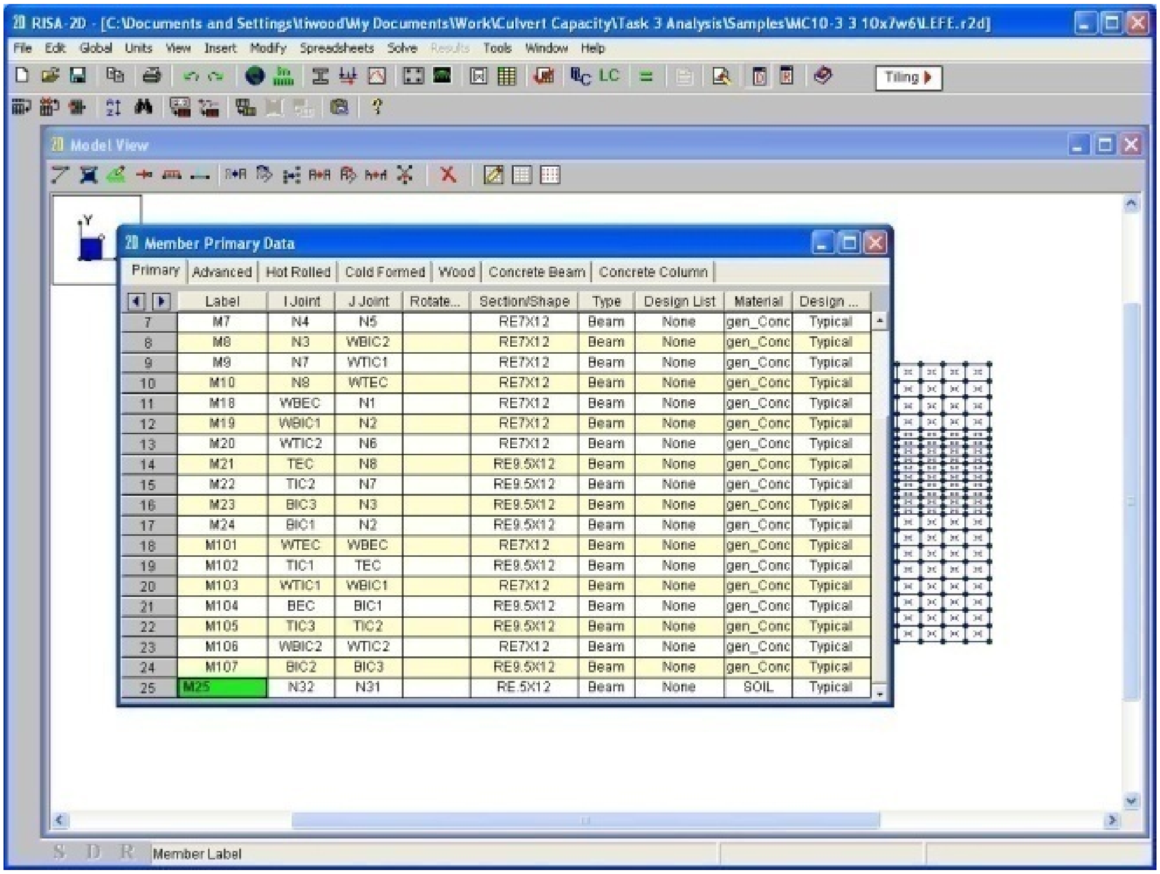


FIGURE IX.33. RISA-2D WITH LEFE PLATE SUBMESH.

e. Create a thin "soil" beam at the ground surface, running from the top left corner of the soil area to the top right corner. Figure IX-34 shows where the soil "member" is defined. This allows RISA to run a moving load across the top surface.



	Label	I Joint	J Joint	Rotate...	Section/Shape	Type	Design List	Material	Design ...
7	M7	N4	N5		RE7X12	Beam	None	gen_Conc	Typical
8	M8	N3	WBIC2		RE7X12	Beam	None	gen_Conc	Typical
9	M9	N7	WTIC1		RE7X12	Beam	None	gen_Conc	Typical
10	M10	N8	WTEC		RE7X12	Beam	None	gen_Conc	Typical
11	M18	WBEC	N1		RE7X12	Beam	None	gen_Conc	Typical
12	M19	WBIC1	N2		RE7X12	Beam	None	gen_Conc	Typical
13	M20	WTIC2	N6		RE7X12	Beam	None	gen_Conc	Typical
14	M21	TEC	N8		RE9.5X12	Beam	None	gen_Conc	Typical
15	M22	TIC2	N7		RE9.5X12	Beam	None	gen_Conc	Typical
16	M23	BIC3	N3		RE9.5X12	Beam	None	gen_Conc	Typical
17	M24	BIC1	N2		RE9.5X12	Beam	None	gen_Conc	Typical
18	M101	WTEC	WBEC		RE7X12	Beam	None	gen_Conc	Typical
19	M102	TIC1	TEC		RE9.5X12	Beam	None	gen_Conc	Typical
20	M103	WTIC1	WBIC1		RE7X12	Beam	None	gen_Conc	Typical
21	M104	BEC	BIC1		RE9.5X12	Beam	None	gen_Conc	Typical
22	M105	TIC3	TIC2		RE9.5X12	Beam	None	gen_Conc	Typical
23	M106	WBIC2	WTIC2		RE7X12	Beam	None	gen_Conc	Typical
24	M107	BIC2	BIC3		RE9.5X12	Beam	None	gen_Conc	Typical
25	M25	N32	N31		RE.5X12	Beam	None	SOIL	Typical

FIGURE IX.34. RISA-2D WITH LEFE SOIL "BEAM".

f. Set the boundary conditions for the outside edge of soil mesh. Figure IX-35 shows how to define the boundary conditions.

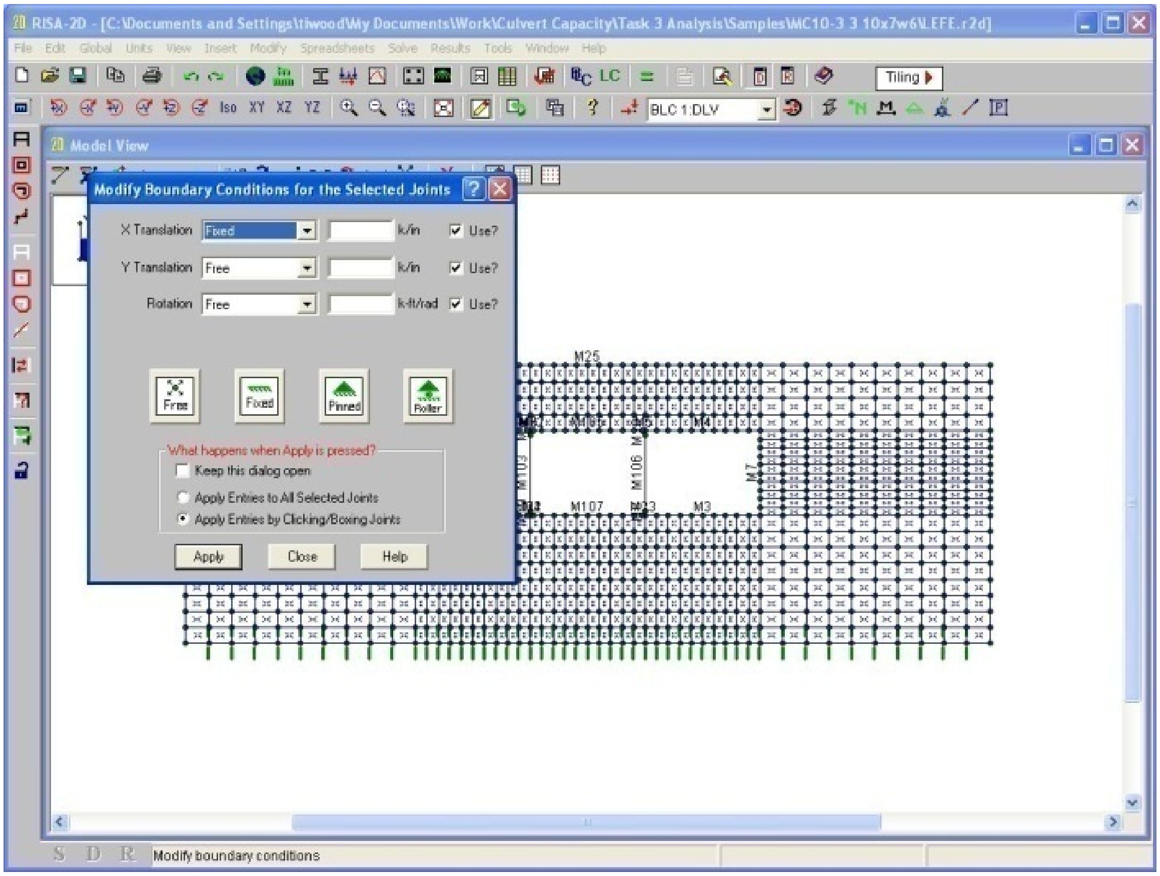


FIGURE IX.35. RISA-2D WITH LEFE BOUNDARY CONDITIONS.

RISA-2D LEFE Step 2. Establish the RISA load cases for dead load and live load.

a. The deal load is simply a -1 gravity loading in the Global Y direction Figure IX-36 illustrates this point.

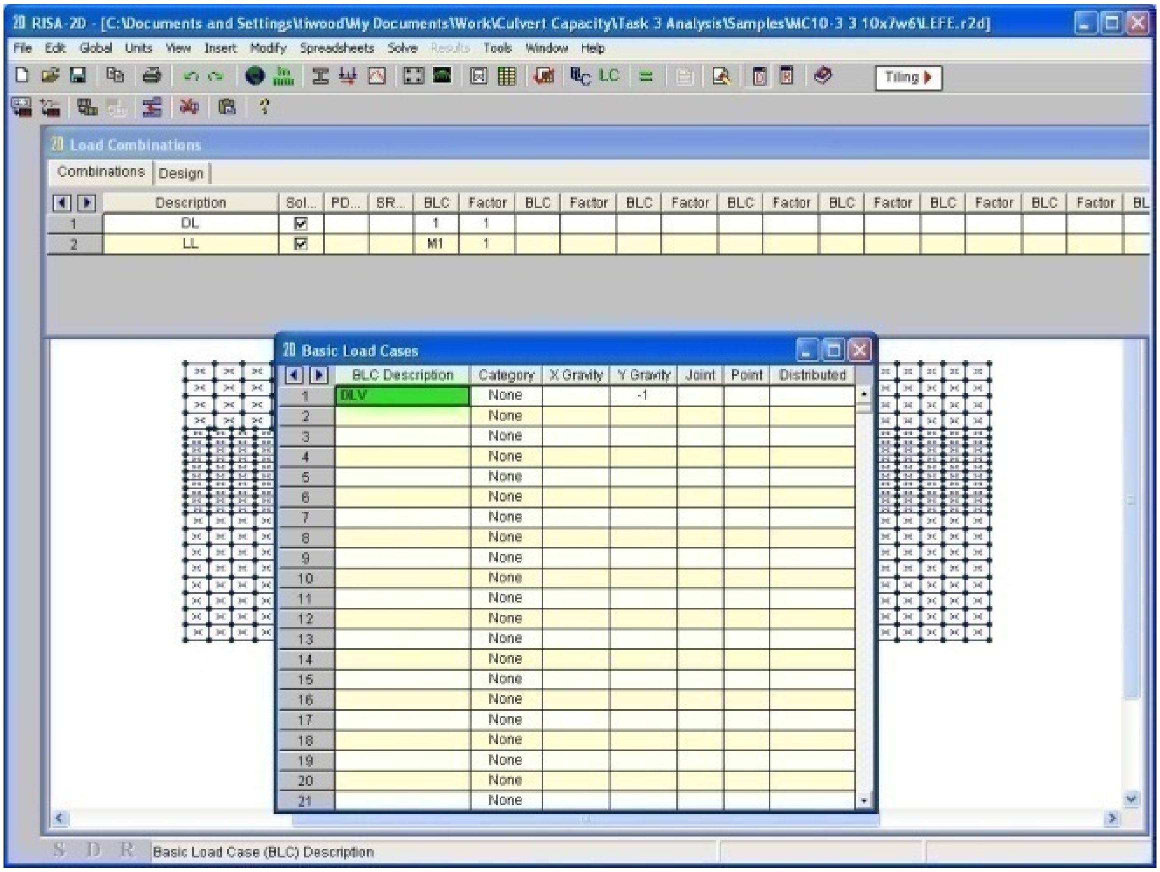


FIGURE IX.36. RISA-2D WITH LEFE BASIC LOAD CASES.

b. The live load is a moving load of magnitude and spacing as illustrated in Figure VI-21 and calculated in Equation VI-12 along the soil "beam" created in step 2.e. Table IX-20 shows the magnitude of the live loads. Figure IX-37 show how to define the moving live load. Figure IX-38 shows the animation of the moving live load. Check the "Both Ways" box to ensure that the load moves from right to left and from left to right.

TABLE IX-20. RISA-2D WITH LEFE LIVE LOAD CALCULATIONS.

Type	*Abbr.*	*Value (klf)*
vertical live load (16k)	LL_{vT}	2.605
vertical live load (4k)	LL_{vT}	0.651

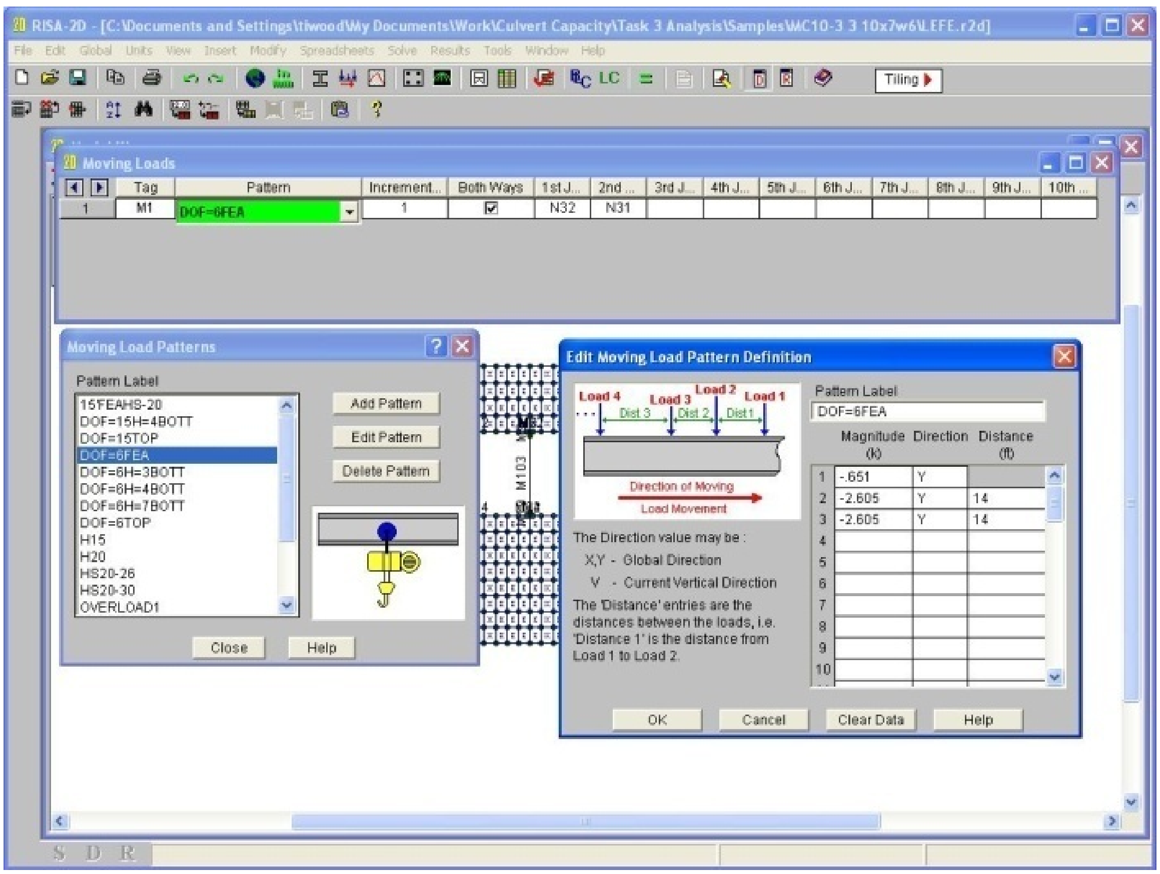


FIGURE IX.37. RISA-2D WITH LEFE MOVING LIVE LOAD DEFINITION.

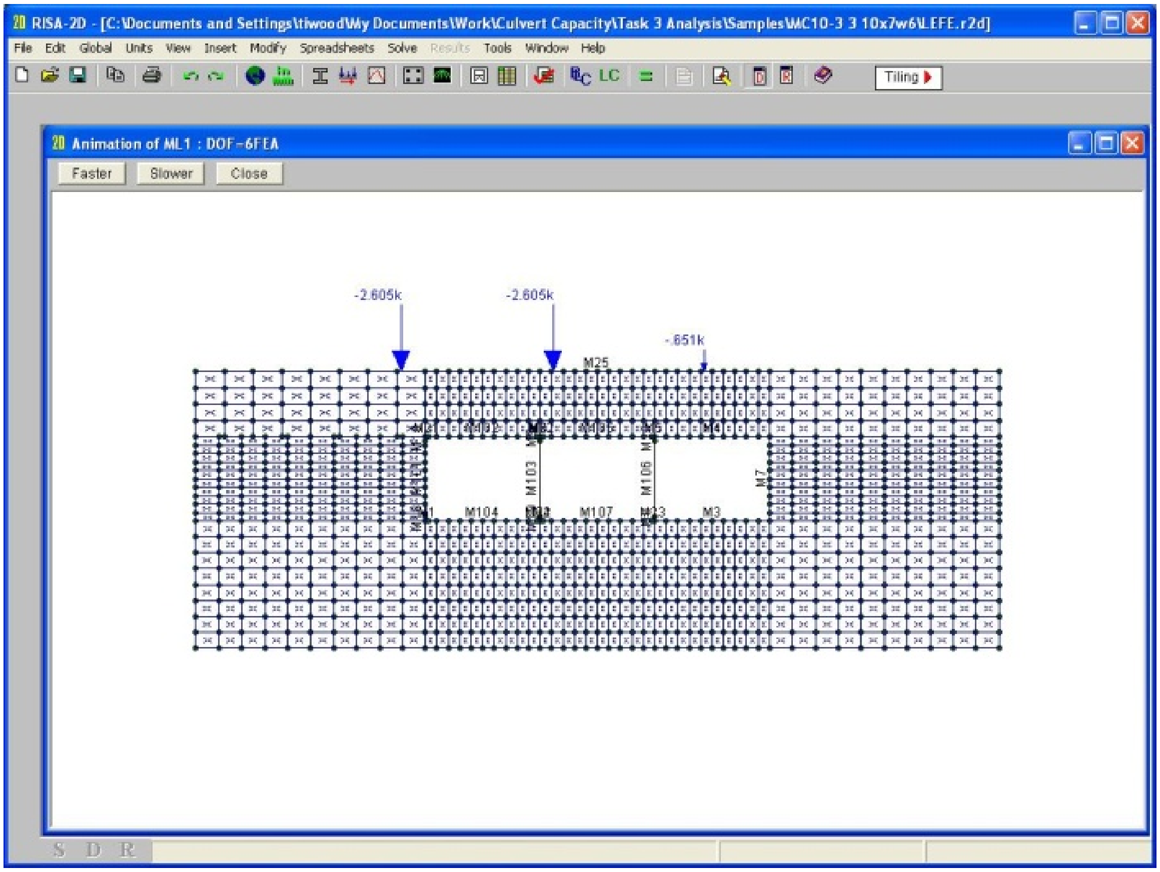


FIGURE IX.38. RISA-2D WITH LEFE MOVING LIVE LOAD ANIMATION.

RISA-2D LEFE Step 3. Use RISA-2D to solve for moment, shear and axial dead and live loads separately. This will require two separate computer runs: one for dead load, one for live load. Figure IX-39 illustrates using the solution box to solve for the dead load demands. Figure IX-40 shows the member results for the dead load demands.

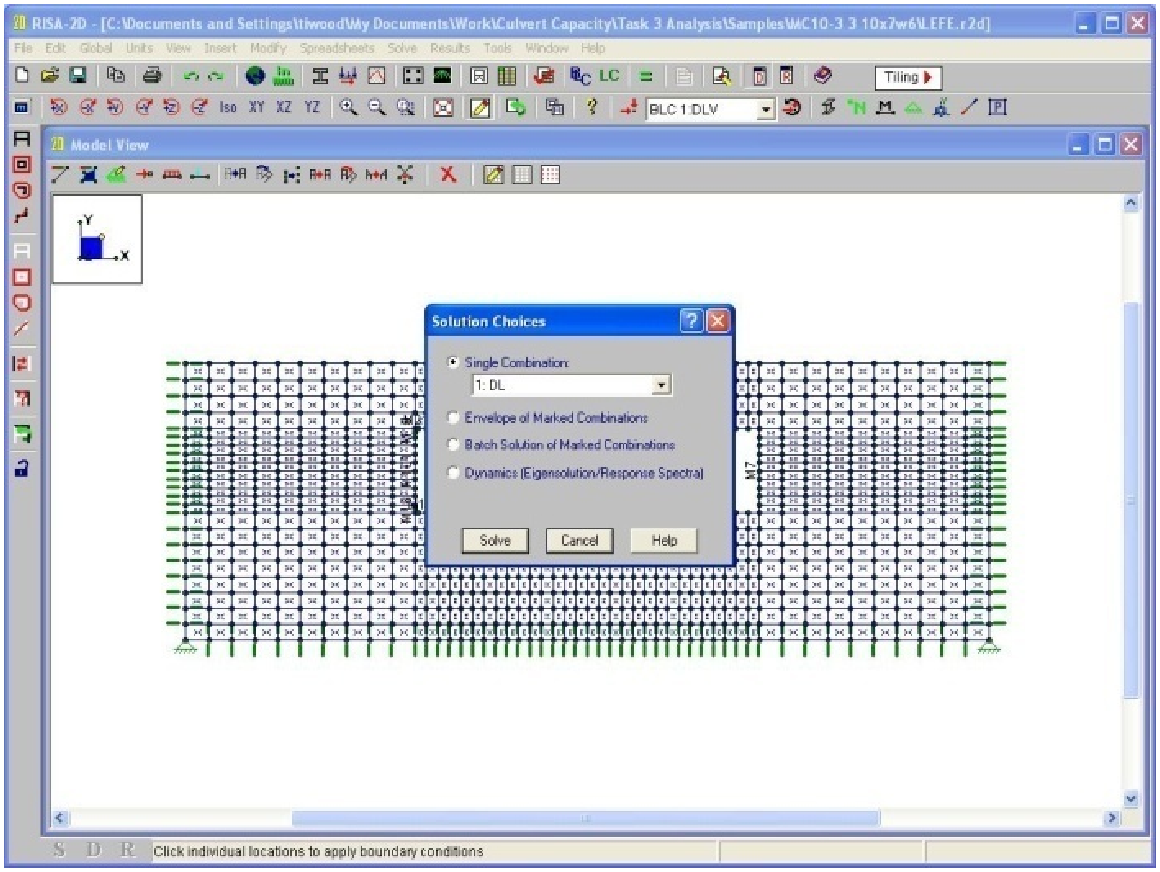
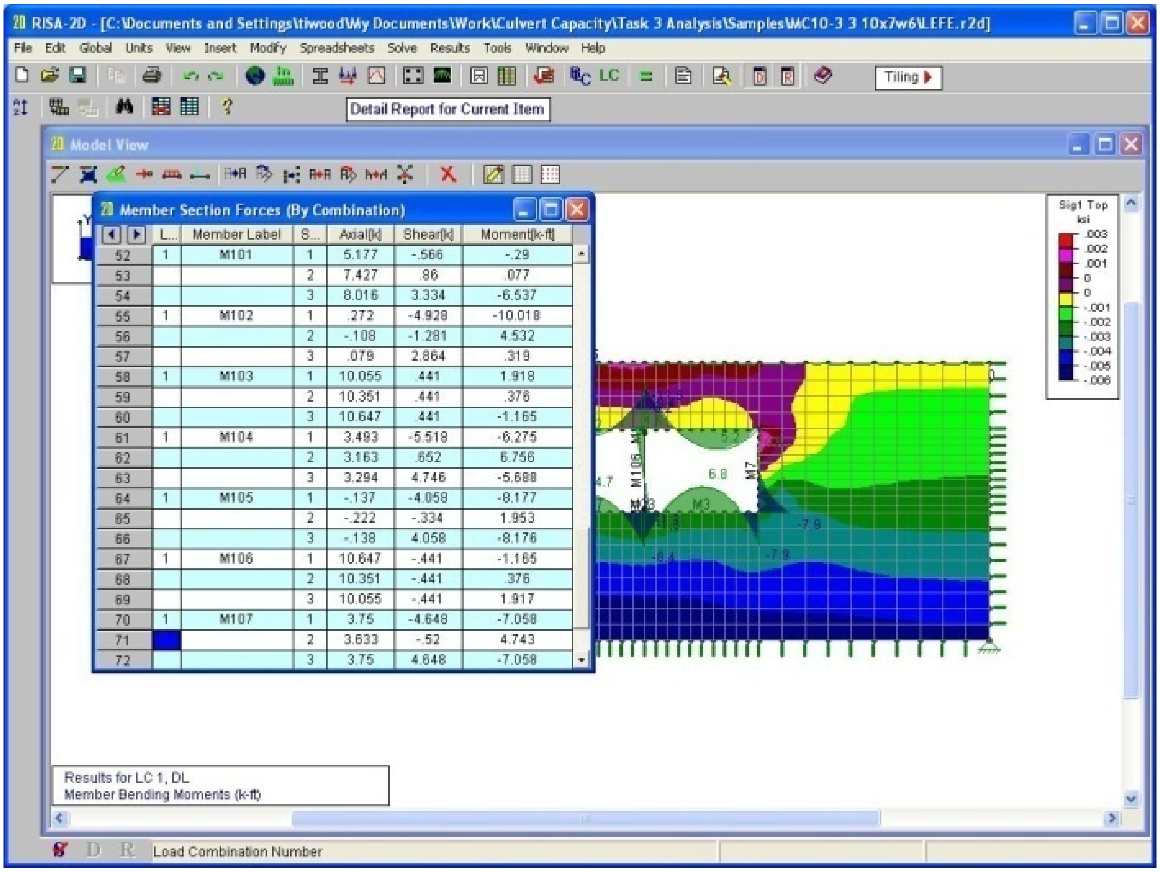


FIGURE IX.39. RISA-2D WITH LEFE SOLVE FOR DEAD LOAD DEMANDS.



	L...	Member Label	S...	Axial[k]	Shear[k]	Moment[k-ft]
52	1	M101	1	5.177	-.566	-.29
53			2	7.427	.86	.077
54			3	8.016	3.334	-6.537
55	1	M102	1	.272	-4.928	-10.018
56			2	-.108	-1.281	4.532
57			3	.079	2.864	.319
58	1	M103	1	10.055	.441	1.918
59			2	10.351	.441	.376
60			3	10.647	.441	-1.165
61	1	M104	1	3.493	-5.518	-6.275
62			2	3.163	.652	6.756
63			3	3.294	4.746	-5.688
64	1	M105	1	-.137	-4.058	-8.177
65			2	-.222	-.334	1.953
66			3	-.138	4.058	-8.176
67	1	M106	1	10.647	-.441	-1.165
68			2	10.351	-.441	.376
69			3	10.055	-.441	1.917
70	1	M107	1	3.75	-4.648	-7.058
71			2	3.633	-.52	4.743
72			3	3.75	4.648	-7.058

FIGURE IX.40. RISA-2D WITH LEFE RESULTS FOR DEAD LOAD DEMANDS.

RISA-2D LEFE Step 4. Record the maximum and minimum demands at each critical section from the member force table.

The data are obtained from the RISA member section force table illustrated in Figure IX-40. Table IX-21 summarizes data for each critical section, for each demand type (moment, shear and thrust). Note that because the live load data are form envelope solutions, these include both maximum and minimum values.

RISA-2D LEFE Step 5. After determining the demands, use Equation VI-1 to verify that actual thrust demand is lower than the incidental axial load assumed in the moment capacity equations. This check is show in the far right column in Table IX-21

TABLE IX-21. RISA-2D WITH LEFE DEMANDS.

Member	*Sections*		*M_D (k-ft/ft)*	*M_L (max) (k-ft/ft)*	*M_L (min) (k-ft/ft)*	*V_D (klf)*	*V_L (max) (klf)*	*V_L (min) (klf)*	*P_D (klf)*	*P_L (max) (klf)*	*P_L (min) (klf)*	*Thrust Check*
M101	1	WBEC	-3.816	0.123	-0.584	2.925	0.369	-0.068	-7.553	0.027	-1.219	OK
	2	WEM	0.575	0.115	-0.050	0.228	0.057	-0.065	-7.192	0.041	-1.425	OK
	3	WTEC	-1.760	0.066	-0.704	-1.358	0.021	-0.370	-5.246	0.041	-1.398	OK
M102	4	TEC	-1.528	0.105	-0.659	2.657	0.826	-0.020	-0.765	0.187	-0.273	OK
	5	TEM	2.879	1.126	-0.067	-0.417	0.174	-0.243	-0.308	0.528	-0.333	OK
	6	TIC1	-6.378	0.018	-1.480	-4.128	0.017	-1.031	-1.251	0.442	-0.418	OK
M103	7	WBIC1	-0.204	0.289	-0.245	0.092	0.102	-0.111	-10.242	0.028	-2.047	OK
	8	WIM1	0.119	0.112	-0.098	0.092	0.102	-0.111	-9.946	0.028	-2.047	OK
	9	WTIC1	0.442	0.470	-0.485	0.092	0.102	-0.111	-9.651	0.028	-2.047	OK
M104	10	BEC	-3.776	0.158	-0.562	-4.187	0.004	-0.605	-2.752	-0.153	-0.357	OK
	11	BEM	3.801	0.579	-0.001	0.527	0.090	-0.016	-1.791	0.284	-0.358	OK
	12	BIC1	-5.796	0.079	-1.315	4.244	0.847	-0.027	-2.451	0.154	-0.355	OK
M105	13	TIC2	-5.956	0.048	-1.419	3.929	1.015	-0.030	-1.221	0.408	-0.386	OK
	14	TIM1	2.339	1.065	-0.076	-0.175	0.214	-0.214	-0.786	0.505	-0.397	OK
	15	TIC3	-5.956	0.048	-1.419	-3.929	0.030	-1.015	-1.221	0.408	-0.386	OK
M107	19	BIC2	-6.036	0.112	-1.385	-4.248	0.020	-0.856	-2.572	0.198	-0.379	OK
	20	BIM1	3.092	0.544	0.000	-0.319	0.055	-0.055	-2.061	0.279	-0.356	OK
	21	BIC3	-6.036	0.112	-1.385	4.247	0.856	-0.020	-2.572	0.198	-0.378	OK

RISA-2D LEFE Step 6. This step goes beyond calculation of demand loads and has to do with calculating the culvert load rating. Per the culvert rating flow chart (Figure III-2) proceed to calculate Inventory and Operating rating factors for each critical section, for each demand type, for each load case per Equation II-1.

When calculating the rating factors, exercise extreme care regarding the signs for both demands and capacities.

a. Live load and capacity must be in the same sign and direction.
b. If the live load and dead load are in opposite directions or the calculated rating is negative, a check should be made to insure that the structure has adequate capacity to support the dead load. I.E. $C \geq 1.3D$

Table IX-22 summarizes this step.

RISA-2D LEFE Step 7. Select the controlling inventory and operating rating factors for each section. This is shown in the far right column of Table IX-22.

RISA-2D LEFE Step 8. Select the overall controlling rating factors for the culvert. This is shown in the bottom right hand corner of Table IX-22.

RISA-2D LEFE Step 9. If shear controls the load rating, the load rater should perform a less-conservative analysis of the shear failure mode based on shear critical sections as per the provisions of Section VI.C. In this example, the controlling failure mode is moment, so additional shear analysis is not required.

RISA-2D LEFE Step 10. Calculate the Inventory and Operating Ratings per Equation II-2. This is shown in the bottom right hand corner of Table IX-22.

Summary: Based on a Level 3 analysis using RISA-2D with LEFE and medium soils, the Inventory Rating is HS-14 while the Operating Rating is HS-24.

TABLE IX-22. RISA-2D WITH LEFE LOAD RATING.

			(Max) Rating Factors						(Min) Rating Factors						Controlling RF	
Member	*Sections*		IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_M	ORF_M	IRF_V	ORF_V	IRF_P	ORF_P	IRF_C	ORF_C
	1	WBEC	27.64	46.13	5.72	9.54	3359.20	5607.27	0.72	1.21	82.56	137.81	74.40	124.20	0.72	1.21
M101	2	WEM	3.98	6.64	65.35	109.09	2261.72	3775.33	61.05	101.90	61.51	102.68	65.07	108.62	3.98	6.64
	3	WTEC	32.84	54.82	222.64	371.63	2245.86	3748.86	2.35	3.92	8.24	13.75	65.87	109.95	2.35	3.92
	4	TEC	53.57	89.43	5.55	9.27	712.85	1189.91	4.89	8.17	369.22	616.32	488.29	815.07	4.89	8.17
M102	5	TEM	2.65	4.43	36.95	61.67	240.89	402.11	56.34	94.05	22.81	38.08	381.96	637.58	2.65	4.43
	6	TIC1	473.93	791.09	508.94	849.53	314.22	524.51	2.60	4.35	3.22	5.37	332.26	554.62	2.60	4.35
	7	WBIC1	4.55	7.60	37.32	62.30	3148.25	5255.15	4.37	7.30	35.29	58.90	43.06	71.88	4.37	7.30
M103	8	WIM1	10.02	16.72	37.32	62.30	3154.58	5265.73	12.90	21.54	35.29	58.90	43.15	72.03	10.02	16.72
	9	WTIC1	1.98	3.30	37.32	62.30	3160.89	5276.26	3.01	5.02	35.29	58.90	43.24	72.17	1.98	3.30
	10	BEC	44.13	73.66	2171.81	3625.26	863.48	1441.35	3.34	5.58	5.43	9.06	370.06	617.72	3.34	5.58
M104	11	BEM	4.20	7.01	65.15	108.74	444.73	742.36	4327.27	7223.21	381.78	637.28	352.80	588.91	4.20	7.01
	12	BIC1	103.57	172.88	4.29	7.17	904.42	1509.69	3.68	6.15	308.71	515.31	392.34	654.91	3.68	6.15
	13	TIC2	172.46	287.87	3.77	6.29	340.45	568.29	2.89	4.83	271.55	453.28	359.85	600.68	2.89	4.83
M105	14	TIM1	3.11	5.19	29.36	49.01	251.30	419.48	45.41	75.81	26.58	44.37	319.66	533.59	3.11	5.19
	15	TIC3	172.46	287.87	284.42	474.77	340.45	568.29	2.89	4.83	3.39	5.66	359.85	600.68	2.89	4.83
	19	BIC2	74.34	124.09	436.19	728.10	703.07	1173.59	3.39	5.66	3.79	6.33	367.30	613.12	3.39	5.66
M107	20	BIM1	5.25	8.77	115.82	193.33	452.12	754.70	NA	NA	101.85	170.01	354.33	591.46	5.25	8.77
	21	BIC3	3.39	5.66	4.25	7.09	703.07	1173.59	3.39	5.66	416.85	695.82	368.28	614.74	3.39	5.66
									Controlling Rating Factor			WBEC - M(min)			0.72	1.21
									Load Rating			*(HS equivalent)*			14	24

Commentary: Figure IX-41 presents the inventory ratings and operating ratings for the example culvert for each level of analysis. As expected, the Level 3 analysis more carefully determines the moment, shear and axial thrust demands and thus yields the highest load rating. Note that the Level 1 analysis using CULV-5 shows that the culvert will require load posting. However, the Level 2 and 3 analyses indicate the culvert may not require load posting depending on the structural condition of the culvert. Per the flow chart in Figure III-1, if the structural condition rating for the culvert is greater than or equal to 5 as determined based on procedures outlined in the TxDOT *Bridge Inspection Manual* (TxDOT 2002), the culvert will not need to be load posted. Otherwise, the culvert would need to be posted at the operating rating level.

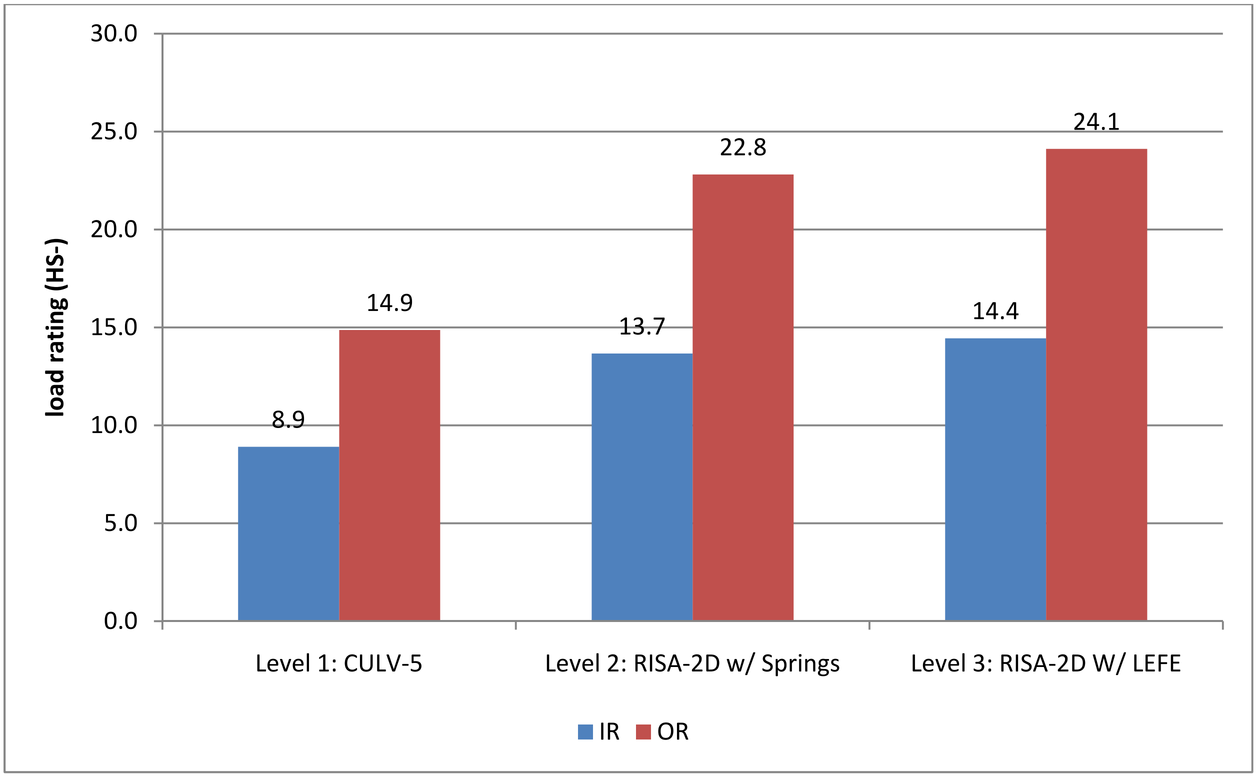


FIGURE IX.41. INVENTORY RATING AND OPERATING RATING FOR MC10-3 3-SPAN 10'X7' WITH 6' FILL.

Careful review of the solutions in Appendices C, D, and E, also shows that for each level of analysis the critical section is different in both location and bending direction. Level 1 shows a positive moment controlling case in the bottom exterior mid-span. Level 2 shows a positive moment controlling case in the top exterior mid-span. However, Level 3 shows a negative moment controlling case in the wall bottom exterior corner. Intuition suggests that this would be the case, that is, the finite element approach yields a more realistic outcome because it models soil-arching effects. Intuition supports this conclusion as well because moment redistribution occurs as the mid-spans crack and become more flexible. This in turn transfers moment load to the corners resulting in greater moments at the exterior corners. This higher order effect could be yet more accurately modeled using non-linear concrete models in a Level 4 analysis. However, because load posting may not be required based on the Level 3 solution, it has been deemed unnecessary to perform a Level 4 analysis.

Appendix F. POLICY SOURCE DOCUMENTS

Manual for Condition Evaluation of Bridges, 1994

Second Edition

As revised by the 1995, 1996, 1998 and 2000 Interim Revisions and as approved by the AASHTO Subcommittee on Bridges and Structures

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6. LOAD RATING

6.1 GENERAL

Bridge load rating calculations provide a basis for determining the safe load capacity of a bridge. Load rating requires engineering judgment in determining a rating value that is applicable to maintaining the safe use of the bridge and arriving at posting and permit decisions. Bridge load rating calculations are based on information in the bridge file including the results of a recent inspection. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or dead load noted during the inspection.

Bridge Owners should implement standardized procedures for determining the load rating of bridges based on this Manual.

This Manual provides a choice of load rating methods. Load ratings at Operating and Inventory levels using the allowable stress method can be calculated and may be especially useful for comparison with past practices. Similarly, load ratings at Operating and Inventory levels based on the load factor method can also be calculated. Each of these rating methods is presented below.

In addition, some Bridge Owners may elect to determine the bridge rating by the load and resistance factor rating method (LRFR). This method is described in the AASHTO *Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges*.

6.1.1 Assumptions

The safe load capacity of a bridge is based on existing structural conditions. To maintain this capacity, it is assumed that the bridges are subject to competent inspections as often as the existing conditions of the structures require, and that sound judgment will be exercised in determining an appropriate safety margin.

6.1.2 Substructure Consideration

Careful attention should be given to all elements of the substructure for evidence of instability which affects the load-carrying capacity of a bridge. Evaluation of the conditions of a bridge's substructure will in many cases be a matter of good engineering judgment.

The adequacy of the substructure should be based on information from as-built plans, construction plans, design calculations, inspection results and other appropriate data. When such information is available, the substructure elements, including piers and abutments, should be checked to ensure that they have at least the capacity of the lowest rated superstructure member. If such information is not available, the substructure should be assumed to be adequate if it is judged by the engineer to be stable after examining the alignment, condition and performance of the substructure elements over time.

6.1.3 Safety Criteria

In general, the safety factors to be used should be taken from this Manual. However, there are some cases where judgment must be exercised in making an evaluation of a structure and the safety factor may be adjusted based on site conditions and/or structure conditions as recorded in the most recent inspection report. This determination most commonly applies to timber which may be of substandard grade or where the material is weathered or otherwise deteriorated. In determining the safety factor for a bridge, consideration should be given to the types of vehicles using the bridge routinely. Every effort should be made to minimize hardships related to economic hauling without jeopardizing the safety of the public.

All data used in the determination of the safety factor should be fully documented.

6.1.4 Application of Standard Design Specifications

For all matters not covered by this Manual, the current applicable AASHTO *Standard Specifications for Highway Bridges* (AASHTO Design Specifications) should be used as a guide. However, there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented. Diagnostic load tests may be helpful in establishing the safe load capacity for such members (see Section 5).

For ease of use and where appropriate, reference is made to specific articles in the AASHTO *Standard Specifications for Highway Bridges*, 14th Edition, 1989 with Interims through 1990.

6.1.5 Nonredundant Structures

There may exist in a structure critical components whose failure would be expected to result in the collapse of the bridge. Special considerations of these nonredundant components may be required in load rating the structure.

6.1.6 Load Rating for Complex Structures

This Manual is intended for use in rating the types of bridges commonly in use in the United States. The computation of the load-carrying capacity of more complex structures, such as suspension bridges, cable-stayed bridges, curved steel girder bridges, arches, continuous trusses, and those bridges with variable girder depth and spacing, requires special analysis methods and procedures. General guidance and direction is available in this Manual, but more complex procedures must be used for the actual determination of the load rating.

6.2 QUALIFICATIONS AND RESPONSIBILITIES

The individual charged with the overall responsibility for load rating bridges should be a licensed professional engineer and preferably have a minimum of 5 years of bridge design and inspection experience. The engineering knowledge and skills necessary to properly evaluate bridges may vary widely depending on the complexity of the bridge involved. The specialized knowledge and skills of other engineers may be needed to ensure proper evaluation.

6.3 RATING LEVELS

Each highway bridge should be load rated at two levels, Inventory and Operating levels.

6.3.1 Inventory Rating Level

The Inventory rating level generally corresponds to the customary design level of stresses but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the Inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load which can safely utilize an existing structure for an indefinite period of time.

6.3.2 Operating Rating Level

Load ratings based on the Operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at Operating level may shorten the life of the bridge.

6.4 RATING METHODS

In the load rating of bridge members, two methods for checking the capacity of the members are provided in this Manual, the Allowable Stress method and Load Factor method.

6.4.1 Allowable Stress (AS)

The allowable or working stress method constitutes a traditional specification to provide structural safety. The actual loadings are combined to produce a maximum stress in a member which is not to exceed the allowable or working stress. The latter is found by taking the limiting stress of the material and applying an appropriate factor of safety.

6.4.2 Load Factor (LF)

The Load Factor method is based on analyzing a structure subject to multiples of the actual loads (factored loads). Different factors are applied to each type of load which reflect the uncertainty inherent in the load calculations. The rating is determined such that the effect of the factored loads does not exceed the strength of the member.

6.5 RATING EQUATION

6.5.1 General

The following general expression should be used in determining the load rating of the structure:

$$RF = \frac{C - A_1D}{A_2L(1 + I)} \qquad (6\text{-}1a)$$

where:

- RF = the rating factor for the live-load carrying capacity. The rating factor multiplied by the rating vehicle in tons gives the rating of the structure (see equation 6-1b)
- C = the capacity of the member (see Article 6.6)
- D = the dead load effect on the member (see Article 6.7.1). For composite members, the dead load effect on the noncomposite section and the dead load effect on the composite section need to be evaluated when the Allowable Stress method is used
- L = the live load effect on the member (see Article 6.7.2)
- I = the impact factor to be used with the live load effect (see Article 6.7.4)
- A_1 = factor for dead loads (see Articles 6.5.2 and 6.5.3)
- A_2 = factor for live load (see Articles 6.5.2 and 6.5.3)

In the equation above "load effect" is the effect of the applied loads on the member. Typical "load effects" used by engineers are axial force, vertical shear force, bending moment, axial stress, shear stress and bending stresses. Once the "load effect" to be evaluated is selected by the engineer, the "capacity" of a member to resist such a load effect may be determined (see Article 6.6).

The Rating Factor (RF) may be used to determine the rating of the bridge member in tons as follows:

$$RT = (RF)W \qquad (6\text{-}1b)$$

where:

- RT = bridge member rating in tons
- W = weight (tons) of nominal truck used in determining the live load effect (L)

The rating of a bridge is controlled by the member with the lowest rating in tons.

6.5.2 Allowable Stress

For the allowable stress method, $A_1 = 1.0$ and $A_2 = 1.0$ in the general rating equation.

The capacity (C) depends on the rating level desired, with the higher value for "C" used for the Operating level. The determination of the nominal capacity of a member is discussed in Article 6.6.2.

6.5.3 Load Factor

For the load factor method, $A_1 = 1.3$ and A_2 varies depending on the rating level desired. For Inventory level, $A_2 = 2.17$ and for Operating level, $A_2 = 1.3$.

The nominal capacity (C) is the same regardless of the rating level desired (see Article 6.6.3).

6.5.4 Condition of Bridge Members

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects when stress is chosen as the evaluation approach, and for the capacity when force or moment is chosen for use in the basic rating equation.

The rating of an older bridge for its load-carrying capacity should be based on a recent thorough field investigation. All physical features of a bridge which have an effect on its structural integrity should be examined as discussed in Section 3. Note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, make a determination of the loss in a cross-sectional area as closely as reasonably possible. Determine if deep pits, nicks or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities below those otherwise permitted or other remedial action may be necessary if such conditions exist.

Size, number, and relative location of bolts and rivets through tension members should be determined and recorded so that the net area of the section can be calculated. Also, in addition to the physical condition, threaded members such as truss rods at turn-buckles should be checked to see if the rod has been upset so that the net area will be properly calculated. This information will normally be taken from plans when they are available, but should be determined in the field otherwise. Any misalignment, bends, or kinks in compression members should be measured carefully. Such defects will have a great effect on the load-carrying capability of a member and may be the controlling factor in the load-carrying capacity of the entire structure. Also, examine the connections of compression members carefully to see if they are detailed such that eccentricities are introduced which must be considered in the structural analysis.

The effective area of members to be used in the calculations shall be the gross area less that portion which has deteriorated due to decay or corrosion. The effective area should be adjusted for rivet or bolt holes in accordance with the AASHTO Design Specifications.

6.5.5 Bridges with Unknown Structural Components

For redundant bridges where necessary details, such as reinforcement in a concrete bridge, are not available from plans or field measurements, a physical inspection of the bridge by a qualified inspector and evaluation by a qualified engineer may be sufficient to approximate Inventory and Operating ratings. Load tests may be helpful in establishing the safe load capacity for such structures (see Section 5).

6.6 NOMINAL CAPACITY (C)

6.6.1 General

The nominal capacity to be used in the rating equation depends on the structural materials, the rating method and rating level used. Nominal capacities based on the Allowable Stress method are discussed in Article 6.6.2 and those based on the Load Factor method are discussed in Article 6.6.3.

The Bridge Owner is responsible for selecting the rating method. The method used should be identified for future reference.

6.6.2 Allowable Stress Method

In the Allowable Stress method, the capacity of a member is based on the rating level evaluated: Inventory level-Allowable Stress, or Operating level-Allowable Stress.

The properties to be used for determining the allowable stress capacity for different materials follow. For convenience, the tables provide, where appropriate, the Inventory, Operating and yield stress values. Allowable stress and strength formulas should be those provided herein or those contained in the AASHTO Design Specifications. When situations arise that are not covered by these specifications, then rational strength of material formulae should be used consistent with data and plans verified in the field investigation. Deviations from the AASHTO Design Specifications should be fully documented.

When the bridge materials or construction are unknown, the allowable stresses should be fixed by the engineer, based on field investigations and/or material testing conducted in accordance with Section 4, and should be substituted for the basic stresses given herein.

6.6.2.1 Structural Steel

The allowable unit stresses used for determining safe load capacity depend on the type of steel used in the structural members. When non-specification metals are encountered, coupon testing may be used to determine a nominal yield point. When information on specifications of the steel is not available, allowable stresses should be taken from the applicable "Date Built" column of Tables 6.6.2.1-1 and 6.6.2.1-2.

Table 6.6.2.1-1 gives allowable Inventory stresses and Table 6.6.2.1-2 gives the allowable Operating stresses for structural steel. The nominal yield stress, F_y, is also shown in Tables 6.6.2.1-1 and 6.6.2.1-2. Tables 6.6.2.1-3 and 6.6.2.1-4 give the allowable Inventory and Operating Stresses for bolts and rivets. For compression members, the effective length (KL) may be determined in accordance with the AASHTO Design Specifications or taken as follows:

KL = 75% of the total length of a column having riveted end connections
= 87.5% of the total length of a column having pinned end connections

The modulus of Elasticity (E) for steel should be 29,000,000 lbs. per sq. in.

If the investigation of shear and stiffener spacing is desirable, such investigation may be based on the AASHTO Design Specifications.

6.6.2.1.1 Combined Stresses

The allowable combined stresses for steel compression members may be calculated by the provisions of AASHTO Design Specifications as modified below or by the procedure contained in Appendix A11.

In using the AASHTO Design Specifications (Article 10.36), the allowable compressive axial stress (F_a) and the allowable compressive bending stresses (F_{bx} and F_{by}) should be based on Tables

TABLE 6.6.2.1-3 Allowable Inventory and Operating Stresses For Low Carbon Steel Bolts and Power Driven Rivets (PSI)

Type of Fastener	Rating Level	Tension	Bearing	Shear: Bearing Type Connection
(A) Low Carbon Steel Bolts: Turned Bolts (ASTM A 307) and Ribbed Bolts[1][2]	INV	18,000	20,000	11,000[3]
	OPR	24,500	27,000	15,000[3]
(B) Power Driven Rivets (rivets driven by pneumatically or electrically operated hammers are considered power driven) Structural Steel Rivet (ASTM A 502 Grade 1 or ASTM A 141)	INV	—	40,000	13,500
	OPR	—	54,500	18,000
Structural Steel Rivet (High Strength) (ASTM A 502 Grade 2)	INV	—	40,000	20,000
	OPR	—	54,500	27,000

(1) The AASHTO Design Specifications indicate that ASTM A 307 bolts shall not be used in connections subject to fatigue.
(2) Based on nominal diameter of bolt.
(3) Threads permitted in the shear plane.

6.6.2.1-1 and 6.6.2.1-2. The safety factor (F.S.) to be used in computing the Euler buckling stress (F'_e) should be as follows:

F.S. = 2.12 at Inventory Level
= 1.70 at Operating Level

6.6.2.1.2 Batten Plate Compression Members

To allow for the reduced strength of batten plate compression members, the actual length of the member shall be multiplied by the following factor to obtain the adjusted value of L/r to be substituted in the compression member formulae discussed in Articles 6.6.2.1 and 6.6.2.1.1.

Actual L/r	FACTOR Spacing center-to-center of batten plates: Up to 2d	4d	6d	10d
40	1.3	2.0	2.8	4.5
80	1.1	1.3	1.7	2.3
120	1.0	1.2	1.3	1.8
160	1.0	1.1	1.2	1.5
200	1.0	1.0	1.1	1.3

d = depth of member perpendicular to battens

For compression members having a solid plate on one side and batten plates on the other, the foregoing factors shall be reduced 50 percent.

Adjusted L/r (batten plate both sides) = Actual L/r x factor.

Adjusted L/r (batten plate one side) = Actual L/r x [1 + 1/2 (factor − 1)].

6.6.2.2 Wrought Iron

Allowable maximum unit stress in wrought iron for tension and bending:

Operating 20,000 psi
Inventory 14,600 psi

Where possible, coupon tests should be performed to confirm material properties used in the rating.

6.6.2.3 Reinforcing Steel

The following are the allowable unit stresses in tension for reinforcing steel. These will ordinarily be used without reduction when the condition of the steel is unknown:

	Stresses (psi): Inventory Rating	Operating Rating	Yield
Structural or unknown grade prior to 1954	18,000	25,000	33,000
Grade 40 billet, intermediate, or unknown grade (after 1954)	20,000	28,000	40,000
Grade 50 rail or hard	20,000	32,500	50,000
Grade 60	24,000	36,000	60,000

6.6.2.4 Concrete

Unit stresses in concrete may be determined in accordance with the Service Load Design Method of the AASHTO Design Specifications (Article 8.15) or be based on the articles below. When the ultimate strength (f'_c) of the concrete is unknown and the concrete is in satisfactory condition, f'_c may be determined from the following table:

Year Built	f'_c (psi)
Prior to 1959	2,500
After 1959	3,000

6.6.2.4.1 Bending

The following maximum allowable bending unit stresses in concrete in lbs/sq. in. may be used:

	Compression Due to Bending f'_c (psi)		
f'_c (psi)	Inventory Level	Operating Level	n
2000–2400	800	1200	15
2500–2900	1000	1500	12
3000–3900	1200	1900	10
4000–4900	1600	2400	8
5000 or more	2000	3000	6

The value of "n" may be varied according to the above table.

6.6.2.4.2 Columns

The determination of the capacity of a compression member based on the AASHTO Design Specifications (Article 8.15.4) results in an Inventory level capacity. The following simplified approach establishes the maximum Operating level capacity:

Maximum safe axial load in columns at Operating rating:

$$P = f_c A_g + f_s A_s \qquad (6\text{-}2)$$

where

P = Allowable axial load on column
f_c = Allowable unit stress of concrete taken from equation 6-3 or 6-4
A_g = Gross area of column
f_s = Allowable stress of steel = 0.55 f_y
f_y = Yield strength of reinforcing steel
A_s = Area of longitudinal reinforcing steel

Compression, short columns, in which L/D is 12 or less:

$$f_c = 0.3\, f'_c \qquad (6\text{-}3)$$

Compression, long columns, in which L/D is greater than 12:

$$f_c = 0.3\, f'_c\,(1.3\text{-}0.03\ L/D) \qquad (6\text{-}4)$$

L = Unsupported length of column
D = Least dimension of column

6.6.2.4.3 Shear (Diagonal Tension)

The Inventory level shear strength should be determined in accordance with the Service Load Design method of the AASHTO Design Specifications (Article 8.15.5).

The Operating level shear strength in beams showing no diagonal tension cracking may be found as follows:

(Total Unit Shear) = (Shear Taken by Steel) + (Shear Taken by Concrete)

or $$v = v_s + v_c \qquad (6\text{-}5)$$

The allowable shear stress carried by the concrete, v_c, may be taken as $1.3\sqrt{f'_c}$. A more detailed calculation of the allowable shear stress can be made using:

$$v_c = 1.25\sqrt{f'_c} + 1{,}600\ \rho_w\ (Vd/M) \leq 2.3\sqrt{f'_c}$$

where d = distance from extreme compression fiber to centroid of tension reinforcement

ρ_w = reinforcement ratio = $A_s/(b_w d)$

b_w = width of the web

Note: (a) M is the moment acting simultaneously with the shear force V at the section being considered

(b) The quantity Vd/M shall not be taken greater than 1.0

Where severe diagonal tension cracking has occurred, v_c should be considered as zero and all shear stress should be taken by the reinforcing steel.

6.6.2.5 Prestressed Concrete

Rating of prestressed concrete members should be based on the criteria presented under section 6.6.6.3.

6.6.2.6 Masonry

Stone, concrete, and clay brick masonry structures should be evaluated using the allowable stress rating method. Mortar used to bind the individual masonry units should be classified in accordance with ASTM C 270.

The allowable Inventory level compressive stresses for masonry assemblies are shown in Table 6.6.2.6. These are minimum values and may be used in the absence of more reliable data such as the results of a prism test conducted in accordance with ASTM E 447. The condition of the masonry unit and mortar should be considered when assigning an allowable stress.

Allowable Operating level stresses for masonry are not included in this Manual. Masonry components should be evaluated at the Inventory level.

Reinforced masonry construction may be evaluated using the allowable unit stresses for reinforcing steel. Article 6.6.2.3 and an appropriate allowable stress in the masonry.

TABLE 6.6.2.6 Allowable Inventory Compressive Stresses for Evaluation of Masonry

Construction: Compressive Strength of Unit, gross area, psi	Allowable Inventory Compressive Stresses gross cross-sectional area, psi: Type M or S Mortar[a]	Type N Mortar[a]
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick		
8000 or greater	350	300
4500	225	200
2400	160	140
1500	115	100
Grouted masonry, of clay or shale; sand-lime or concrete:		
4500 or greater	225	200
2500	160	140
1500	115	100
Solid masonry of solid concrete masonry units:		
3000 or greater	225	200
2000	160	140
1200	115	100
Masonry of hollow load-bearing units:		
2000 or greater	140	120
1500	115	100
1000	75	70
700	60	55
Stone ashlar masonry:		
Granite	720	640
Limestone or marble	450	400
Sandstone or cast stone	360	320
Rubble stone masonry Coarse, rough, or random	120	100

[a] Mortar is classified in accordance with ASTM C 270.

6.6.2.7 Timber

Determining allowable stresses for timber in existing bridges will require sound judgment on the part of the engineer making the field investigation.

(1) Inventory Stress
The Inventory unit stresses should be equal to the allowable stresses for stress-grade lumber given in the AASHTO Design Specifications.

Allowable Inventory unit stresses for timber columns should be in accordance with the applicable provisions of the AASHTO Design Specifications.

(2) Operating Stress
The maximum allowable Operating unit stresses should not exceed 1.33 times the allowable stresses for stress-grade lumber given in the current AASHTO Design Specifications. Reduction from the maximum allowable stress will depend upon the grade and condition of the timber and should be determined at the time of the inspection.

Allowable Operating stress in pounds per square inch of cross-sectional area of simple solid columns should be determined by the following formulae but the allowable Operating stress should not exceed 1.33 times the values for compression parallel to grain given in the design stress table of the AASHTO Design Specifications.

$$\frac{P}{A} = \frac{4.8E}{(l/r)^2} \tag{6-6}$$

in which

P = total load in pounds
A = cross-sectional area in square inches
E = modulus of elasticity
l = unsupported overall length, in inches, between points of lateral support of simple columns
r = least radius of gyration of the section in inches

For columns of square or rectangular cross section, this formula becomes:

$$\frac{P}{A} = \frac{0.40E}{(l/d)^2} \tag{6-7}$$

in which d = dimension in inches of the narrowest face.

The above formula applies to long columns with (l/d) over 11, but not greater than 50.

For short columns, (l/d) not over 11, use the allowable design unit stress in compression parallel to grain times 1.33 for the grade of timber used.

6.6.3 Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in the load factor sections of the AASHTO Design Specifications. Nominal strength calculations should take into consideration the observable effects of deterioration, such as loss of concrete or steel-sectional area, loss of composite action or corrosion.

Allowable fatigue strength should be checked based on the AASHTO Design Specifications. Special structural or operational conditions and policies of the Bridge Owner may also influence the determination of fatigue strength.

6.6.3.1 Structural Steel

The yield stresses used for determining ratings should depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine yield characteristics. The nominal yield value should be substituted in strength formulas and is typically taken as the mean test value minus 1.65 standard deviations. When specifications of the steel are not available, yield strengths should be taken from the applicable "Date Built" column of the tables set forth in Article 6.6.2.1.

The capacity of structural steel members should be based on the load factor requirements stated in the AASHTO Design Specifications. The capacity (C) for typical steel bridge members is summarized in Appendix C. For beams, the overload limitations of Article 10.57 should also be considered.

The Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table 10.56A in the AASHTO Design Specifications.

The Operating rating for friction joint fasteners (A 325 bolts) should be determined using a stress of 21 ksi. A_1 and A_2 should be taken as 1.0 in the basic rating equation.

6.6.3.2 Reinforced Concrete

The following are the yield stresses for reinforcing steel.

Reinforcing Steel	Yield Point F_y (psi)
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Billet or Intermediate Grade and unknown after 1954 (Grade 40)	40,000
Rail or Hard Grade (Grade 50)	50,000
Grade 60	60,000

The capacity of concrete members should be based on the strength requirements stated in AASHTO Design Specifications (Article 8.16). Appendix C contains formulas for the capacity (C) of typical reinforced concrete members. The area of tension steel at yield to be used in computing the ultimate moment capacity of flexural members should not exceed that available in the section or 75 percent of the reinforcement required for balanced conditions.

6.6.3.3 Prestressed Concrete

The rating of prestress concrete members at both Inventory and Operating level, should be established in accordance with the strength requirements of Article 9.17 of the AASHTO Design Specifications. Additionally at Inventory level, the rating must consider the allowable stresses at service load as specified in Article 9.15.2.2 of the AASHTO Design Specifications. In situations of unusual design with wide dispersion of the tendons, the Operating rating might further be controlled by stresses not to exceed 0.90 of the yield point stress in the prestressing steel nearest the extreme tension fibre of the member.

Formulas for the capacity (C) of typical prestressed concrete members are included in Appendix C. A summary of the strength and allowable stress rating equations is presented at the end of this section. More stringent allowable stress values may be established by the Bridge Owner.

Typically, prestressed concrete members used in bridge structures will meet the minimum reinforcement requirements of Article 9.18.2.1 of the AASHTO Design Specifications. While there is no reduction in the flexural strength of the member in the event that these provisions are not satisfied, an owner, as part of the flexural rating, may choose to limit live loads to those that preserve the relationship between ϕM_n and M_{cr} that is prescribed for a new design. The use of this option necessitates an adjustment to the value of the nominal moment capacity ϕM_n used in the flexural strength rating equations. Thus when $\phi M_n < 1.2\ M_{cr}$, the nominal moment capacity becomes $(k)(\phi)(M_n)$,

$$k = \frac{\phi M_n}{1.2\ M_{cr}}$$

Rating Equations:

Inventory Rating:

$$RF = \frac{6\sqrt{f_c'} - (F_d + F_p + F_s)}{F_l}$$

Concrete Tension

$$RF = \frac{0.6 f_c' - (F_d + F_p + F_s)}{F_l}$$

Concrete Compression

$$RF = \frac{0.4 f_c' - \frac{1}{2}(F_d + F_p + F_s)}{F_l}$$

Concrete Compression

$$RF = \frac{0.8 f_y^* - (F_d + F_p + F_s)}{F_l}$$

Prestressing Steel Tension

$$RF = \frac{\phi R_n - (1.3D + S)}{2.17L(1+I)}$$

Flexural and Shear Strength

Operating Rating:

$$RF = \frac{\phi R_n - (1.3D + S)}{1.3L(1+I)}$$

Flexural and Shear Strength

$$RF = \frac{0.9 f_y^* - (F_d + F_p + F_s)}{F_l}$$

Prestressing Steel Tension

Where:

RF = rating factor

f_c' = concrete compressive strength

$6\sqrt{f_c'}$ = allowable concrete tensile stress. A factor of $3\sqrt{f_c'}$ may be applicable, or this allowable stress may be zero, as provided by Article 9.15 of the AASHTO Standard Specifications.

F_d = unfactored dead load stress

F_p = unfactored stress due to prestress force after all losses

F_s = unfactored stress due to secondary prestress forces

F_l = unfactored live load stress including impact

ϕR_n = nominal strength of section satisfying the ductility limitations of Article 9.18 and Article 9.20 of the AASHTO Standard Specifications. Both moment (ϕM_n) and shear (ϕV_n) should be evaluated

D = unfactored dead load moment or shear

S = unfactored prestress secondary moment or shear

L = unfactored live load moment or shear

f_y^* = prestressing steel yield stress

I = impact factor

Note: In the rating equations, effects of dead load, prestress force and secondary prestress forces are subtracted from the allowable stress or capacity. The actual effect of each load relative to the allowable stress or capacity should be considered in the rating equations through using appropriate signs.

6.7 LOADINGS

This section discusses the loads to be used in determining the load effects in the basic rating equation (6-1a).

6.7.1 Dead Load (D)

The dead load effects of the structure should be computed in accordance with the conditions existing at the time of analysis. Minimum unit weight of materials to be used in computing the dead load stresses should be in accordance with current AASHTO Design Specifications.

For composite members, the portion of the dead load acting on the noncomposite section and the portion acting on the composite section should be determined.

Care should be exercised in estimating the weight of concrete decks since significant variations of deck thickness have been found, particularly on bridges built prior to 1965.

Nominal values of dead weight should be based on dimensions shown on the plans with allowances for normal construction tolerances.

The approximate overlay thickness should be measured at the time of the inspection.

6.7.2 Rating Live Load

The live load to be used in the basic rating equation (6-1a) should be the HS20 truck or lane loading as defined in the AASHTO Design Specifications and shown in Figures 6.7.2.1 and 6.7.2.2. Other loadings used by Bridge Owners for posting and permit decisions are discussed in Section 7.

6.7.2.1 Wheel Loads (Deck)

In general, stresses in the deck do not control the load rating except in special cases. The calculation of bending moments in the deck should be in accordance with AASHTO Design Specifications. Wheel loads should be in accordance with the current AASHTO Design Specifications.

6.7.2.2 Truck Loads

The live or moving loads to be applied on the deck for determining the rating should be the Standard AASHTO "HS" loading.

The number of traffic lanes to be loaded, and the transverse placement of wheel lines should be in conformance with the current AASHTO Design Specifications and the following:

Roadway widths from 18 to 20 feet should have two design lanes, each equal to one-half the roadway width. Live loadings should be centered in these lanes.

(2) = Roadway widths less than 18 feet should carry one traffic lane only.

When conditions of traffic movements and volume would warrant it, fewer traffic lanes than specified by AASHTO may be considered.

6.7.2.3 Lane Loads

The Bridge Owner may use the Standard AASHTO HS lane load for all span lengths where it may result in load effects which are greater than those produced by the AASHTO standard HS truck.

6.7.2.4 Sidewalk Loadings

Sidewalk loadings used in calculations for safe load capacity ratings should be the probable maximum loads anticipated. Because of site variations, the determination of loading to be used will require engineering judgment, but in no case should it exceed the value given in AASHTO Design Specifications.

The Operating level should be considered when full truck and sidewalk live loads act simultaneously on the bridge.

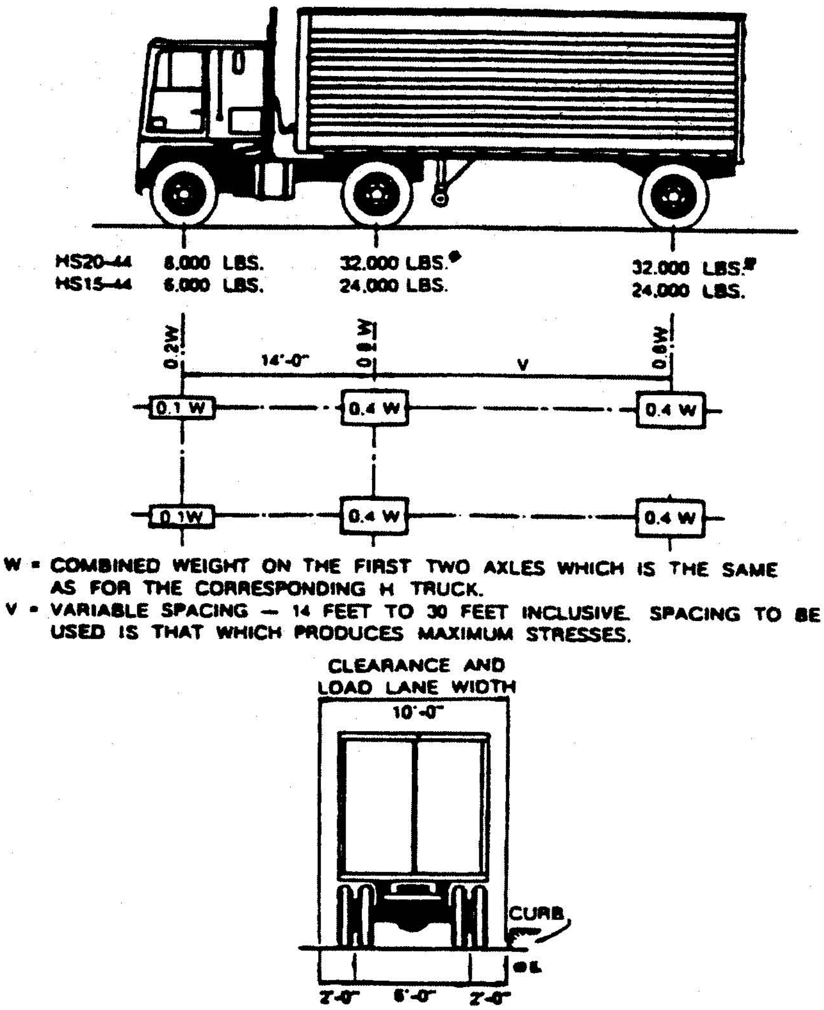


Figure 6.7.2.1 Standard HS Truck

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for HS20 loading, one-axle load of 24,000 pounds or two-axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

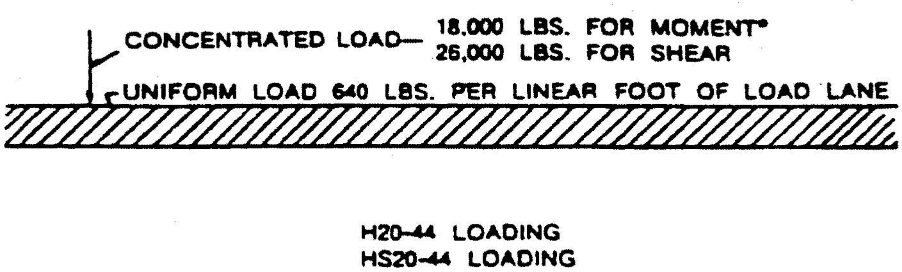


Figure 6.7.2.2 Standard HS Lane Load

*For the determination of maximum negative moment in continuous spans, the lane load shown shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect.

6.7.2.5 Live Load Effects (L)

Live load moments in longitudinal stringers and girders may be calculated using the moment table, Appendix A3, for live load moments produced by the HS20 load.

Live load moments in the intermediate and end floor beams of trusses and through girders may be calculated by using the tables of live load reactions, Appendices A4 and A5. The tables, along with the moment formulas on the same sheets, provide a convenient means of computing the live load moments based on the HS20 load.

Live loads in truss members can be calculated by using the formulas for maximum shear and moments given in Appendices A6 through A10. Using these formulas will give the maximum live load stresses for the HS20 truck. Note that the formulas are valid only when used within the given limits. Modifications of the formulas may be required under loadings not meeting these limits. Such modifications may be found necessary when the structure or panels are too short to permit the entire load to be on the structure with the load positioned to produce the maximum shear or moment.

6.7.3 Distribution of Loads

The fraction of live load transferred to a single member should be selected in accordance with the current AASHTO Design Specifications. These values represent a possible combination of diverse circumstances. The option exists to substitute field measured values, analytically calculated values or those determined from advanced structural analysis methods based on the properties of the existing structure. Loadings should be placed in positions causing the maximum response in the components being evaluated.

6.7.4 Impact (I)

Impact should be added to the live load used for rating in accordance with the current AASHTO Design Specifications. However, specification impact may be reduced when conditions of alignment, enforced speed posting, and similar situations require a vehicle to substantially reduce speed in crossing the structure.

6.7.5 Deflection

Live load deflection limitations should not be considered in load rating except in special cases.

6.7.6 Longitudinal Loads

The rating of the bridge members to include the effects of longitudinal loads in combination with dead and live load effects should be done at the Operating level. Where longitudinal stability is considered inadequate, the structure may be posted for restricted speed. In addition, longitudinal loads should be used in the evaluation of the adequacy of the substructure elements.

6.7.7 Environmental Loads

The rating of the bridge members to include the effects of environmental loads in combination with dead and live load effects should be done at the Operating level.

6.7.7.1 Wind

Lateral loads due to wind normally need not be considered in load rating.

However, the effects of wind on special structures such as movable bridges, suspension bridges and other high-level structures should be evaluated.

6.7.7.2 Earthquake

Earthquake loads should not be considered in calculating load ratings or in determining live load restrictions.

To evaluate the resistance of the structure to seismic forces, the methods described in Division I-A, Seismic Design of the AASHTO Design Specifications may be used.

6.7.7.3 Thermal Effects

Stresses caused by thermal changes should not be considered in calculating load ratings except for long-span bridges and concrete arches.

6.7.7.4 Stream Flow

Forces caused by water movements should not be considered in calculating the load rating. However, remedial action should be considered if these forces are especially critical to the structure's stability.

6.7.7.5 Ice Pressure

Forces caused by ice pressure should be considered in the evaluation of substructure elements in those regions where such effect can be significant. If these forces are especially important, then corrective action should be recommended.

6.8 DOCUMENTATION OF RATING

The load rating of a bridge should be completely documented in writing including all background information such as field inspection reports, material and load test data, all supporting computations, and a clear statement of all assumptions used in calculating the load rating. If a computer model was used, the input data file should be retained for future use.

APPENDIX C
FORMULAS FOR THE CAPACITY (C) OF TYPICAL BRIDGE COMPONENTS BASED ON THE LOAD FACTOR METHOD

C.1 GENERAL

When using the Load Factor Method, the capacity (C) in the basic load rating equation (6-la) is based on procedures in the latest edition of AASHTO *Standard Specifications for Highway Bridges* (AASHTO Design Specifications) including all Interims. This Appendix summarizes the capacity determination for typical bridge members of steel, reinforced concrete or prestressed concrete. For more conditions not covered in this Appendix, the AASHTO Design Specifications should be used.

The formulas shown below have been taken from the AASHTO Design Specifications. All equation and article numbers cited below refer to this Specification. The notation used in the formulas is as defined in the AASHTO Design Specifications.

C.2 CAPACITY OF STEEL MEMBERS (PART D, STRENGTH DESIGN METHOD)

C.2.1 SECTIONS IN BENDING

The capacities specified in C.2.1.1 and C.2.1.2 are applicable to compact rolled or welded beams and girders, satisfying the applicable cross-sectional limitations, which are rolled or fabricated from steels with a specified minimum yield strength between 33,000 and 50,000 psi. The capacities specified in C.2.1.3 through C.2.1.5 are applicable to non-compact rolled, riveted or welded beams and girders satisfying the applicable cross-sectional limitations, which are rolled or fabricated from steels with a minimum specified yield strength between 33,000 and 100,000 psi. The equations found in C.2.1.1 through C.2.1.5 are not applicable to hybrid girders.

C.2.1.1 Compact, Braced, Non-Composite

$$C = F_y Z \qquad \text{(10-92)}$$

C.2.1.2 Compact, Composite

Positive Moment Sections
For composite positive moment sections satisfying the cross-sectional limitations specified in Article 10.50.1.1.2:

In simple spans or in continuous spans with compact non-composite negative-moment pier sections:

$$C = M_u$$

where M_u is determined according to Equation (10-129b) or Equation (10-129c), as applicable, in Article 10.50.1.1.2. For steel with $F_y = 33{,}000$ psi, $\beta = 0.9$ in Article 10.50.1.1.2.

In continuous spans with non-compact non-composite or composite negative-moment pier sections:

Tension and compression flange:

$$C = F_y$$

Alternatively, C may be taken as M_u, where M_u is determined according to Equation (10-129d) in Article 10.50.1.1.2.

Note: According to the preceding requirements, the capacity of a composite positive moment section satisfying the cross-sectional limitations for a compact section specified in Article 10.50.1.1.2 will be at or just below the full plastic moment capacity, M_p, in simple spans and in continuous spans with compact pier sections. In this case, the dead and live load moments are to be used in the basic load rating equation to compute a rating factor for the section. In continuous spans with non-compact pier sections, the capacity of a compact composite positive moment section will typically be taken equal to the yield stress F_y. In this case, the dead and live load stresses in each flange are to be used in the basic load rating equation to compute a rating factor for each flange. In either case, however, the web slenderness requirement for the positive moment section given by Equation (10-129) is to be checked using the depth of the web in compression at the plastic moment D_{cp}. The elastic depth of the web in compression, D_c is not to be used in checking the web slenderness requirement for these sections.

Negative Moment Sections

For composite negative moment sections satisfying the cross-sectional limitations specified in Article 10.50.2.1:

$$C = M_u$$

where M_u is determined according to the provisions of Article 10.50.2.1.

C.2.1.3 Non-Compact, Non-Composite

The lesser of:

$$C = F_y S_{xt} \qquad (10\text{-}98)$$

or

If Equation (10-101) is satisfied:

$$C = F_y S_{xt} \qquad (10\text{-}99)$$

where $F_{cr} = \left(4{,}400\frac{t}{b}\right)^2 \leq F_y$. R_b shall be calculated from the provisions of Article 10.48.4.1 with F_{cr} substituted for the term M_r / S_{xc} when Equation (10-103b) applies.

If Equation (10-101) is not satisfied:

$$C = F_{cr} S_{xc} Rb \leq M_u$$

where M_u is determined according to the provisions of Article 10.48.4.1.

C.2.1.4 Non-Compact, Composite, Positive Moment Section

Tension flange:

$$C = F_y$$

Compression flange:

$$C = F_y R_b$$

When R_b is determined from Equation (10-103b), F_y shall be substituted for the term M_r / S_{xc} and A_{fc} shall be taken as the effective combined transformed area of the top flange and concrete deck that yields D_c calculated in accordance with Article 10.50(b). The resulting R_b factor shall be distributed to the top flange and concrete deck n proportion to their relative stiffness.

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of Article 10.50(b), an iterative procedure may be required to determine the rating factor for the compression flange.

C.2.1.5 Non-Compact, Composite, Negative Moment Section

Tension flange:

$$C = F_y$$

Compression flange:

If Equation (10-101) is satisfied:

$$C = F_{cr} R_b$$

where $F_{cr} = \left(4{,}400\frac{t}{b}\right)^2 \leq F_y$. R_b shall be calculated from the provisions of Article 10.48.4.1 with F_{cr} substituted for the term M_r / S_{xc} when Equation (10-103b) applies.

If Equation (10-101) is not satisfied:

$$C = F_{cr} R_b \leq M_u / S_{xc}$$

where M_u and S_{xc} are determined according to the provisions of Article 10.48.4.1.

D_c of the composite section consisting of the steel section plus the longitudinal reinforcement may conservatively be used in lieu of D_c calculated according to the provisions of Article 10.50(b).

where M_u is found in accordance with Article 10.48.4.1

C.2.2 SECTIONS IN SHEAR

$$C = V_u \qquad \text{(10-113 or 10-114)}$$

where V_u is found in accordance with Article 10.48.8.1

C.2.3 SECTIONS IN SHEAR AND BENDING (ARTICLE 10.48.8.2)

For sections subject to combined shear and bending where the shear capacity is governed by Equation (10-114) for stiffened girders, the load rating shall be determined according to the following procedure. For composite non-compact sections, replace the moments (M_D and $M_{L(1+I)}$) with the corresponding stresses (f_D and $f_{L(1+I)}$) and the maximum bending strength (M_u) of the section with the maximum bending strength (F_u) of the compression or tension flange, as applicable, in the following equations.

STEP 1: Determine the initial load rating factors for shear and bending moment ignoring moment-shear interaction:

$$RF_{Vi} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}}$$ *initial shear rating factor*

$$RF_{Mi} = \frac{M_u - A_1 M_D}{A_2 M_{L(1+I)}}$$ *initial moment rating factor*

where: M_u is found as described above for sections in bending
V_u is found as for sections in shear
M_D is the dead load bending moment
V_D is the dead load shear
$M_{L(1+I)}$ is the maximum live load plus impact bending moment
$V_{L(1+I)}$ is the maximum live load plus impact shear

For composite non-compact sections, the initial moment rating factor shall be taken as the smaller of the rating factors determined separately for the compression and tension flange.

STEP 2: Determine the initial controlling rating factor ignoring moment-shear interaction:

$$RF = \text{minimum of } (RF_{Vi}, RF_{Mi}) \text{ from STEP 1}$$ *initial controlling rating factor*

STEP 3: Determine the factored moment and shear using the initial controlling rating factor from STEP 2 as follows:

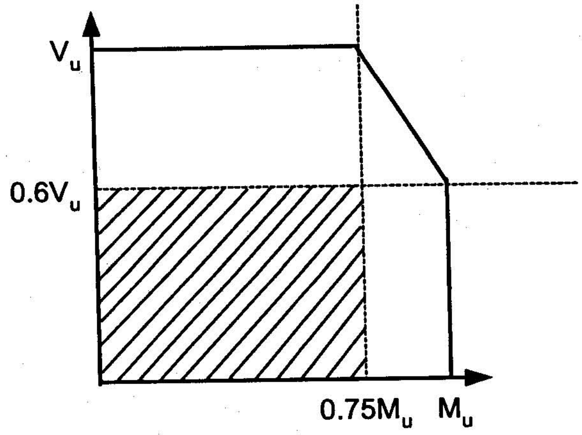
$$V = A_1 V_D + RF \times A_2 \times V_{L(1+I)}$$
$$M = A_1 M_D + RF \times A_2 \times M_{L(1+I)}$$

STEP 4: Determine the final controlling rating factor as follows:

RF = **minimum of** (RF_{Vi}, RF_M) **determined from one of the following four cases:** *final controlling rating factor*

CASE A:

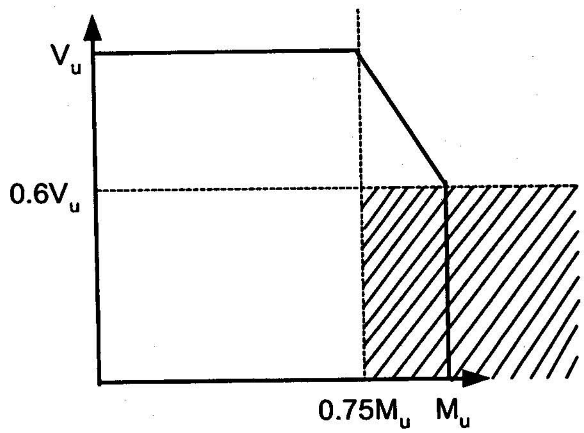
If $V \le 0.6V_u$ and $M \le 0.75M_\omega$ then:



$RF_v = RF_{vi}$ and $RF_M = RF_{Mi}$

***CASE B*:**

If $V \leq 0.6V_u$ and $M > 0.75M_\omega$ then:



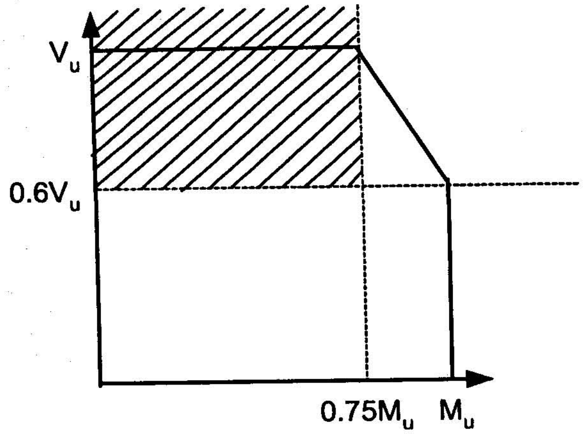
$$RF_V = \frac{V_{Limit} - A_1 V_D}{A_2 V_{L(1+I)}}$$ ***reduced shear rating factor***

$$RF_M = \frac{M_u - A_1 M_D}{A_2 M_{L(1+I)}}$$ ***moment rating factor***

where: $V_{Limit} = 0.6V_u \geq CV_p$

***CASE C*:**

If $V > 0.6V_u$ and $M \leq 0.75M_\omega$ then:

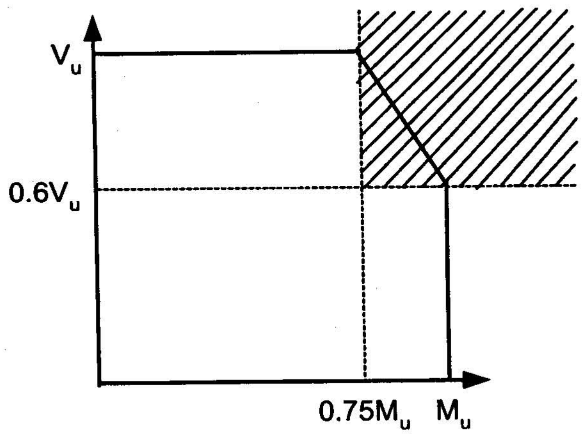


$$RF_V = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}}$$ *shear rating factor*

$$RF_M = \frac{0.75M_u - A_1 M_D}{A_2 M_{L(1+I)}}$$ *reduced moment rating factor*

***CASE D*:**

Otherwise:



$$RF_M = RF_V = RF_{M-V} = \frac{2.2V_u M_u - A_1 V_D M_u - 1.6A_1 M_D V_u}{A_2 V_{L(1+i)} M_u + 1.6A_2 M_{L(1+i)} V_u}$$ *moment-shear rating factor*

1) If $RF_{M-V} \geq \dfrac{CV_p - A_1 V_D}{A_2 V_{L(1+i)}} \Rightarrow RF_M = RF_V = RF_{M-V}$

2) Otherwise: $RF_V = \dfrac{CV_p - A_1 V_D}{A_2 V_{L(1+i)}}$; $RF_M = \dfrac{M_u - A_1 M_D}{A_2 M_{L(1+i)}}$

STEP 5: If the final controlling rating factor is different than the initial controlling rating factor, STEPS 2 through 4 can be repeated (using the final controlling rating factor as the initial controlling rating factor) only if a more-accurate rating factor is justified.

STEP 6: When CASE B, C or D controls the rating and a higher rating is desired for moment and/or shear, STEPS 2 through 5 may be repeated using sets of concurrent factored live-load plus impact moments and shears to determine the final controlling rating factor. In lieu of investigating numerous combinations of concurrent moments and shears, it is recommended that the rating be repeated using: i) the maximum factored live-load plus impact moment in conjunction with a percentage (less than 100 percent) of the maximum factored live-load plus impact shear, and ii) the maximum factored live-load plus impact shear in conjunction with a percentage (less than 100 percent) of the maximum factored live-load plus impact moment. The final controlling rating factor is the lesser of the factors obtained using i) and ii). If the resulting final controlling rating factor is affected by moment-shear interaction, it must not exceed the initial rating factor for the controlling action. In lieu of a more rigorous analysis, the determination of the appropriate percentage to be applied should be based upon rational engineering judgment. The percentage that is applied should not reduce the maximum factored live-load plus impact moment or shear, as applicable, below the actual concurrent factored live-load plus impact moment or shear.

<u>Example #1:</u>

Load Factor Design
Inventory Rating $\left(A_1 = 1,3; A_2 = 2.17\right)$
Composite Non-Compact Section

Assume the following:

$V_u = 411.7$ kips	$f_D = 20$ ksi
$V_D = 100$ kips	$f_{L(1+I)} = 10.05$ ksi
$V_{L(1+I)} = 90$ kips	$F_u = 50$ ksi
$V_p = 700$ kips	$C = 0.42$

$$RF_{Vi} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(100)}{2.17(90)} = 1.44$$

$$RF_{Mi} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{50 - 1.3(20)}{2.17(10.05)} = 1.10$$

$$\therefore RF = RF_{Mi} = 1.10$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(1.10)(10.05)\right] = 50.0 \text{ ksi} > 37.5 \text{ ksi } (= 0.75F_u)$$

$$\left(1.3V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(100) + 2.17(1.10)(90)\right] = 344.9 \text{ k} > 247.0 \text{ k } (= 0.6V_u)$$

Therefore:

$$RF_M = RF_V = RF_{M-V} = \frac{2.2V_u F_u - A_1 V_D F_u - 1.6 A_1 f_D V_u}{A_2 V_{L(1+I)} F_u + 1.6 A_2 f_{L(1+I)} V_u}$$

$$= \frac{2.2(411.7)(50) - 1.3(100)(50) - 1.6(1.3)(20)(411.7)}{2.17(90)(50) + 1.6(2.17)(10.05)(411.7)} = 0.90$$

To illustrate that the above equation is valid, determine the shear and moment ratings (as affected by moment-shear interaction) using a more indirect approach. These calculations are solely to demonstrate the validity of the preceding equation and need not be repeated unless such a check is desired:

First, the shear rating:

$$f_s = \left[1.3 f_D + 2.17(RF)\left(f_{L(1+I)}\right)\right] = \left[1.3(20) + 2.17(0.90)(10.05)\right] = 45.6 \text{ ksi}$$

$$\frac{f_s}{F_u} = \frac{45.6}{50} = 0.912$$

$$V_{u\,reduced} = \left[2.2 - 1.6(0.912)\right]V_u = 0.74V_u$$

$$RF_V = \frac{0.74(411.7) - 1.3(100)}{2.17(90)} = 0.894 \ vs.\ 0.90 \ say\,ok$$

Followed by the moment rating:

$$V = \left[1.3V_D + 2.17(RF)\left(V_{L(1+I)}\right)\right] = \left[1.3(100) + 2.17(0.90)(90)\right] = 305.8 \text{ k}$$

$$V / V_u = 305.8 / 411.7 = 0.743$$

$$F_{u\,reduced} = \left[1.375 - 0.625(0.743)\right]F_u = 0.91F_u$$

$$RF_M = \frac{0.91(50) - 1.3(20)}{2.17(10.05)} = 0.894 \ vs.\ 0.90 \ say\,ok$$

Continuing:

$$\frac{CV_p - A_1V_D}{A_2V_{L(1+I)}} = \frac{0.42(700) - 1.3(100)}{2.17(90)} = 0.840 < RF_{M-V} = 0.90$$

$$\therefore RF = RF_{M-V} = 0.90 \text{ (Case D1 controls)}$$

Try second iteration:

$$\left(1.3f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(0.90)(10.05)\right] = 45.6 \text{ ksi} > 37.5 \text{ ksi} (= 0.75F_u)$$

$$\left(1.3V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(100) + 2.17(0.90)(90)\right] = 305.8 \text{ k} > 247.0 \text{ k} (= 0.6V_u)$$

Therefore:

$$RF_M = RF_V = RF_{M-V} = \frac{2.2V_uF_u - A_1V_DF_u - 1.6A_1f_DV_u}{A_2V_{L(1+I)}F_u + 1.6A_2f_{L(1+I)}V_u} = 0.90 \text{ (converged)}$$

Example #2:

Load Factor Design
Inventory Rating $(A_1 = 1.3; A_2 = 2.17)$
Composite Non-Compact Section

Assume the following: $V_u = 411.7$ kips $\quad f_D = 18$ ksi
$V_D = 30$ kips $\quad f_{L(1+I)} = 9.86$ ksi
$V_{L(1+I)} = 60$ kips $\quad F_u = 48$ ksi
$V_p = 600$ kips $\quad C = 0.383$

$$RF_{VI} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(30)}{2.17(60)} = 2.87$$

$$RF_{MI} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{48 - 1.3(18)}{2.17(9.86)} = 1.15$$

$$\therefore RF = RF_{MI} = 1.15$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(18) + 2.17(1.15)(9.86)\right] = 48.0 \text{ ksi} > 36.0 \text{ ksi} (= 0.75 F_u) a$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(30) + 2.17(1.15)(60)\right] = 188.7 \text{ k} < 247.0 \text{ k} (= 0.6 V_u)$$

$$V_{Limit} = 0.6 V_u \geq C V_p = 247.0 \text{ kips} > (0.383)(600) = 230 \text{ kips}$$

Therefore:

$$RF_M = \frac{F_u - 1.3 f_D}{2.17 f_{L(1+I)}} = \frac{48 - 1.3(18)}{2.17(9.86)} = 1.15$$

$$RF_V = \frac{V_{Limit} - 1.3 V_D}{2.17 V_{L(1+I)}} = \frac{247.0 - 1.3(30)}{2.17(60)} = 1.60$$

$\therefore RF = RF_M = 1.15$ **(Case B controls)** **(converged by inspection)**

Example #3:

Load Factor Design
Inventory Rating $(A_1 = 1.3; A_2 = 2.17)$
Composite Non-Compact Section

Assume the following: $V_u = 411.7$ kips $f_D = 5$ ksi
$V_D = 60$ kips $f_{L(1+I)} = 6$ ksi
$V_{L(1+I)} = 90$ kips $F_u = 48$ ksi
$V_p = 700$ kips $C = 0.353$

$$RF_{VI} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(60)}{2.17(90)} = 1.71$$

$$RF_{MI} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{48 - 1.3(5)}{2.17(6)} = 3.19$$

$$\therefore RF = RF_{VI} = 1.71$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(L+I)}\right) = \left[1.3(5) + 2.17(1.71)(6)\right] = 29.0 \text{ ksi} < 36.0 \text{ ksi} (= 0.75 F_u)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(60) + 2.17(1.71)(90)\right] = 411.7 \text{ k} > 247.0 \text{ k} (= 0.6 V_u)$$

Therefore:

$$RF_M = \frac{0.75 F_u - 1.3 f_D}{2.17 f_{L(1+I)}} = \frac{0.75(48) - 1.3(5)}{2.17(6)} = 2.27$$

$$RF_V = \frac{V_u - 1.3 V_D}{2.17 V_{L(1+I)}} = \frac{411.7 - 1.3(60)}{2.17(90)} = 1.71$$

$\therefore RF = RF_V = 1.71$ **(Case C controls) (converged by inspection)**

Example #4:

Load Factor Design
Inventory Rating $(A_1 = 1.3; A_2 = 2.17)$
Composite Non-Compact Section

Assume the following: $V_u = 411.7$ kips $f_D = 5$ ksi
$V_D = 30$ kips $f_{L(1+I)} = 6$ ksi
$V_{L(1+I)} = 60$ kips $F_u = 48$ ksi
$V_p = 700$ kips $C = 0.353$

$$RF_{VI} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(30)}{2.17(60)} = 2.87$$

$$RF_{MI} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{48 - 1.3(5)}{2.17(6)} = 3.19$$

$$\therefore RF = RF_{VI} = 2.87$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(5) + 2.17(2.87)(6)\right] = 44.0 \text{ ksi} > 36.0 \text{ ksi} (= 0.75F_u)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(30) + 2.17(2.87)(60)\right] = 411.7 \text{ k} > 247.0 \text{ k} (= 0.6V_u)$$

Therefore:

$$RF_M = RF_V = RF_{M\text{-}V} = \frac{2.2V_u F_u - A_1 V_D F_u - 1.6 A_1 f_D V_u}{A_2 V_{L(1+I)} F_u + 1.6 A_2 f_{L(1+I)} V_u}$$

$$= \frac{2.2(411.7)(48) - 1.3(30)(48) - 1.6(1.3)(5)(411.7)}{2.17(60)(48) + 1.6(2.17)(6)(411.7)} = 2.52$$

Continuing:

$$\frac{CV_p - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{0.353(700) - 1.3(30)}{2.17(60)} = 1.60 < RF_{M\text{-}V} = 2.52$$

$\therefore RF = RF_{M\text{-}V} = 2.52$ **(Case D1 controls)**

Try a second iteration:

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(5) + 2.17(2.52)(6)\right] = 39.3 \text{ ksi} > 36.0 \text{ ksi} (= 0.75F_u)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(30) + 2.17(2.52)(60)\right] = 367.1 \text{ k} > 247.0 \text{ k} (= 0.6V_u)$$

Therefore:

$$RF_M = RF_V = RF_{M\text{-}V} = \frac{2.2V_u F_u - A_1 V_D F_u - 1.6 A_1 f_D V_u}{A_2 V_{L(1+I)} F_u + 1.6 A_2 f_{L(1+I)} V_u} = 2.52 \quad \text{(converged)}$$

Example #5:

Load Factor Design
Inventory Rating $(A_1 = 1.3;\ A_2 = 2.17)$
Composite Non-Compact Section

Assume the following: $V_u = 411.7$ kips $f_D = 20$ ksi
$V_D = 70$ kips $f_{L(1+I)} = 10$ ksi
$V_{L(1+I)} = 90$ kips $F_u = 50$ ksi
$V_p = 700$ kips $C = 0.42$

$$RF_{Vi} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(70)}{2.17(90)} = 1.64$$

$$RF_{Mi} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{50 - 1.3(20)}{2.17(10)} = 1.11$$

$$\therefore RF = RF_{Mi} = 1.11$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(1.11)(10)\right] = 50.0 \text{ ksi} > 37.5 \text{ ksi} (= 0.75 F_u)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(70) + 2.17(1.11)(90)\right] = 307.0 \text{ k} > 247.0 \text{ k} (= 0.6 V_u)$$

Therefore:

$$RF_M = RF_V = RF_{M-V} = \frac{2.2 V_u F_u - A_1 V_D F_u - 1.6 A_1 f_D V_u}{A_2 V_{L(1+I)} F_u + 1.6 A_2 f_{L(1+I)} V_u}$$

$$= \frac{2.2(411.7)(50) - 1.3(70)(50) - 1.6(1.3)(20)(411.7)}{2.17(90)(50) + 1.6(2.17)(10)(411.7)} = 0.98$$

Continuing:

$$\frac{C V_p - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{0.42(700) - 1.3(70)}{2.17(90)} = 1.04 > RF_{M-V} = 0.98$$

Therefore:

$$RF_V = 1.04$$

$$RF_M = \frac{F_u - 1.3 f_D}{2.17 f_{L(1+I)}} = \frac{50 - 1.3(20)}{2.17(10)} = 1.11$$

$\therefore RF = RF_V = 1.04$ **(Case D2 controls)**

Try second iteration:

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(1.04)(10)\right] = 48.6 \text{ ksi} > 37.5 \text{ ksi} (= 0.75 F_u)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(70) + 2.17(1.04)(90)\right] = 294.0 \text{ k} > 247.0 \text{ k} (= 0.6 V_u) \quad \textbf{(converged)}$$

$A_1 = 1.3$ $A_2 = 2.17$ **Inventory**

								Step 1		Step 2	First Iteration: Step 3				Step 4			
Example	V_D	V_L	V_p	CV_p	f_D	f_L	F_y	RF_V	RF_M	RF_1	V	$0.6V_p$	f	$0.75F_y$	Cas	RF_V	RF_M	Min RF
1	100	90	411.7	294.0	20	10.05	50	1.44	1.10	1.10	344.9	247.0	50	38	D1	0.90	0.90	0.90
2	30	60	411.7	230.0	18	9.86	48	2.87	1.15	1.15	188.7	247.0	48	36	B	1.60	1.15	1.15
3	60	90	411.7	247.0	5	6	48	1.71	3.19	1.71	411.7	247.0	29	36	C	1.71	2.27	1.71
4	30	60	411.7	247.0	5	6	48	2.87	3.19	2.87	411.7	247.0	44	36	D1	2.52	2.52	2.52
5	70	90	411.7	294.0	20	10	50	1.64	1.11	1.11	307.0	247.0	50	38	D2	1.04	1.11	1.04
											Second Iteration:			(second iteration was not needed)				
								0.90	0.90	0.90	305.3	247.0	46	38	D1	0.90	0.90	0.90
								1.60	1.15	1.15	188.7	247.0	48	36	B	1.60	1.15	1.15
								1.71	2.27	1.71	411.7	247.0	29	36	C	1.71	2.27	1.71
								2.52	2.52	2.52	367.1	247.0	39	36	D1	2.52	2.52	2.52
								1.04	1.11	1.04	294.0	247.0	49	38	D2	1.04	1.11	1.04

C.2.4 COMPRESSION MEMBERS

C.2.4.1 Concentrically Loaded Members

$$C = 0.85 A_s F_{cr} \qquad (10\text{-}150)$$

where F_{cr} is found in accordance with Article 10.54.1.1.

C.2.4.2 Combined Axial Load and Bending

Interaction equations (10-155 and 10-156) must be satisfied by factored axial force (P) and factored axial moment (M). See Article 10.54.2

C.2.5 CAPACITY BASED ON OVERLOAD PROVISIONS OF ARTICLE 10.57

Note $A_1 = 1.0$ and $A_2 = 1.67$ in the basic rating equation (6-1a) when making this check.

C.2.5.1 Non-Composite Beams

$$C = 0.8 F_y S \qquad \text{(Article 10.57.1)}$$

C.2.5.2 Composite Beams

$$C = 0.95 F_y \qquad \text{(Article 10.57.2)}$$

C.2.5.3 Web Compressive Stress

$$C = F_{cr} \qquad \text{(Article 10.57)}$$

where F_{cr} is found in accordance with Equation (10-173).

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of Article 10.50(b), an iterative procedure may be necessary to determine the rating factor at composite positive moment sections. At composite negative moment sections, D_c of the composite section consisting of the steel section plus the longitudinal reinforcement may conservatively be used in lieu of D_c calculated according to the provisions of Article 10.50(b).

C.3 REINFORCED CONCRETE MEMBERS (ARTICLE 8.16)

C.3.1 SECTIONS IN BENDING

C.3.1.1 Rectangular Sections with Tension Reinforcement Only

$$C = \varnothing M_n = \varnothing\left[A_s f_y\left(d - \frac{a}{2}\right)\right] \qquad (8\text{-}16)$$

where:

$$a = \frac{A_s f_y}{0.85 f_c b} \qquad (8\text{-}17)$$

C.3.1.2 Tee Section (Flanged) With Tension Reinforcement Only

C.3.1.2.1 Compression Zone Within Flange Area

$C = \varnothing M_n$ as for C.3.1.1 above

C.3.1.2.2 Compression Zone Includes Both Flange Area and a Portion of the Web

$$C = \varnothing M_n \qquad (8\text{-}19)$$

where M_n is found in accordance with Article 8.16.3.3.2.

C.3.2 SECTIONS IN COMPRESSION

See Article 8.16.4.

C.3.3 SECTIONS IN SHEAR

$$C = V_u \qquad (8\text{-}46)$$

See Article 8.16.6 for the procedure for computing V_u.

C.4 PRESTRESSED CONCRETE MEMBERS (SECTION 9)

C.4.1 SECTIONS IN BENDING

C.4.1.1 Rectangular Sections Without Non-Prestressed Reinforcement

$$C = \varnothing M_n = \varnothing\left[A_s^{*} f_{su}^{*} d\left(1 - 0.6\frac{\rho^{*} f_{su}^{*}}{f_c}\right)\right] \qquad (9\text{-}13)$$

C.4.1.2 Tee (Flanged) Sections Without Non-Prestressed Reinforcement

C.4.1.2.1 Compression Zone Within Flange Area

$C = \varnothing M_n$ as for Rectangular Sections, C.4.1.1 above

C.4.1.2.2 Compression Zone Includes Flange Area and Part of Web

$$C = \varnothing M_n \qquad (9\text{-}14)$$

See Article 9.17.3 for the evaluation of this equation.

C.4.2 SECTIONS IN SHEAR

$$C = V_u \tag{9-26}$$

V_u should be found in accordance with Article 9.20.

C.3.1.2 Tee Section (Flanged) With Tension Reinforcement Only

C.3.1.2.1 Compression Zone Within Flange Area

$C = \phi M_n$ as for C.3.1.1 above

C.3.1.2.2 Compression Zone Includes Both Flange Area and a Portion of the Web

$$C = \phi M_n \qquad (8\text{-}19)$$

where M_n is found in accordance with Article 8.16.3.3.2.

C.3.2 SECTIONS IN COMPRESSION

See Article 8.16.4.

C.3.3 SECTIONS IN SHEAR

$$C = V_u \qquad (8\text{-}46)$$

See Article 8.16.6 for the procedure for computing V_u.

C.4 PRESTRESSED CONCRETE MEMBERS (SECTION 9)

C.4.1 SECTIONS IN BENDING

C.4.1.1 Rectangular Sections Without Non-Prestressed Reinforcement

$$C = \phi M_n = \phi \left[A_s^* f_{su}^* d \left(1 - 0.6 \frac{\rho^* f_{su}^*}{f_c'} \right) \right] \qquad (9\text{-}13)$$

C.4.1.2 Tee (Flanged) Sections Without Non-Prestressed Reinforcement

C.4.1.2.1 Compression Zone Within Flange Area

$C = \phi M_n$ as for Recta..gular Sections, C.4.1.1 above

C.4.1.2.2 Compression Zone Includes Flange Area and Part of Web

$$C = \phi M_n \qquad (9\text{-}14)$$

See Article 9.17.3 for the evaluation of this equation.

C.4.2 SECTIONS IN SHEAR

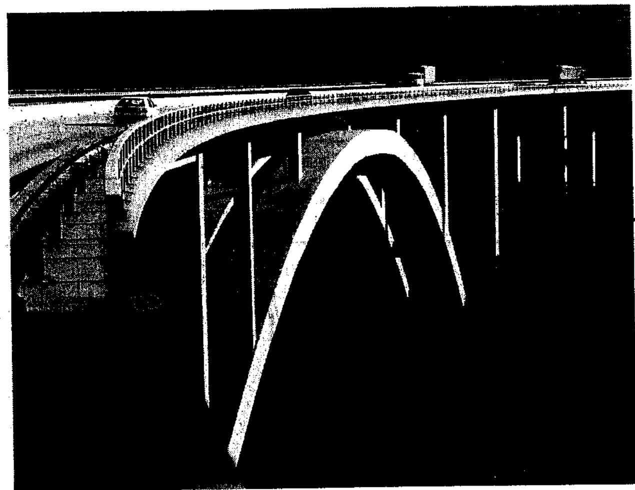
$$C = V_u \qquad (9\text{-}26)$$

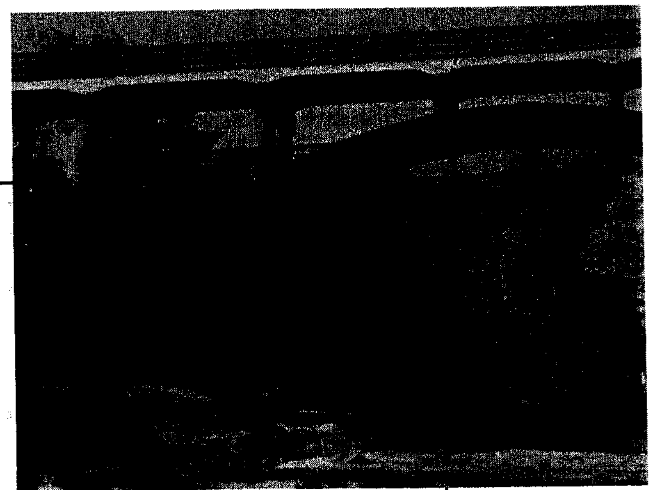
V_u should be found in accordance with Article 9.20.

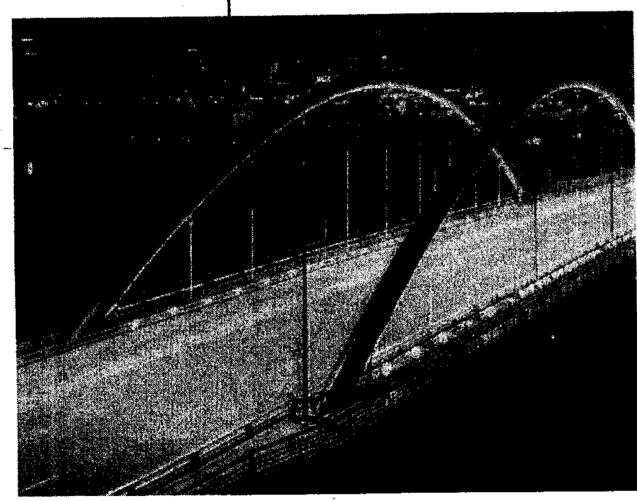
Standard Specifications for Highway Bridges

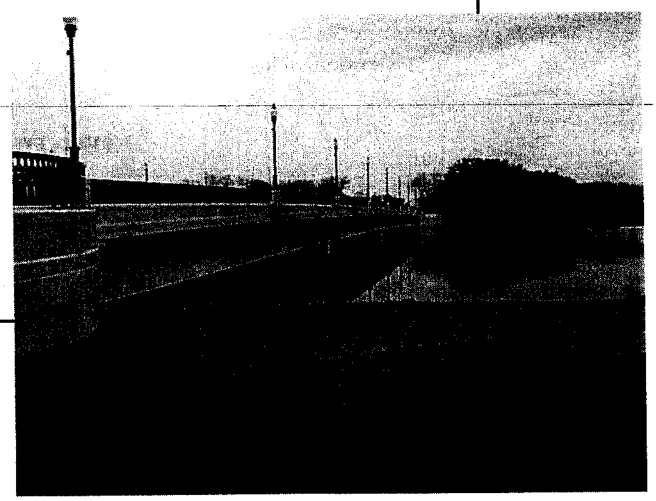
Culvert
Compelation

17th Edition – 2002









*Upper right-hand and lower left-hand pictures courtesy of the National Steel Bridge Alliance.
Lower right-hand picture courtesy of William Oliva and Scott Becker.*

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Z = reduction for ductility and risk assessment
β = (with appropriate script) coefficient applied to actual loads for service load and load factor designs (Article 3.22)
γ = load factor (Article 3.22)
σ_{PL} = proportional limit stress perpendicular to grain (Article 3.25.1.4)
β_B = load combination coefficient for buoyancy (Article 3.22.1)
β_C = load combination coefficient for centrifugal force (Article 3.22.1)
β_D = load combination coefficient for dead load (Article 3.22.1)
β_E = load combination coefficient for earth pressure (Article 3.22.1)
β_{EQ} = load combination coefficient for earthquake (Article 3.22.1)
β_{ICE} = load combination coefficient for ice (Article 3.22.1)
β_L = load combination coefficient for live load (Article 3.22.1)
β_R = load combination coefficient for rib shortening, shrinkage, and temperature (Article 3.22.1)
β_S = load combination coefficient for stream flow (Article 3.22.1)
β_W = load combination coefficient for wind (Article 3.22.1)
β_{WL} = load combination coefficient for wind on live load (Article 3.22.1)
μ = Poisson's ratio (Article 3.23.4.3)

3.2 GENERAL

3.2.1 Structures shall be designed to carry the following loads and forces:

Dead load.
Live load.
Impact or dynamic effect of the live load.
Wind loads.
Other forces, when they exist, as follows:
Longitudinal forces; centrifugal force; thermal forces; earth pressure; buoyancy; shrinkage stresses; rib shortening; erection stresses; ice and current pressure; and earthquake stresses.

Provision shall be made for the transfer of forces between the superstructure and substructure to reflect the effect of friction at expansion bearings or shear resistance at elastomeric bearings.

3.2.2 Members shall be proportioned either with reference to service loads and allowable stresses as provided in Service Load Design (Allowable Stress Design) or, alternatively, with reference to load factors and factored strength as provided in Strength Design (Load Factor Design).

3.2.3 When stress sheets are required, a diagram or notation of the assumed loads shall be shown and the stresses due to the various loads shall be shown separately.

3.2.4 Where required by design conditions, the concrete placing sequence shall be indicated on the plans or in the special provisions.

3.2.5 The loading combinations shall be in accordance with Article 3.22.

3.2.6 When a bridge is skewed, the loads and forces carried by the bridge through the deck system to pin connections and hangers should be resolved into vertical, lateral, and longitudinal force components to be considered in the design.

3.3 DEAD LOAD

3.3.1 The dead load shall consist of the weight of the entire structure, including the roadway, sidewalks, car tracks, pipes, conduits, cables, and other public utility services.

3.3.2 The snow and ice load is considered to be offset by an accompanying decrease in live load and impact and shall not be included except under special conditions.

3.3.2.1 If differential settlement is anticipated in a structure, consideration should be given to stresses resulting from this settlement.

3.3.3 If a separate wearing surface is to be placed when the bridge is constructed, or is expected to be placed in the future, adequate allowance shall be made for its weight in the design dead load. Otherwise, provision for a future wearing surface is not required.

3.3.4 Special consideration shall be given to the necessity for a separate wearing surface for those regions where the use of chains on tires or studded snow tires can be anticipated.

3.3.5 Where the abrasion of concrete is not expected, the traffic may bear directly on the concrete slab. If considered desirable, ¼ inch or more may be added to the slab for a wearing surface.

3.3.6 The following weights are to be used in computing the dead load:

	#/cu.ft.
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete, plain or reinforced	150
Compacted sand, earth, gravel, or ballast	120
Loose sand, earth, and gravel	100
Macadam or gravel, rolled	140
Cinder filling	60
Pavement, other than wood block	150
Railway rails, guardrails, and fastenings (per linear foot of track)	200
Stone masonry	170
Asphalt plank, 1 in. thick	9 lb. sq. ft.

3.4 LIVE LOAD

The live load shall consist of the weight of the applied moving load of vehicles, cars, and pedestrians.

3.5 OVERLOAD PROVISIONS

3.5.1 For all loadings less than H 20, provision shall be made for an infrequent heavy load by applying Loading Combination IA (see Article 3.22), with the live load assumed to be H or HS truck and to occupy a single lane without concurrent loading in any other lane. The overload shall apply to all parts of the structure affected, except the roadway deck, or roadway deck plates and stiffening ribs in the case of orthotropic bridge superstructures.

3.5.2 Structures may be analyzed for an overload that is selected by the operating agency in accordance with Loading Combination Group IB in Article 3.22.

3.6 TRAFFIC LANES

3.6.1 The lane loading or standard truck shall be assumed to occupy a width of 10 feet.

3.6.2 These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width measured between curbs.

3.6.3 Fractional parts of design lanes shall not be used, but roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

3.6.4 The traffic lanes shall be placed in such numbers and positions on the roadway, and the loads shall be placed in such positions within their individual traffic lanes, so as to produce the maximum stress in the member under consideration.

3.7 HIGHWAY LOADS

3.7.1 Standard Truck and Lane Loads*

3.7.1.1 The highway live loadings on the roadways of bridges or incidental structures shall consist of standard trucks or lane loads that are equivalent to truck trains. Two systems of loading are provided, the H loadings and the HS loadings—the HS loadings being heavier than the corresponding H loadings.

3.7.1.2 Each lane load shall consist of a uniform load per linear foot of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans—see Article 3.11.3), so placed on the span as to produce maximum stress. The concentrated load and uniform load shall be considered as uniformly distributed over a 10-foot width on a line normal to the center line of the lane.

3.7.1.3 For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 3.7.6B. The lighter concentrated loads shall be used when the stresses are primarily bending stresses, and the heavier concentrated loads shall be used when the stresses are primarily shearing stresses.

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*Note: The system of lane loads defined here (and illustrated in Figure 3.7.6.B) was developed in order to give a simpler method of calculating moments and shears than that based on wheel loads of the truck.

Appendix B shows the truck train loadings of the 1935 Specifications of AASHO and the corresponding lane loadings.

In 1944, the HS series of trucks was developed. These approximate the effect of the corresponding 1935 truck preceded and followed by a train of trucks weighing three-fourths as much as the basic truck.

3.7.2 Classes of Loading

There are four standard classes of highway loading: H 20, H 15, HS 20, and HS 15. Loading H 15 is 75% of Loading H 20. Loading HS 15 is 75% of Loading HS 20. If loadings other than those designated are desired, they shall be obtained by proportionately changing the weights shown for both the standard truck and the corresponding lane loads.

3.7.3 Designation of Loadings

The policy of affixing the year to loadings to identify them was instituted with the publication of the 1944 Edition in the following manner:

H 15 Loading, 1944 Edition shall be designated..	H 15-44
H 20 Loading, 1944 Edition shall be designated..	H 20-44
H 15-S 12 Loading, 1944 Edition shall be designated..	HS 15-44
H 20-S 16 Loading, 1944 Edition shall be designated..	HS 20-44

The affix shall remain unchanged until such time as the loading specification is revised. The same policy for identification shall be applied, for future reference, to loadings previously adopted by AASHTO.

3.7.4 Minimum Loading

Bridges supporting Interstate highways or other highways which carry, or which may carry, heavy truck traffic, shall be designed for HS 20-44 Loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000 pounds, whichever produces the greatest stress.

3.7.5 H Loading

The H loadings consist of a two-axle truck or the corresponding lane loading as illustrated in Figures 3.7.6A and 3.7.6B. The H loadings are designated H followed by a number indicating the gross weight in tons of the standard truck.

3.7.6 HS Loading

The HS loadings consist of a tractor truck with semitrailer or the corresponding lane load as illustrated in Figures 3.7.7A and 3.7.6B. The HS loadings are designated by the letters HS followed by a number indicating the gross weight in tons of the tractor truck. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans, in that heavy axle loads may be so placed on adjoining spans as to produce maximum negative moments.

3.8 IMPACT

3.8.1 Application

Highway Live Loads shall be increased for those structural elements in Group A, below, to allow for dynamic, vibratory and impact effects. Impact allowances shall not be applied to items in Group B. It is intended that impact be included as part of the loads transferred from superstructure to substructure, but shall not be included in loads transferred to footings nor to those parts of piles or columns that are below ground.

3.8.1.1 Group A—Impact shall be included.

(1) Superstructure, including legs of rigid frames.
(2) Piers, (with or without bearings regardless of type) excluding footings and those portions below the ground line.
(3) The portions above the ground line of concrete or steel piles that support the superstructure.

3.8.1.2 Group B—Impact shall not be included.

(1) Abutments, retaining walls, piles except as specified in Article 3.8.1.1 (3).
(2) Foundation pressures and footings.
(3) Timber structures.
(4) Sidewalk loads.
(5) Culverts and structures having 3 feet or more cover.

3.8.2 Impact Formula

3.8.2.1 The amount of the impact allowance or increment is expressed as a fraction of the live load stress, and shall be determined by the formula:

$$I = \frac{50}{L+125} \qquad (3\text{-}1)$$

in which,

I = impact fraction (maximum 30 percent);

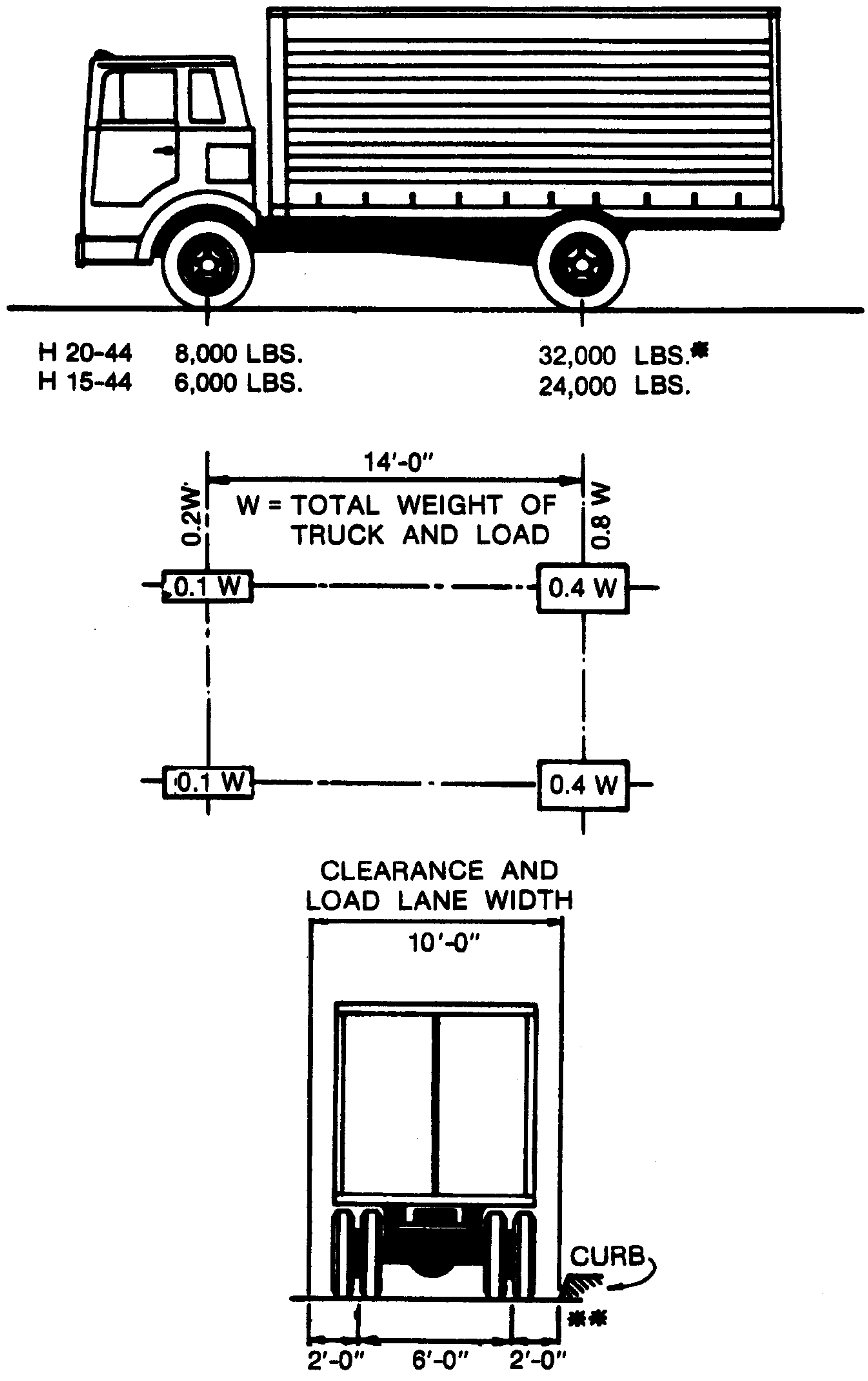
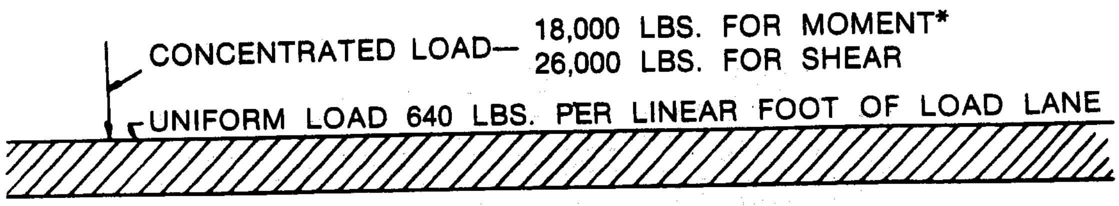


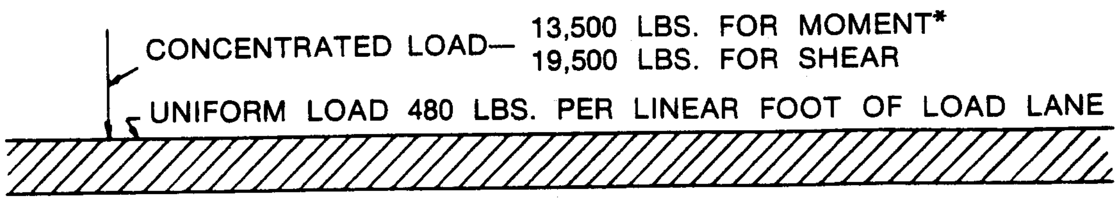
FIGURE 3.7.6A Standard H Trucks

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H 20 Loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2.)



H20-44 LOADING
HS20-44 LOADING



H15-44 LOADING
HS15-44 LOADING

FIGURE 3.7.6B Lane Loading

*For the loading of continuous spans involving lane loading refer to Article 3.11.3 which provides for an additional concentrated load.

L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

3.8.2.2 For uniformity of application, in this formula, the loaded length, L, shall be as follows:

(a) For roadway floors: the design span length.
(b) For transverse members, such as floor beams: the span length of member center to center of supports.
(c) For computing truck load moments: the span length, or for cantilever arms the length from the moment center to the farthermost axle.
(d) For shear due to truck loads: the length of the loaded portion of span from the point under consideration to the far reaction; except, for cantilever arms, use a 30% impact factor.
(e) For continuous spans: the length of span under consideration for positive moment, and the average of two adjacent loaded spans for negative moment.

3.8.2.3 For culverts with cover

0′0″ to 1′-0″ inc. I = 30%
1′-1″ to 2′-0″ inc. I = 20%
2′-1″ to 2′-11″ inc. I = 10%

3.9 LONGITUDINAL FORCES

Provision shall be made for the effect of a longitudinal force of 5% of the live load in all lanes carrying traffic headed in the same direction. All lanes shall be loaded for bridges likely to become one directional in the future. The load used, without impact, shall be the lane load plus the concentrated load for moment specified in Article 3.7, with reduction for multiple-loaded lanes as specified in Article 3.12. The center of gravity of the longitudinal force shall be assumed to be located 6 feet above the floor slab and to be transmitted to the substructure through the superstructure.

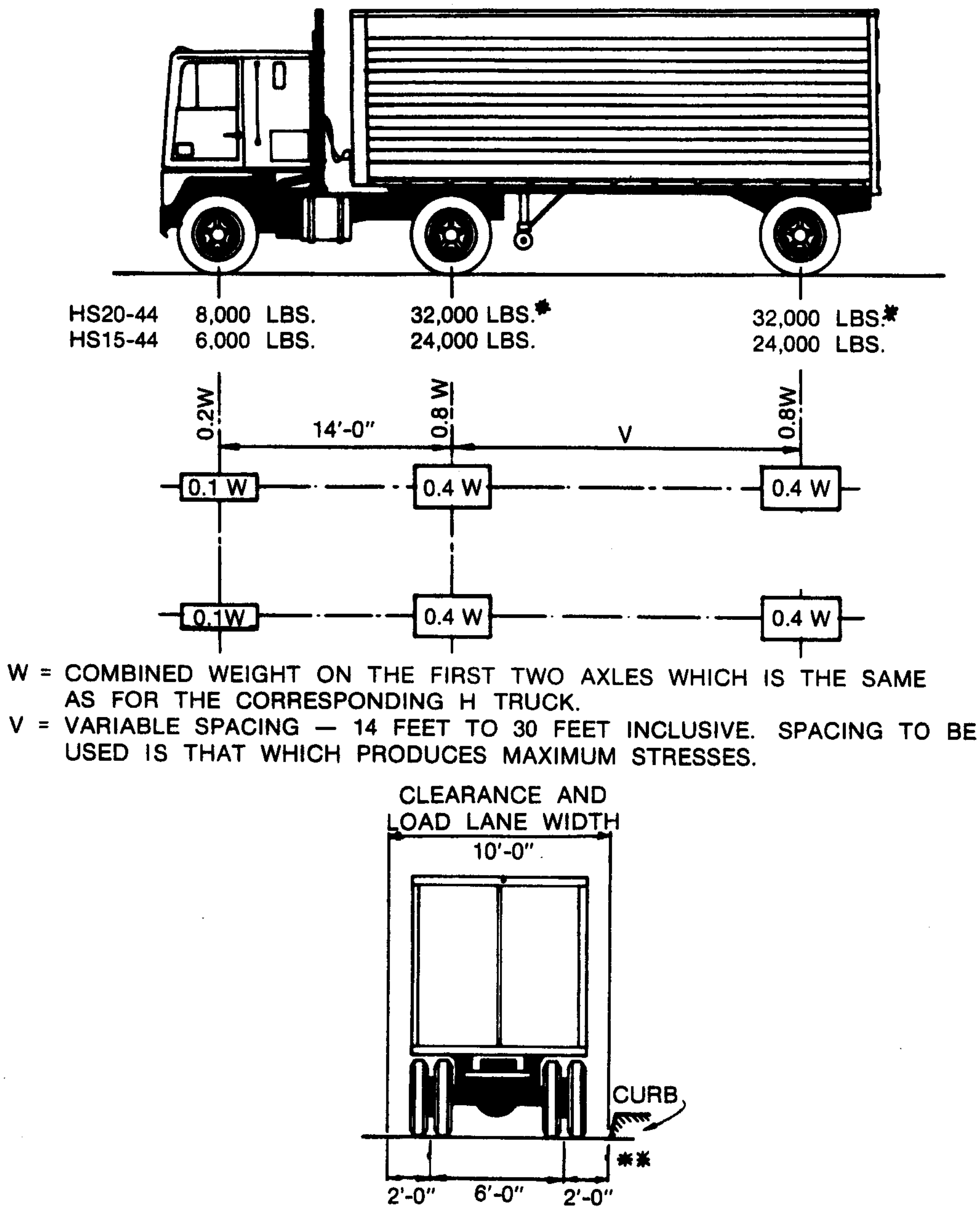


FIGURE 3.7.7A Standard HS Trucks

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H 20 Loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2.)

3.10 CENTRIFUGAL FORCES

3.10.1 Structures on curves shall be designed for a horizontal radial force equal to the following percentage of the live load, without impact, in all traffic lanes:

$$C = 0.00117S^2D = \frac{6.68S^2}{R} \qquad (3\text{-}2)$$

where,

- C = the centrifugal force in percent of the live load, without impact;
- S = the design speed in miles per hour;
- D = the degree of curve;
- R = the radius of the curve in feet.

3.10.2 The effects of superelevation shall be taken into account.

3.10.3 The centrifugal force shall be applied 6 feet above the roadway surface, measured along the center line of the roadway. The design speed shall be determined with regard to the amount of superelevation provided in the roadway. The traffic lanes shall be loaded in accordance with the provisions of Article 3.7 with one standard truck on each design traffic lane placed in position for maximum loading.

3.10.4 Lane loads shall not be used in the computation of centrifugal forces.

3.10.5 When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the centrifugal forces acting on the live load.

3.11 APPLICATION OF LIVE LOAD

3.11.1 Traffic Lane Units

In computing stresses, each 10-foot lane load or single standard truck shall be considered as a unit, and fractions of load lane widths or trucks shall not be used.

3.11.2 Number and Position of Traffic Lane Units

The number and position of the lane load or truck loads shall be as specified in Article 3.7 and, whether lane or truck loads, shall be such as to produce maximum stress, subject to the reduction specified in Article 3.12.

3.11.3 Lane Loads on Continuous Spans

For the determination of maximum negative moment in the design of continuous spans, the lane load shown in Figure 3.7.6B shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect. For maximum positive moment, only one concentrated load shall be used per lane, combined with as many spans loaded uniformly as are required to produce maximum moment.

3.11.4 Loading for Maximum Stress

3.11.4.1 On both simple and continuous spans, the type of loading, whether lane load or truck load, to be used shall be the loading which produces the maximum stress. The moment and shear tables given in Appendix A show which types of loading controls for simple spans.

3.11.4.2 For continuous spans, the lane loading shall be continuous or discontinuous; only one standard H or HS truck per lane shall be considered on the structure.

3.12 REDUCTION IN LOAD INTENSITY

3.12.1 Where maximum stresses are produced in any member by loading a number of traffic lanes simultaneously, the following percentages of the live loads may be used in view of the improbability of coincident maximum loading:

	Percent
One or two lanes	100
Three lanes	90
Four lanes or more	75

3.12.2 The reduction in load intensity specified in Article 3.12.1 shall not be applicable when distribution factors from Table 3.23.1 are used to determine moments in longitudinal beams.

3.12.3 The reduction in intensity of loads on transverse members such as floor beams shall be determined as in the case of main trusses or girders, using the number of traffic lanes across the width of roadway that must be loaded to produce maximum stresses in the floor beam.

3.13 ELECTRIC RAILWAY LOADS

If highway bridges carry electric railway traffic, the railway loads shall be determined from the class of traffic which the bridge may be expected to carry. The possibility that the bridge may be required to carry railroad freight cars shall be given consideration.

3.14 SIDEWALK, CURB, AND RAILING LOADING

3.14.1 Sidewalk Loading

3.14.1.1 Sidewalk floors, stringers, and their immediate supports shall be designed for a live load of 85 pounds per square foot of sidewalk area. Girders, trusses, arches, and other members shall be designed for the following sidewalk live loads:

Spans 0 to 25 feet in length ....85 lb./ft.^{2}
Spans 26 to 100 feet in length ....60 lb./ft.^{2}
Spans over 100 feet in length according to the formula

$$P = \left(30 + \frac{3{,}000}{L}\right)\left(\frac{55 - W}{50}\right) \qquad (3\text{-}3)$$

in which

P = live load per square foot, max. 60-lb. per sq. ft.
L = loaded length of sidewalk in feet.
W = width of sidewalk in feet.

3.14.1.2 In calculating stresses in structures that support cantilevered sidewalks, the sidewalk shall be fully loaded on only one side of the structure if this condition produces maximum stress.

3.14.1.3 Bridges for pedestrian and/or bicycle traffic shall be designed for a live load of 85 PSF.

3.14.1.4 Where bicycle or pedestrian bridges are expected to be used by maintenance vehicles, special design consideration should be made for these loads.

3.14.2 Curb Loading

3.14.2.1 Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot of curb, applied at the top of the curb, or at an elevation 10 inches above the floor if the curb is higher than 10 inches.

3.14.2.2 Where sidewalk, curb, and traffic rail form an integral system, the traffic railing loading shall be applied and stresses in curbs computed accordingly.

3.14.3 Railing Loading

For Railing Loads, see Article 2.7.1.3.

3.15 WIND LOADS

The wind load shall consist of moving uniformly distributed loads applied to the exposed area of the structure. The exposed area shall be the sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. The forces and loads given herein are for a base wind velocity of 100 miles per hour. For Group II and Group V loadings, but not for Group III and Group VI loadings, they may be reduced or increased in the ratio of the square of the design wind velocity to the square of the base wind velocity provided that the maximum probable wind velocity can be ascertained with reasonable accuracy, or provided that there are permanent features of the terrain which make such changes safe and advisable. If a change in the design wind velocity is made, the design wind velocity shall be shown on the plans.

3.15.1 Superstructure Design

3.15.1.1 Group II and Group V Loadings

3.15.1.1.1 A wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure:

For trusses and arches ....75 pounds per square foot
For girders and beams ....50 pounds per square foot

3.15.1.1.2 The total force shall not be less than 300 pounds per linear foot in the plane of the windward chord and 150 pounds per linear foot in the plane of the leeward chord on truss spans, and not less than 300 pounds per linear foot on girder spans.

3.15.1.2 Group III and Group VI Loadings

Group III and Group VI loadings shall comprise the loads used for Group II and Group V loadings reduced by 70% and a load of 100 pounds per linear foot applied at right angles to the longitudinal axis of the structure and 6 feet above the deck as a wind load on a moving live load.

forces transverse to the longitudinal axis shall in no case be taken as less than 20% of the total force.

3.18.2.2.7 In the case of slender and flexible piers, consideration should be given to the vibrating nature of dynamic ice forces and to the possibility of high momentary pressures and structural resonance.

3.18.2.3 Static Ice Pressure

Ice pressure on piers frozen into ice sheets on large bodies of water shall receive special consideration where there is reason to believe that the ice sheets are subject to significant thermal movements relative to the piers.

3.19 BUOYANCY

Buoyancy shall be considered where it affects the design of either substructure, including piling, or the superstructure.

3.20 EARTH PRESSURE

3.20.1 Structures which retain fills shall be proportioned to withstand pressure as given by Coulomb's Equation or by other expressions given in Section 5, "Retaining Walls"; provided, however, that no structure shall be designed for less than an equivalent fluid weight (mass) of 30 pounds per cubic foot.

3.20.2 For rigid frames a maximum of one-half of the moment caused by earth pressure (lateral) may be used to reduce the positive moment in the beams, in the top slab, or in the top and bottom slab, as the case may be.

3.20.3 When highway traffic can come within a horizontal distance from the top of the structure equal to one-half its height, the pressure shall have added to it a live load surcharge pressure equal to not less than 2 feet of earth.

3.20.4 Where an adequately designed reinforced concrete approach slab supported at one end by the bridge is provided, no live load surcharge need be considered.

3.20.5 All designs shall provide for the thorough drainage of the back-filling material by means of weep holes and crushed rock, pipe drains or gravel drains, or by perforated drains.

3.21 EARTHQUAKES

In regions where earthquakes may be anticipated, structures shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure in accordance with Division I-A—Seismic Design.

Part B
COMBINATIONS OF LOADS

3.22 COMBINATIONS OF LOADS

3.22.1 The following Groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned to withstand safely all group combinations of these forces that are applicable to the particular site or type. Group loading combinations for Service Load Design and Load Factor Design are given by:

$$\begin{aligned}\text{Group (N)} = \gamma[&\beta_D \cdot D + \beta_L (L + I) + \beta_C CF + \beta_E E \\ &+ \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL \\ &+ \beta_L \cdot LF + \beta_R (R + S + T) \\ &+ \beta_{EQ} EQ + \beta_{ICE} ICE]\end{aligned} \tag{3-10}$$

where,

N = group number;
γ = load factor, see Table 3.22.1A;
β = coefficient, see Table 3.22.1A;
D = dead load;
L = live load;
I = live load impact;
E = earth pressure;
B = buoyancy;
W = wind load on structure;
WL = wind load on live load—100 pounds per linear foot;
LF = longitudinal force from live load;
CF = centrifugal force;
R = rib shortening;
S = shrinkage;
T = temperature;
EQ = earthquake;
SF = stream flow pressure;
ICE = ice pressure.

TABLE 3.22.1A Table of Coefficients γ and β

	Col. No.	1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
			β FACTORS														
	GROUP	γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE	%	
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
SERVICE LOAD	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
SERVICE LOAD	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
SERVICE LOAD	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
SERVICE LOAD	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
SERVICE LOAD	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
SERVICE LOAD	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
SERVICE LOAD	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
SERVICE LOAD	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
SERVICE LOAD	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
SERVICE LOAD	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
SERVICE LOAD	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
LOAD FACTOR DESIGN	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0	Not Applicable	
LOAD FACTOR DESIGN	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
LOAD FACTOR DESIGN	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0	Not Applicable	
LOAD FACTOR DESIGN	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0	Not Applicable	
LOAD FACTOR DESIGN	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0	Not Applicable	
LOAD FACTOR DESIGN	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0	Not Applicable	
LOAD FACTOR DESIGN	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0	Not Applicable	
LOAD FACTOR DESIGN	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0	Not Applicable	
LOAD FACTOR DESIGN	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1	Not Applicable	
LOAD FACTOR DESIGN	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1	Not Applicable	
LOAD FACTOR DESIGN	X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0		Culvert

$(L + I)_n$ - Live load plus impact for AASHTO Highway H or HS loading
$(L + I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

* 1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.

$$** \text{ Percentage} = \frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$$

For Service Load Design

% (Column 14) Percentage of Basic Unit Stress

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = 1.00 for vertical and lateral loads on all other structures.

For culvert loading specifications, see Article 6.2.

β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Article 3.20.

For Load Factor Design

β_E = 1.3 for lateral earth pressure for retaining walls and rigid frames excluding rigid culverts. For lateral at-rest earth pressures, β_E = 1.15

β_E = 0.5 for lateral earth pressure when checking positive moments in rigid frames. This complies with Article 3.20.

β_E = 1.0 for vertical earth pressure

β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity . . . . . For Column Design

β_D = 1.0 when checking member for maximum axial load and minimum moment . . . . Column Design

β_D = 1.0 for flexural and tension members

β_E = 1.0 for Rigid Culverts

β_E = 1.5 for Flexible Culverts

For Group X loading (culverts) the β_E factor shall be applied to vertical and horizontal loads.

3.22.2 For service load design, the percentage of the basic unit stress for the various groups is given in Table 3.22.1A.

The loads and forces in each group shall be taken as appropriate from Articles 3.3 to 3.21. The maximum section required shall be used.

3.22.3 For load factor design, the gamma and beta factors given in Table 3.22.1A shall be used for designing structural members and foundations by the load factor concept.

3.22.4 When long span structures are being designed by load factor design, the gamma and beta factors specified for Load Factor Design represent general conditions and should be increased if, in the Engineer's judgment, expected loads, service conditions, or materials of construction are different from those anticipated by the specifications.

3.22.5 Structures may be analyzed for an overload that is selected by the operating agency. Size and configuration of the overload, loading combinations, and load distribution will be consistent with procedures defined in permit policy of that agency. The load shall be applied in Group IB as defined in Table 3.22.1A. For all loadings less than H 20, Group IA loading combination shall be used (see Article 3.5).

Part C
DISTRIBUTION OF LOADS

3.23 DISTRIBUTION OF LOADS TO STRINGERS, LONGITUDINAL BEAMS, AND FLOOR BEAMS*

3.23.1 Position of Loads for Shear

3.23.1.1 In calculating end shears and end reactions in transverse floor beams and longitudinal beams and stringers, no longitudinal distribution of the wheel load shall be assumed for the wheel or axle load adjacent to the transverse floor beam or the end of the longitudinal beam or stringer at which the stress is being determined.

3.23.1.2 Lateral distribution of the wheel loads at ends of the beams or stringers shall be that produced by assuming the flooring to act as a simple span between stringers or beams. For wheels or axles in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment, except that the calculations of horizontal shear in rectangular timber beams shall be in accordance with Article 13.3.

*Provisions in this Article shall not apply to orthotropic deck bridges.

3.23.2 Bending Moments in Stringers and Longitudinal Beams**

3.23.2.1 General

In calculating bending moments in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows.

3.23.2.2 Interior Stringers and Beams

The live load bending moment for each interior stringer shall be determined by applying to the stringer the fraction of a wheel load (both front and rear) determined in Table 3.23.1.

3.23.2.3 Outside Roadway Stringers and Beams

3.23.2.3.1 Steel-Timber-Concrete T-Beams

3.23.2.3.1.1 The dead load supported by the outside roadway stringer or beam shall be that portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surface, if placed after the slab has cured, may be distributed equally to all roadway stringers or beams.

3.23.2.3.1.2 The live load bending moment for outside roadway stringers or beams shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams.

3.23.2.3.1.3 When the outside roadway beam or stringer supports the sidewalk live load as well as traffic live load and impact and the structure is to be designed by the service load method, the allowable stress in the beam or stringer may be increased by 25% for the combination of dead load, sidewalk live load, traffic live load, and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalks. When the combination of sidewalk live load and traffic live load plus impact governs the design and the structure is to be designed by the load factor method, 1.25 may be used as the beta factor in place of 1.67.

3.23.2.3.1.4 In no case shall an exterior stringer have less carrying capacity than an interior stringer.

**In view of the complexity of the theoretical analysis involved in the distribution of wheel loads to stringers, the empirical method herein described is authorized for the design of normal highway bridges.

TABLE 3.23.1 Distribution of Wheel Loads in Longitudinal Beams

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or more Traffic Lanes
Timber:[a]		
Plank[b]	S/4.0	S/3.75
Nail laminated[c] 4" thick or multiple layer[d] floors over 5" thick	S/4.5	S/4.0
Nail laminated[c] 6" or more thick	S/5.0 If S exceeds 5′ use footnote f.	S/4.25 If S exceeds 6.5′ use footnote f.
Glued laminated[e] Panels on glued laminated stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/6.0 If S exceeds 6′ use footnote f.	S/5.0 If S exceeds 7.5′ use footnote f.
On steel stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/5.25 If S exceeds 5.5′ use footnote f.	S/4.5 If S exceeds 7′ use footnote f.
Concrete:		
On steel I-Beam stringers[g] and prestressed concrete girders	S/7.0 If S exceeds 10′ use footnote f.	S/5.5 If S exceeds 14′ use footnote f.
On concrete T-Beams	S/6.5 If S exceeds 6′ use footnote f.	S/6.0 If S exceeds 10′ use footnote f.
On timber stringers	S/6.0 If S exceeds 6′ use footnote f.	S/5.0 If S exceeds 10′ use footnote f.
Concrete box girders[h]	S/8.0 If S exceeds 12′ use footnote f.	S/7.0 If S exceeds 16′ use footnote f.
On steel box girders	See Article 10.39.2.	
On prestressed concrete spread box Beams	See Article 3.28.	
Steel grid:		
(Less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6′ use footnote f.	S/5.0 If S exceeds 10.5′ use footnote f.
Steel bridge Corrugated plank[i] (2" min. depth)	S/5.5	S/4.5

S = average stringer spacing in feet.

[a]Timber dimensions shown are for nominal thickness.

[b]Plank floors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports (see Article 16.3.11—Division II).

[c]Nail laminated floors consist of pieces of lumber laid face to face with the narrow edges bearing on the supports, each piece being nailed to the preceding piece (see Article 16.3.12—Division II).

[d]Multiple layer floors consist of two or more layers of planks, each layer being laid at an angle to the other (see Article 16.3.11—Division II).

[e]Glued laminated panel floors consist of vertically glued laminated members with the narrow edges of the laminations bearing on the supports (see Article 16.3.13—Division II).

[f]In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam.

[g]"Design of I-Beam Bridges" by N. M. Newmark—Proceedings, ASCE, March 1948.

[h]The sidewalk live load (see Article 3.14) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.

[i]Distribution factors for Steel Bridge Corrugated Plank set forth above are based substantially on the following reference:

Journal of Washington Academy of Sciences, Vol. 67, No. 2, 1977 "Wheel Load Distribution of Steel Bridge Plank," by Conrad P. Heins, Professor of Civil Engineering, University of Maryland.

These distribution factors were developed based on studies using 6" × 2" steel corrugated plank. The factors should yield safe results for other corrugated configurations provided primary bending stiffness is the same as or greater than the 6" × 2" corrugated plank used in the studies.

3.23.2.3.1.5 In the case of a span with concrete floor supported by 4 or more steel stringers, the fraction of the wheel load shall not be less than:

$$\frac{S}{5.5}$$

where, S = 6 feet or less and is the distance in feet between outside and adjacent interior stringers, and

$$\frac{S}{4.0 + 0.25S}$$

where, S is more than 6 feet and less than 14 feet. When S is 14 feet or more, use footnote f, Table 3.23.1.

3.23.2.3.2 Concrete Box Girders

3.23.2.3.2.1 The dead load supported by the exterior girder shall be determined in the same manner as for steel, timber, or concrete T-beams, as given in Article 3.23.2.3.1.

3.23.2.3.2.2 The factor for the wheel load distribution to the exterior girder shall be $W_e/7$, where W_e is the width of exterior girder which shall be taken as the top slab width, measured from the midpoint between girders to the outside edge of the slab. The cantilever dimension of any slab extending beyond the exterior girder shall preferably not exceed half the girder spacing.

3.23.2.3.3 Total Capacity of Stringers and Beams

The combined design load capacity of all the beams and stringers in a span shall not be less than required to support the total live and dead load in the span.

3.23.3 Bending Moments in Floor Beams (Transverse)

3.23.3.1 In calculating bending moments in floor beams, no transverse distribution of the wheel loads shall be assumed.

3.23.3.2 If longitudinal stringers are omitted and the floor is supported directly on floor beams, the beams shall be designed for loads determined in accordance with Table 3.23.3.1.

TABLE 3.23.3.1 Distribution of Wheel Loads in Transverse Beams

Kind of Floor	Fraction of Wheel Load to Each Floor Beam
Plank[a,b]	$\frac{S}{4}$
Nail laminated[c] or glued laminated[e], 4 inches in thickness, or multiple layer[d] floors more than 5 inches thick	$\frac{S}{4.5}$
Nail laminated[c] or glued laminated[e], 6 inches or more in thickness	$\frac{S^f}{5}$
Concrete	$\frac{S^f}{6}$
Steel grid (less than 4 inches thick)	$\frac{S}{4.5}$
Steel grid (4 inches or more)	$\frac{S^f}{6}$
Steel bridge corrugated plank (2 inches minimum depth)	$\frac{S}{5.5}$

Note:
S = spacing of floor beams in feet.

[a–e]For footnotes a through e, see Table 3.23.1.

[f]If S exceeds denominator, the load on the beam shall be the reaction of the wheels loads assuming the flooring between beams to act as a simple beam.

3.23.4 Precast Concrete Beams Used in Multi-Beam Decks

3.23.4.1 A multi-beam bridge is constructed with precast reinforced or prestressed concrete beams that are placed side by side on the supports. The interaction between the beams is developed by continuous longitudinal shear keys used in combination with transverse tie assemblies which may, or may not, be prestressed, such as bolts, rods, or prestressing strands, or other mechanical means. Full-depth rigid end diaphragms are needed to ensure proper load distribution for channel, single- and multi-stemmed tee beams.

3.23.4.2 In calculating bending moments in multi-beam precast concrete bridges, conventional or prestressed, no longitudinal distribution of wheel load shall be assumed.

3.23.4.3 The live load bending moment for each section shall be determined by applying to the beam the fraction of a wheel load (both front and rear) determined by the following equation:

$$\text{Load Fraction} = \frac{S}{D} \tag{3-11}$$

where,

S = width of precast member;

$$D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2 \tag{3-12}$$

N_L = number of traffic lanes from Article 3.6;

$C = K(W/L)$ fow $W/L < 1$

$= K$ for $W/L \geq 1$ (3-13)

where,

W = overall width of bridge measured perpendicular to the longitudinal girders in feet;

L = span length measured parallel to longitudinal girders in feet; for girders with cast-in-place end diaphragms, use the length between end diaphragms;

$K = \{(1 + \mu)\, I/J\}^{1/2}$

If the value of $\sqrt{I/J}$ exceeds 5.0, or the skew exceeds 45 degrees, the live load distribution should be determined using a more precise method, such as the Articulate Plate Theory or Grillage Analysis. The Load Fraction, S/D, need not be greater than 1.

where,

I = moment of inertia;

J = Saint-Venant torsion constant;

μ = Poisson's ratio for girders.

In lieu of more exact methods, "J" may be estimated using the following equations:

For Non-voided Rectangular Beams, Channels, Tee Beams:

$$J = \Sigma\{(1/3)bt^3(1 - 0.630t/b)\}$$

where,

b = the length of each rectangular component within the section,
t = the thickness of each rectangular component within the section.

The flanges and stems of stemmed or channel sections are considered as separate rectangular components whose values are summed together to calculate "J". Note that for "Rectangular Beams with Circular Voids" the value of "J" can usually be approximated by using the equation above for rectangular sections and neglecting the voids.

For Box-Section Beams:

$$J = \frac{2tt_f(b-t)^2(d-t_f)^2}{bt + dt_f - t^2 - t_f^2}$$

where

b = the overall width of the box,
d = the overall depth of the box,
t = the thickness of either web,
t_f = the thickness of either flange.

The formula assumes that both flanges are the same thickness and uses the thickness of only one flange. The same is true of the webs.

For preliminary design, the following values of K may be used:

Bridge Type	Beam Type	K
Multi-beam	Non-voided rectangular beams	0.7
	Rectangular beams with circular voids	0.8
	Box section beams	1.0
	Channel, single- and multi-stemmed tee beams	2.2

3.24 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS*

3.24.1 Span Lengths (See Article 8.8)

3.24.1.1 For simple spans the span length shall be the distance center to center of supports but need not exceed clear span plus thickness of slab.

3.24.1.2 The following effective span lengths shall be used in calculating the distribution of loads and bending moments for slabs continuous over more than two supports:

(a) Slabs monolithic with beams or slabs monolithic with walls without haunches and rigid top flange prestressed beams with top flange width to minimum thickness ratio less than 4.0. "S" shall be the clear span.
(b) Slabs supported on steel stringers, or slabs supported on thin top flange prestressed beams with top flange width to minimum thickness ratio equal to or greater than 4.0. "S" shall be the distance between edges of top flange plus one-half of stringer top flange width.
(c) Slabs supported on timber stringers. S shall be the clear span plus one-half thickness of stringer.

3.24.2 Edge Distance of Wheel Loads

3.24.2.1 In designing slabs, the center line of the wheel load shall be 1 foot from the face of the curb. If curbs or sidewalks are not used, the wheel load shall be 1 foot from the face of the rail.

3.24.2.2 In designing sidewalks, slabs and supporting members, a wheel load located on the sidewalk shall be 1 foot from the face of the rail. In service load design, the combined dead, live, and impact stresses for this loading shall be not greater than 150% of the allowable stresses. In load factor design, 1.0 may be used as the beta factor in place of 1.67 for the design of deck slabs. Wheel loads shall not be applied on sidewalks protected by a traffic barrier.

3.24.3 Bending Moment

The bending moment per foot width of slab shall be calculated according to methods given under Cases A and

*The slab distribution set forth herein is based substantially on the "Westergaard" theory. The following references are furnished concerning the subject of slab design.
Public Roads, March 1930, "Computation of Stresses in Bridge Slabs Due to Wheel Loads," by H. M. Westergaard.
University of Illinois, Bulletin No. 303, "Solutions for Certain Rectangular Slabs Continuous over Flexible Supports," by Vernon P. Jensen; Bulletin 304, "A Distribution Procedure for the Analysis of Slabs Continuous over Flexible Beams," by Nathan M. Newmark; Bulletin 315, "Moments in Simple Span Bridge Slabs with Stiffened Edges," by Vernon P. Jensen; and Bulletin 346, "Highway Slab Bridges with Curbs; Laboratory Tests and Proposed Design Method."

B, unless more exact methods are used considering tire contact area. The tire contact area needed for exact methods is given in Article 3.30.

In Cases A and B:

S = effective span length, in feet, as defined under "Span Lengths" Articles 3.24.1 and 8.8;
E = width of slab in feet over which a wheel load is distributed;
P = load on one rear wheel of truck (P_{15} or P_{20});
P_{15} = 12,000 pounds for H 15 loading;
P_{20} = 16,000 pounds for H 20 loading.

3.24.3.1 Case A—Main Reinforcement Perpendicular to Traffic (Spans 2 to 24 Feet Inclusive)

The live load moment for simple spans shall be determined by the following formulas (impact not included):

HS 20 Loading:

$$\left(\frac{S+2}{32}\right)P_{20} = \text{Moment in foot-pounds per foot-width of slab} \quad (3\text{-}15)$$

HS 15 Loading:

$$\left(\frac{S+2}{32}\right)P_{15} = \text{Moment in foot-pounds per foot-width of slab} \quad (3\text{-}16)$$

In slabs continuous over three or more supports, a continuity factor of 0.8 shall be applied to the above formulas for both positive and negative moment.

3.24.3.2 Case B—Main Reinforcement Parallel to Traffic

For wheel loads, the distribution width, E, shall be (4 + 0.06S) but shall not exceed 7.0 feet. Lane loads are distributed over a width of 2E. Longitudinally reinforced slabs shall be designed for the appropriate HS loading.

For simple spans, the maximum live load moment per foot width of slab, without impact, is closely approximated by the following formulas:

HS 20 Loading:

Spans up to and including 50 feet: LLM = 900S foot-pounds
Spans 50 feet to 100 feet: LLM = 1,000 (1.30S-20.0) foot-pounds

HS 15 Loading:

Use ¾ of the values obtained from the formulas for HS 20 Loading

Moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading.

3.24.4 Shear and Bond

Slabs designed for bending moment in accordance with Article 3.24.3 shall be considered satisfactory in bond and shear.

3.24.5 Cantilever Slabs

3.24.5.1 Truck Loads

Under the following formulas for distribution of loads on cantilever slabs, the slab is designed to support the load independently of the effects of any edge support along the end of the cantilever. The distribution given includes the effect of wheels on parallel elements.

3.24.5.1.1 Case A—Reinforcement Perpendicular to Traffic

Each wheel on the element perpendicular to traffic shall be distributed over a width according to the following formula:

$$E = 0.8X + 3.75 \quad (3\text{-}17)$$

The moment per foot of slab shall be (P/E) X foot-pounds, in which X is the distance in feet from load to point of support.

3.24.5.1.2 Case B—Reinforcement Parallel to Traffic

The distribution width for each wheel load on the element parallel to traffic shall be as follows:

$$E = 0.35X + 3.2\text{, but shall not exceed 7.0 feet} \quad (3\text{-}18)$$

The moment per foot of slab shall be (P/E) X foot-pounds.

3.24.5.2 Railing Loads

Railing loads shall be applied in accordance with Article 2.7. The effective length of slab resisting post loadings shall be equal to E = 0.8X + 3.75 feet where no parapet is used and equal to E = 0.8X + 5.0 feet where a parapet is used, where X is the distance in feet from the center of the post to the point under investigation. Railing and wheel loads shall not be applied simultaneously.

3.24.6 Slabs Supported on Four Sides

3.24.6.1 For slabs supported along four edges and reinforced in both directions, the proportion of the load carried by the short span of the slab shall be given by the following equations:

$$\text{For uniformly distributed load, } p = \frac{b^4}{a^4 + b^4} \qquad (3\text{-}19)$$

$$\text{For concentrated load at center, } p = \frac{b^3}{a^3 + b^3} \qquad (3\text{-}20)$$

where,

p = proportion of load carried by short span;
a = length of short span of slab;
b = length of long span of slab.

3.24.6.2 Where the length of the slab exceeds 1½ times its width, the entire load shall be carried by the transverse reinforcement.

3.24.6.3 The distribution width, E, for the load taken by either span shall be determined as provided for other slabs. The moments obtained shall be used in designing the center half of the short and long slabs. The reinforcement steel in the outer quarters of both short and long spans may be reduced by 50%. In the design of the supporting beams, consideration shall be given to the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams.

3.24.7 Median Slabs

Raised median slabs shall be designed in accordance with the provisions of this article with truck loadings so placed as to produce maximum stresses. Combined dead, live, and impact stresses shall not be greater than 150% of the allowable stresses. Flush median slabs shall be designed without overstress.

3.24.8 Longitudinal Edge Beams

3.24.8.1 Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb.

3.24.8.2 The edge beam of a simple span shall be designed to resist a live load moment of 0.10 PS, where,

P = wheel load in pounds P_{15} or P_{20};
S = span length in feet.

3.24.8.3 For continuous spans, the moment may be reduced by 20% unless a greater reduction results from a more exact analysis.

3.24.9 Unsupported Transverse Edges

The design assumptions of this article do not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges shall be supported by diaphragms or other suitable means. The diaphragms shall be designed to resist the full moment and shear produced by the wheel loads which can come on them.

3.24.10 Distribution Reinforcement

3.24.10.1 To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottoms of all slabs except culvert or bridge slabs where the depth of fill over the slab exceeds 2 feet.

3.24.10.2 The amount of distribution reinforcement shall be the percentage of the main reinforcement steel required for positive moment as given by the following formulas:

For main reinforcement parallel to traffic,

$$\text{Percentage} = \frac{100}{\sqrt{S}} \text{ Maximum } 50\% \qquad (3\text{-}21)$$

For main reinforcement perpendicular to traffic,

$$\text{Percentage} = \frac{220}{\sqrt{S}} \text{ Maximum } 67\% \qquad (3\text{-}22)$$

where, S = the effective span length in feet.

3.24.10.3 For main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement shall be used in the middle half of the slab span, and not less than 50% of the specified amount shall be used in the outer quarters of the slab span.

3.25 DISTRIBUTION OF WHEEL LOADS ON TIMBER FLOORING

For the calculation of bending moments in timber flooring each wheel load shall be distributed as follows.

3.25.1 Transverse Flooring

3.25.1.1 In the direction of flooring span, the wheel load shall be distributed over the width of tire as given in Article 3.30.

Normal to the direction of flooring span, the wheel load shall be distributed as follows:

Plank floor: the width of plank, but not less than 10 inches.

Non-interconnected* nail laminated panel floor: 15 inches, but not to exceed panel width.

Non-interconnected glued laminated panel floor: 15 inches plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated panel floor, with adequate shear transfer between panels**: 15 inches plus thickness of floor, but not to exceed panel width.

Interconnected* glued laminated panel floor, with adequate shear transfer between panels**, not less than 6 inches thick: 15 inches plus twice thickness of floor, but not to exceed panel width.

3.25.1.2 For transverse flooring the span shall be taken as the clear distance between stringers plus one-half the width of one stringer, but shall not exceed the clear span plus the floor thickness.

3.25.1.3 One design method for interconnected glued laminated panel floors is as follows: For glued laminated panel decks using vertically laminated lumber with the panel placed in a transverse direction to the stringers and with panels interconnected using steel dowels, the determination of the deck thickness shall be based on the following equations for maximum unit primary moment and shear.† The maximum shear is for a wheel position assumed to be 15 inches or less from the center line of the support. The maximum moment is for a wheel position assumed to be centered between the supports.

$$M_x = P(.51 \log_{10} s - K) \qquad (3\text{-}23)$$

$$\mathrm{R_x} = .034P \qquad (3\text{-}24)$$

Thus,

$$t = \sqrt{\frac{6\mathrm{M_x}}{\mathrm{F_b}}} \qquad (3\text{-}25)$$

or,

$$t = \frac{3\mathrm{R_x}}{2\mathrm{F_v}} \textit{ whichever is greater} \qquad (3\text{-}26)$$

where,

- $\mathrm{M_x}$ = primary bending moment in inch-pounds per inch;
- $\mathrm{R_x}$ = primary shear in pounds per inch;
- x = denotes direction perpendicular to longitudinal stringers;
- P = design wheel load in pounds;
- s = effective deck span in inches;
- t = deck thickness, in inches, based on moment or shear, whichever controls;
- K = design constant depending on design load as follows:

  H 15 K = 0.47

  H 20 K = 0.51

- $\mathrm{F_b}$ = allowable bending stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B);
- $\mathrm{F_v}$ = allowable shear stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B).

3.25.1.4 The determination of the minimum size and spacing required of the steel dowels required to transfer the load between panels shall be based on the following equation:

$$\mathrm{n} = \frac{1{,}000}{\sigma_{\mathrm{PL}}} \times \left[\frac{\overline{\mathrm{R}}_\mathrm{y}}{\mathrm{R_D}} + \frac{\overline{\mathrm{M}}_\mathrm{y}}{\mathrm{M_D}}\right] \qquad (3\text{-}27)$$

where,

- n = number of steel dowels required for the given spans;
- σ_{PL} = proportional limit stress perpendicular to grain (for Douglas fir or Southern pine, use 1,000 psi);
- $\overline{\mathrm{R}}_\mathrm{y}$ = total secondary shear transferred, in pounds, determined by the relationship:

*The terms interconnected and non-interconnected refer to the joints between the individual nail laminated or glued laminated panels.

**This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or other suitable means.

†The equations are developed for deck panel spans equal to or greater than the width of the tire (as specified in Article 3.30), but not greater than 200 inches.

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Section 6
CULVERTS

6.1 CULVERT LOCATION, LENGTH, AND WATERWAY OPENINGS

Recommendations on culvert location, length, and waterway openings are given in the AASHTO *Guide on Hydraulic Design of Culverts.*

6.2 DEAD LOADS

Vertical and horizontal earth pressures on culverts may be computed by recognized or appropriately documented analytical techniques based on the principles of soil mechanics and soil structure interaction, or design pressures shall be calculated as being the result of an equivalent fluid weight as follows.

6.2.1 Culvert in trench, or culvert untrenched on yielding foundation

A. Rigid culverts except reinforced concrete boxes:
  - (1) For vertical earth pressure—120 pcf
    For lateral earth pressure— 30 pcf
  - (2) For vertical earth pressure—120 pcf
    For lateral earth pressure— 120 pcf

B. Reinforced concrete boxes:
  - (1) For vertical earth pressure—120 pcf
    For lateral earth pressure— 30 pcf
  - (2) For vertical earth pressure—120 pcf
    For lateral earth pressure— 60 pcf

C. Flexible Culverts:
  - For vertical earth pressure—120 pcf
  - For lateral earth pressure— 120 pcf

When concrete pipe culverts are designed by the Indirect Design Method of Article 16.4.5, the design lateral earth pressure shall be determined using the procedures given in Article 16.4.5.2.1 for embankment installations and in Article 16.4.5.2.2 for trench installations.

6.2.2 Culvert untrenched on unyielding foundation

A special analysis is required.

6.3 FOOTINGS

Footings for culverts shall be carried to an elevation sufficient to secure a firm foundation, or a heavy reinforced floor shall be used to distribute the pressure over the entire horizontal area of the structure. In any location subject to erosion, aprons or cutoff walls shall be used at both ends of the culvert and, where necessary, the entire floor area between the wing walls shall be paved. Baffle walls or struts across the unpaved bottom of a culvert barrel shall not be used where the stream bed is subject to erosion. When conditions require, culvert footings shall be reinforced longitudinally.

6.4 DISTRIBUTION OF WHEEL LOADS THROUGH EARTH FILLS

6.4.1 When the depth of fill is 2 feet or more, concentrated loads shall be considered as uniformly distributed over a square with sides equal to 1¾ times the depth of fill.

6.4.2 When such areas from several concentrations overlap, the total load shall be uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab. For single spans, the effect of live load may be neglected when the depth of fill is more than 8 feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between faces of end supports or abutments. When the depth of fill is less than 2 feet the wheel load shall be distributed as in slabs with concentrated loads. When the calculated live load and impact moment in concrete slabs, based on the distribution of the wheel load through earth fills, exceeds the live load and impact moment calculated according to Article 3.24, the latter moment shall be used.

6.5 DISTRIBUTION REINFORCEMENT

Where the depth of fill exceeds 2 feet, reinforcement to provide for the lateral distribution of concentrated loads is not required.

6.6 DESIGN

For culvert design guidelines, see Section 16.

formly distributed within two-thirds of the effective depth adjacent to A_s.

8.15.5.8.5 Ratio $\rho = A_s/bd$ shall not be taken less than $0.04(f_c'/f_y)$.

8.15.5.8.6 At the front face of a bracket or corbel, primary tension reinforcement, A_s, shall be anchored by one of the following:

(a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;
(b) bending primary tension bars A_s back to form a horizontal loop; or
(c) some other means of positive anchorage.

8.15.5.8.7 Bearing area of load on a bracket or corbel shall not project beyond the straight portion of primary tension bars A_s, nor project beyond the interior face of a transverse anchor bar (if one is provided).

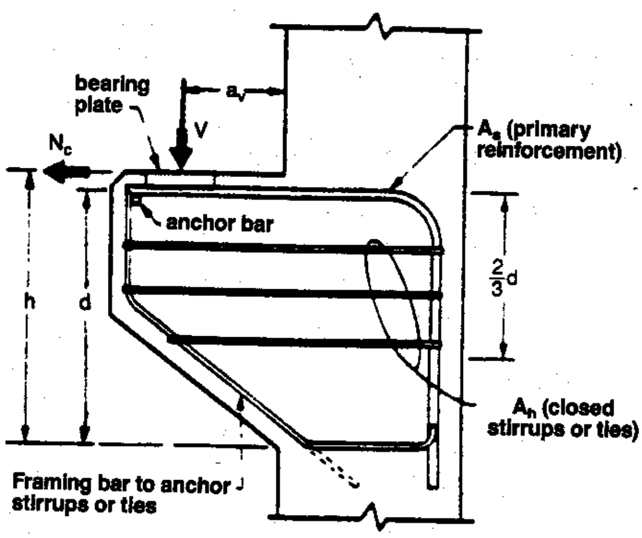


FIGURE 8.15.5.8

8.16 STRENGTH DESIGN METHOD (LOAD FACTOR DESIGN)

8.16.1 Strength Requirements

8.16.1.1 Required Strength

The required strength of a section is the strength necessary to resist the factored loads and forces applied to the structure in the combinations stipulated in Article 3.22. All sections of structures and structural members shall have design strengths at least equal to the required strength.

8.16.1.2 Design Strength

8.16.1.2.1 The design strength provided by a member or cross section in terms of load, moment, shear, or stress shall be the nominal strength calculated in accordance with the requirements and assumptions of the strength-design method, multiplied by a strength-reduction factor ϕ.*

8.16.1.2.2 The strength-reduction factors, ϕ, shall be as follows:

(a) Flexure . . . . $\phi = 0.90$
(b) Shear . . . . $\phi = 0.85$
(c) Axial compression with—
  Spirals . . . . $\phi = 0.75$
  Ties . . . . $\phi = 0.70$
(d) Bearing on concrete . . . . $\phi = 0.70$

The value of ϕ may be increased linearly from the value for compression members to the value for flexure as the design axial load strength, ϕP_n, decreases from $0.10f_c'A_g$ or ϕP_b, whichever is smaller, to zero.

8.16.1.2.3 The development and splice lengths of reinforcement specified in Articles 8.24 through 8.32 do not require a strength-reduction factor.

8.16.2 Design Assumptions

8.16.2.1 The strength design of members for flexure and axial loads shall be based on the assumptions given in this article, and on the satisfaction of the applicable conditions of equilibrium of internal stresses and compatibility of strains.

8.16.2.2 The strain in reinforcement and concrete is directly proportional to the distance from the neutral axis.

8.16.2.3 The maximum usable strain at the extreme concrete compression fiber is equal to 0.003.

*The coefficient ϕ provides for the possibility that small adverse variations in material strengths, workmanship, and dimensions, while individually within acceptable tolerances and limits of good practice, may combine to result in understrength.

8.16.2.4 The stress in reinforcement below its specified yield strength, f_y, shall be E_s times the steel strain. For strains greater than that corresponding to f_y, the stress in the reinforcement shall be considered independent of strain and equal to f_y.

8.16.2.5 The tensile strength of the concrete is neglected in flexural calculations.

8.16.2.6 The concrete compressive stress/strain distribution may be assumed to be a rectangle, trapezoid, parabola, or any other shape that results in prediction of strength in substantial agreement with the results of comprehensive tests.

8.16.2.7 A compressive stress/strain distribution, which assumes a concrete stress of 0.85 f'_c uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a line parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain, may be considered to satisfy the requirements of Article 8.16.2.6. The distance c from the fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis. The factor β_1 shall be taken as 0.85 for concrete strengths, f_c', up to and including 4,000 psi. For strengths above 4,000 psi, β_1 shall be reduced continuously at a rate of 0.05 for each 1,000 psi of strength in excess of 4,000 psi but β_1 shall not be taken less than 0.65.

8.16.3 Flexure

8.16.3.1 Maximum Reinforcement of Flexural Members

8.16.3.1.1 The ratio of reinforcement ρ provided shall not exceed 0.75 of the ratio ρ_b that would produce balanced strain conditions for the section. The portion of ρ_b balanced by compression reinforcement need not be reduced by the 0.75 factor.

8.16.3.1.2 Balanced strain conditions exist at a cross section when the tension reinforcement reaches the strain corresponding to its specified yield strength, f_y, just as the concrete in compression reaches its assumed ultimate strain of 0.003.

8.16.3.2 Rectangular Sections with Tension Reinforcement Only

8.16.3.2.1 The design moment strength, ϕM_n, may be computed by:

$$\phi M_n = \phi\left[A_s f_y d\left(1 - 0.6\frac{\rho f_y}{f'_c}\right)\right] \qquad (8\text{-}15)$$

$$= \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right] \qquad (8\text{-}16)$$

where,

$$a = \frac{A_s f_y}{0.85\, f'_c b} \qquad (8\text{-}17)$$

8.16.3.2.2 The balanced reinforcement ratio, ρ_b, is given by:

$$\rho_b = \frac{0.85\,\beta_1 f'_c}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right) \qquad (8\text{-}18)$$

8.16.3.3 Flanged Sections with Tension Reinforcement Only

8.16.3.3.1 When the compression flange thickness is equal to or greater than the depth of the equivalent rectangular stress block, a, the design moment strength, ϕM_n, may be computed by Equations (8-15) and (8-16).

8.16.3.3.2 When the compression flange thickness is less than a, the design moment strength may be computed by:

$$\phi M_n = \phi[(A_s - A_{sf})f_y(d - a/2) + A_{sf} f_y (d - 0.5h_f)] \qquad (8\text{-}19)$$

where,

$$A_{sf} = \frac{0.85 f'_c (b - b_w) h_f}{f_y} \qquad (8\text{-}20)$$

$$a = \frac{(A_s - A_{sf}) f_y}{0.85 f'_c b_w} \qquad (8\text{-}21)$$

8.16.3.3.3 The balanced reinforcement ratio, ρ_b, is given by:

$$\rho_b = \left(\frac{b_w}{b}\right)\left[\left(\frac{0.85\beta_1 f'_c}{f_y}\right)\left(\frac{87{,}000}{87{,}000 + f_y}\right) + \rho_f\right] \qquad (8\text{-}22)$$

where,

$$\rho_f = \frac{A_{sf}}{b_w d} \qquad (8\text{-}23)$$

8.16.3.3.4 For T-girder and box-girder construction, the width of the compression face, b, shall be equal to the effective slab width as defined in Article 8.10.

8.16.3.4 Rectangular Sections with Compression Reinforcement

8.16.3.4.1 The design moment strength, ϕM_n, may be computed as follows:

If $$\left(\frac{A_s - A_s'}{bd}\right) \geq 0.85\beta_1\left(\frac{f_c'd'}{f_yd}\right)\left(\frac{87{,}000}{87{,}000 - f_y}\right) \qquad (8\text{-}24)$$

then,

$$\phi M_n = \phi[(A_s - A_s')f_y(d - a/2) + A_s'f_y(d - d')] \qquad (8\text{-}25)$$

where,

$$a = \frac{(A_s - A_s')f_y}{0.85f_c'b} \qquad (8\text{-}26)$$

8.16.3.4.2 When the value of $(A_s - A_s')/bd$ is less than the value required by Equation (8-24), so that the stress in the compression reinforcement is less than the yield strength, f_y, or when effects of compression reinforcement is less than the yield strength, f_y, or when effects of compression reinforcement are neglected, the design moment strength may be computed by the equations in Article 8.16.3.2. Alternatively, a general analysis may be made based on stress and strain compatibility using the assumptions given in Article 8.16.2.

8.16.3.4.3 The balanced reinforcement ratio ρ_b for rectangular sections with compression reinforcement is given by:

$$\rho_b = \left[\frac{0.85\beta_1 f_c'}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right)\right] + \rho'\left(\frac{f_s'}{f_y}\right) \qquad (8\text{-}27)$$

where,

$$f_s' = 87{,}000\left[1 - \left(\frac{d'}{d}\right)\left(\frac{87{,}000 + f_y}{87{,}000}\right)\right] \leq f_y \qquad (8\text{-}28)$$

8.16.3.5 Other Cross Sections

For other cross sections the design moment strength, ϕM_n, shall be computed by a general analysis based on stress and strain compatibility using assumptions given in Article 8.16.2. The requirements of Article 8.16.3.1 shall also be satisfied.

8.16.4 Compression Members

8.16.4.1 General Requirements

8.16.4.1.1 The design of members subject to axial load or to combined flexure and axial load shall be based on stress and strain compatibility using the assumptions given in Article 8.16.2. Slenderness effects shall be included according to the requirements of Article 8.16.5.

8.16.4.1.2 Members subject to compressive axial load combined with bending shall be designed for the maximum moment that can accompany the axial load. The factored axial load, P_u, at a given eccentricity shall not exceed the design axial load strength $\phi P_{n(max)}$ where:

(a) For members with spiral reinforcement conforming to Article 8.18.2.2

$$P_{n(max)} = 0.85[0.85\, f_c'\, (A_g - A_{st}) + f_yA_{st}] \qquad (8\text{-}29)$$
$$\phi = 0.75$$

(b) For members with tie reinforcement conforming to Article 8.18.2.3

$$P_{n(max)} = 0.80[0.85\, f_c'\, (A_g - A_{st}) + f_yA_{st}] \qquad (8\text{-}30)$$
$$\phi = 0.70$$

The maximum factored moment, Mu, shall be magnified for slenderness effects in accordance with Article 8.16.5.

8.16.4.2 Compression Member Strengths

The following provisions may be used as a guide to define the range of the load-moment interaction relationship for members subjected to combined flexure and axial load.

8.16.4.2.1 Pure Compression

The design axial load strength at zero eccentricity, ϕP_o, may be computed by:

$$\phi P_o = \phi[0.85f_c'\,(A_g - A_{st}) + A_{st}f_y] \qquad (8\text{-}31)$$

For design, pure compressive strength is a hypothetical condition since Article 8.16.4.1.2 limits the axial load strength of compression members to 85 and 80% of the axial load at zero eccentricity.

8.16.4.2.2 Pure Flexure

The assumptions given in Article 8.16.2 or the applicable equations for flexure given in Article 8.16.3 may be used to compute the design moment strength, ϕM_n, in pure flexure.

8.16.4.2.3 Balanced Strain Conditions

Balanced strain conditions for a cross section are defined in Article 8.16.3.1.2. For a rectangular section with reinforcement in one face, or located in two faces at approximately the same distance from the axis of bending, the balanced load strength, ϕP_b, and balanced moment strength, ϕM_b, may be computed by:

$$\phi P_b = \phi[0.85 f'_c\, b a_b + A'_s f'_s - A_s f_y] \qquad (8\text{-}32)$$

and,

$$\phi M_b = \phi[0.85 f'_c b a_b (d - d'' - a_b/2) + A'_s f'_s (d - d' - d'') + A_s f_y d''] \qquad (8\text{-}33)$$

where,

$$a_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\beta_1 d \qquad (8\text{-}34)$$

and,

$$f'_s = 87{,}000\left[1 - \left(\frac{d'}{d}\right)\left(\frac{87{,}000 + f_y}{87{,}000}\right)\right] \le f_y \qquad (8\text{-}35)$$

8.16.4.2.4 Combined Flexure and Axial Load

The strength of a cross section is controlled by tension when the nominal axial load strength, P_n, is less than the balanced load strength, P_b, and is controlled by compression when P_n is greater than P_b.

The nominal values of axial load strength, P_n, and moment strength, M_n, must be multiplied by the strength reduction factor, ϕ, for axial compression as given in Article 8.16.1.2.

8.16.4.3 Biaxial Loading

In lieu of a general section analysis based on stress and strain compatibility, the design strength of noncircular members subjected to biaxial bending may be computed by the following approximate expressions:

$$\frac{1}{P_{nxy}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o} \qquad (8\text{-}36)$$

when the factored axial load,

$$P_u \ge 0.1 f'_c A_g \qquad (8\text{-}37)$$

or,

$$\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}} \le 1 \qquad (8\text{-}38)$$

when the factored axial load,

$$P_u < 0.1 f'_c A_g \qquad (8\text{-}39)$$

8.16.4.4 Hollow Rectangular Compression Members

8.16.4.4.1 The wall slenderness ratio of a hollow rectangular cross section, X_u/t, is defined in Figure 8.16.4.4.1. Wall slenderness ratios greater than 35.0 are not permitted, unless specific analytical and experimental evidence is provided justifying such values.

8.16.4.4.2 The equivalent rectangular stress block method shall not be employed in the design of hollow rectangular compression members with a wall slenderness ratio of 15 or greater.

8.16.4.4.3 If the wall slenderness ratio is less than 15, then the maximum usable strain at the extreme concrete compression fiber is equal to 0.003. If the wall slenderness ratio is 15 or greater, then the maximum usable strain at the extreme concrete compression fiber is equal to the computed local buckling strain of the widest flange of the cross section, or 0.003, whichever is less.

8.16.4.4.4 The local buckling strain of the widest flange of the cross section may be computed assuming simply supported boundary conditions on all four edges of the flange. Nonlinear material behavior shall be considered by incorporating the tangent material moduli of the concrete and reinforcing steel in computations of the local buckling strain.

8.16.4.4.5 In lieu of the provisions of Articles 8.16.4.4.2, 8.16.4.4.3 and 8.16.4.4.4, the following approximate method may be used to account for the strength reduction due to wall slenderness. The maximum usable strain at the extreme concrete compression fiber shall be taken as 0.003 for all wall slenderness ratios up to and including 35.0. A strength reduction factor ϕ_w shall be applied in addition to the usual strength reduction factor, ϕ, in Article 8.16.1.2. The strength reduction factor ϕ_w shall be taken as 1.0 for all wall slenderness ratios up to and including 15.0. For wall slenderness ratios greater than

8.16.6.6.3 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

(a) Shear strength V_n shall be computed by Equation (8-47), where shear strength V_c shall be in accordance with paragraph (d) and shear strength V_s shall be in accordance with paragraph (e).
(b) Shear strength shall be investigated at the critical section defined in Article 8.16.6.6.1(b), and at successive sections more distant from the support.
(c) Shear strength V_n shall not be taken greater than $6\sqrt{f_c'}b_od$, where b_o is the perimeter of the critical section defined in paragraph (b).
(d) Shear strength V_c at any section shall not be taken greater than $2\sqrt{f_c'}b_od$, where b_o is the perimeter of the critical section defined in paragraph (b).
(e) Where the factored shear force V_u exceeds the shear strength ϕV_c as given in paragraph (d), the required area A_v and shear strength V_s of shear reinforcement shall be calculated in accordance with Article 8.16.6.3.

8.16.6.7 Special Provisions for Slabs of Box Culverts

8.16.6.7.1 For slabs of box culverts under 2 feet or more fill, shear strength V_c may be computed by:

$$V_c = \left(2.14\sqrt{f_c'} + 4{,}600\,\rho\frac{V_u d}{M_u}\right)bd \qquad (8\text{-}59)$$

but V_c shall not exceed $4\sqrt{f_c'}\,bd$. For single cell box culverts only, V_c for slabs monolithic with walls need not be taken less than $3\sqrt{f_c'}\,bd$, and V_c for slabs simply supported need not be taken less than $2.5\sqrt{f_c'}\,bd$. The quantity V_ud/M_u shall not be taken greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

8.16.6.8 Special Provisions for Brackets and Corbels*

8.16.6.8.1 Provisions of Article 8.16.6.8 shall apply to brackets and corbels with a shear span-to-depth ratio a_v/d not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u. Distance d shall be measured at the face of support.

*These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-83" contains an example design of beam ledges—Part 16, example 16-3.

8.16.6.8.2 Depth at the outside edge of bearing area shall not be less than 0.5d.

8.16.6.8.3 The section at the face of the support shall be designed to resist simultaneously a shear V_u, a moment $(V_ua_v + N_{uc}(h - d))$, and a horizontal tensile force N_{uc}. Distance h shall be measured at the face of support.

(a) In all design calculations in accordance with Article 8.16.6.8, the strength reduction factor ϕ shall be taken equal to 0.85.
(b) Design of shear-friction reinforcement A_{vf} to resist shear V_u shall be in accordance with Article 8.16.6.4. For normal weight concrete, shear strength V_n shall not be taken greater than $0.2f_c'b_wd$ nor $800b_wd$ in pounds. For "all lightweight" or "sand-lightweight" concrete, shear strength V_n shall not be taken greater than $(0.2 - 0.07a_v/d)f_c'b_wd$ nor $(800 - 280a_v/d)b_wd$ in pounds.
(c) Reinforcement A_f to resist moment $(V_ua_v + N_{uc}(h - d))$ shall be computed in accordance with Articles 8.16.2 and 8.16.3.
(d) Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \leq \phi A_nf_y$. Tensile force N_{uc} shall not be taken less than $0.2V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc} shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.
(e) Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or:

$$\frac{2A_{vf}}{3} + A_n.$$

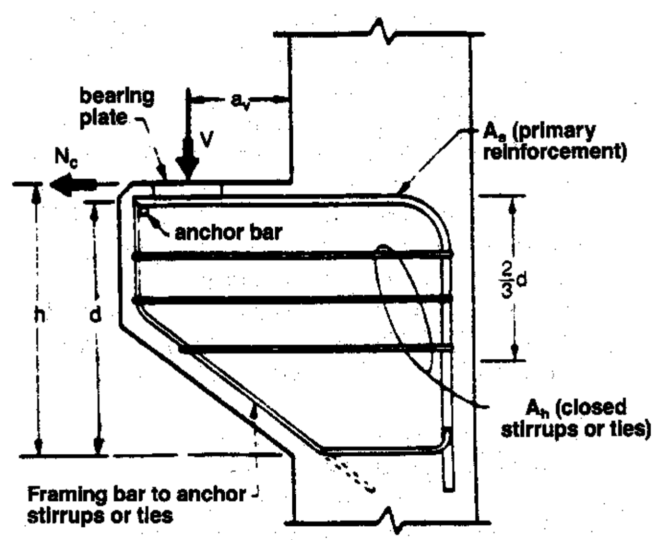


FIGURE 8.16.6.8

Section 16

SOIL-REINFORCED CONCRETE STRUCTURE INTERACTION SYSTEMS

16.1 GENERAL

16.1.1 Scope

Specifications in this Section govern the design of buried reinforced concrete structures. A buried reinforced concrete element becomes part of a composite system comprising the reinforced concrete section and the soil envelope, both of which contribute to the structural behavior of the system.

16.1.2 Notations

A = effective tension area of concrete surrounding the flexural tension reinforcement and hav-ing the same centroid as that reinforcement, divided by the number of bars or wires, sq in.; when the flexural reinforcement consists of several bar sizes or wire the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used (Articles 16.6.4 and 16.7.4)

A_p = total active lateral pressure acting on pipe, lbs/ft (Article 16.4.5 and Figure 16.4C)

A_s = tension reinforcement area on width b, in.^{2}/ft (Articles 16.4.6.6, 16.6.4.7, 16.7.4.7, and 16.8.5.7)

A_{si} = area of total inner cage reinforcement required in length b, in^2/ft (Article 16.4.6.6)

A_{so} = area of total outer cage reinforcement required in length b, in^2/ft (Article 16.4.6.6)

A_{vr} = stirrup reinforcement area to resist radial tension forces on width b, in.^{2}/ft in each line of stirrups at circumferential spacing s (Article 16.4.6)

A_{vs} = required area of stirrups for shear reinforcement, in.^{2} (Article 16.4.6.6.6.2)

A_{wr} = steel area required for an individual circumferential wire for flexure at a splice or at the point of maximum moment for quadrant mat reinforcement, in^2 (Article 16.4.7)

b = width of section which resists M, N, V—Usually b = 12 inches (Article 16.4.6)

B_c = out-to-out horizontal span of pipe or box, ft (Articles 16.4.4, 16.4.5, 16.6.4, and 16.7.4.)

B_d = horizontal width of trench at top of pipe or box, ft (Articles 16.4.4, 16.6.4, and 16.7.4.)

B_f = bedding factor (Article 16.4.5)

B_{fe} = earth load bedding factor

B_{fLL} = live load bedding factor

B_1 = crack control coefficient for effect of cover and spacing of reinforcement (Article 16.4.6)

B_c' = out-to-out vertical rise of pipe, ft (Figure 16.4C)

C_c = load coefficient for embankment installations (Article 16.4.5)

C_d = load coefficient for trench installations (Article 16.4.4)

C_A = constant corresponding to the shape of pipe (Article 16.4.5)

C_N = parameter which is a function of the distribution of the vertical load and the vertical reaction (Article 16.4.5)

C_1 = crack control coefficient for type of reinforcement (Article 16.4.6)

d = distance from compression face to centroid of tension reinforcement, in. (Articles 16.4.6.6, 16.6.4.7, 16.7.4.7, and 16.8.5.7)

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto (Articles 16.6.4.7, 16.7.4.7, and 16.8.5.7)

D = D-load of pipe, three-edge bearing test load expressed in pounds per linear foot per foot of span to produce a 0.01-inch crack (Article 16.4.5)

D_i = inside diameter of pipe, in.

f_s = service load stress in reinforcing steel for crack control (Articles 16.6.4.7, 16.7.4.7, and 16.8.5.7)

f_v = maximum allowable strength of stirrup material, lbs/in.^{2} (Article 16.4.6.6.6)

f_y = specified yield strength of reinforcement, lbs/in.^{2} (Article 16.4.6)

F_c = factor for effect of curvature on diagonal tension (shear) strength in curved components (Article 16.4.6.6.5)
F_{cr} = factor for adjusting crack control relative to average maximum crack width of 0.01 inch when F_{cr} = 1.0 (Article 16.4.6.6.4)
F_d = factor for crack depth effect resulting in increase in diagonal tension (shear) strength with decreasing d (Article 16.4.6.6.5)
F_e = soil-structure interaction factor (Articles 16.4.4, 16.6.4, and 16.7.4)
F_{e1} = soil structure interaction factor for embankment installations (Articles 16.4.4, 16.6.4, and 16.7.4)
F_{e2} = soil-structure interaction factor for trench installations (Articles 16.4.4, 16.6.4, and 16.7.4)
F_{rp} = factor for process and local materials that affect the radial tension strength of pipe (Article 16.4.6)
F_{rt} = factor for pipe size effect on radial tension strength (Article 16.4.6.6.3.1)
F_{vp} = factor for process and local materials that affect the shear strength of pipe (Article 16.4.6.6.5)
F_N = coefficient for effect of thrust on shear strength (Article 16.4.6.6.5)
f'_c = design compressive strength of concrete, lbs/in.^{2} (Articles 16.4.6, 16.6.2, and 16.7.2)
h = overall thickness of member (wall thickness), in. (Articles 16.4.6.6, 16.6.4.7, 16.7.4.7, and 16.8.5.7)
H = height of fill above top of pipe or box, ft (Articles 16.4.4, 16.4.5, 16.6.4, and 16.7.4)
HAF = horizontal arching factor (Figure 16.4A)
i = coefficient for effect of axial force at service load stress, f_s (Articles 16.4.6.6.4, 16.6.4.7, 16.7.4.7, and 16.8.5.7)
j = coefficient for moment arm at service load stress, f_s (Articles 16.4.6.6.4, 16.6.4.7, 16.7.4.7, and 16.8.5.7)
K = ratio of the active unit lateral soil pressure to unit vertical soil pressure-Rankine's coefficient of active earth pressure (Figures 16.4B-D)
L_d = development length of reinforcing wire or bar, in (Article 16.4.7)
M_{nu} = factored moment acting on length b as modified for effects of compressive or tensile thrust, in-lbs/ft (Article 16.4.6.6.5)
M_s = moment acting on cross section of width, b, service load conditions, in-lbs/ft (Taken as absolute value in design equations, always +) (Articles 16.4.6.6.4, 16.6.4.7, 16.7.4.7, and 16.8.5.7)
M_u = factored moment acting on cross section of width b, in.-lbs/ft (Article 16.4.6.6.6.1)
n = number of layers of reinforcement in a cage—1 or 2 (Article 16.4.6.6.4)
N_s = axial thrust acting on cross section of width b, service load condition (+ when compressive, − when tensile), lbs/ft (Articles 16.4.6.6.4, 16.6.4.7, 16.7.4.7, and 16.8.5.7)
N_u = factored axial thrust acting on cross section of width b, lbs/ft (Article 16.4.6)
p = projection ratio (Article 16.4.5.2.1)
p′ = negative projection ratio (Figure 16.4A and Table 16.4A)
PL = PL denotes the prism load (weight of the column of earth) over the pipe's outside diameter, lbs/ft (Figure 16.4.A)
q = ratio of the total lateral pressure to the total vertical load (Article 16.4.5)
r_s = radius of the inside reinforcement, in. (Article 16.4.6.6.3.1)
r_{sd} = settlement ratio (Article 16.4.5.2.1)
s = spacing of reinforcement wire or bar, in. (Article 16.4.6.6.4)
s_v = circumferential spacing of stirrups, in. (Article 16.4.6.6.6)
s_ℓ = spacing of circumferential reinforcement, in. (Article 16.4.6.6.4)
S_i = internal horizontal span of pipe, in. (Articles 16.4.6.6 and 16.4.5.1)
t_b = clear cover over reinforcement, in. (Article 16.4.6.6.4)
V_b = basic shear strength of critical section, lbs/ft where $M_{nu}/(V_u d) = 3.0$ (Article 16.4.6.6.5)
V_c = nominal shear strength provided by width b of concrete cross section, lbs/ft (Article 16.4.6.6.6)
V_u = factored shear force acting on cross section of width b, lbs/ft (Article 16.4.6.6.5)
V_{uc} = factored shear force at critical section, lbs/ft where $M_{nu}/(V_u d) = 3.0$ (Article 16.4.6.6.5)
VAF = vertical arching factor (Article 16.4.4.2.1.1)
w = unit weight of soil, lbs/ft^3 (Article 16.4.4)
W_E = total earth load on pipe or box, lbs/ft (Articles 16.4.4, 16.4.5, 16.6.4, and 16.7.4)
W_f = fluid load in the pipe as determined according to Article 16.4.4.2.2, lbs/ft
W_L = total live load on pipe or box, lbs/ft (Articles 16.4.4 and 16.4.5)
W_T = total load, earth and live, on pipe or box, lbs/ft (Articles 16.4.4 and 16.4.5)
x = parameter which is a function of the area of the vertical projection of the pipe over which lateral pressure is effective (Article 16.4.5)
μ = coefficient of internal friction of the soil (Figure 16.4B)
μ' = coefficient of friction between backfill and trench walls (Figure 16.4B)
ψ = central angle of pipe subtended by assumed distribution of external reactive force (Figure 16.4F)

ρ = ratio of reinforcement area to concrete area (Article 16.4.6)
ϕ_f = strength reduction factor for flexure (Article 16.4.6.6.1)
ϕ_r = strength reduction factor for radial tension (Article 16.4.6.6.3.1)
ϕ_v = strength reduction factors for shear (Article 16.4.6.6.5)

16.1.3 Loads

Design loads shall be determined by the forces acting on the structure. For earth loads, see Article 3.20. For live loads see Articles 3.4 through 3.8 and Articles 3.11 and 3.12. For loading combinations see Article 3.22.

16.1.4 Design

Design may be based on working stress or ultimate strength principles. The design criteria shall include structural aspects (e.g. flexure, thrust, shear), handling and installation, and crack control. Footing design for cast-in-place boxes and arches shall be in conformity with Article 4.4.

16.1.5 Materials

The materials shall conform to the AASHTO materials specifications referenced herein.

16.1.6 Soil

Structural performance is dependent on soil structure interaction. The type and anticipated behavior of the material beneath the structure, adjacent to the structure, and over the structure must be considered.

16.1.7 Abrasive or Corrosive Conditions

Where abrasive or corrosive conditions exist, suitable protective measures shall be considered.

16.1.8 End Structures

Structures may require special consideration where erosion may occur. Skewed alignment may require special end wall designs.

16.1.9 Construction and Installation

The construction and installation shall conform to Section 27, Division II.

16.2 SERVICE LOAD DESIGN

16.2.1 For soil-reinforced concrete structure interaction systems designed with reference to service loads and allowable stresses, the service load stresses shall not exceed the values shown in Article 8.15 except as modified herein.

16.2.2 For precast reinforced concrete circular pipe, elliptical pipe, and arch pipe, the results of three edge-bearing tests made in accordance with AASHTO materials specifications may be used in lieu of service load design.

16.3 LOAD FACTOR DESIGN

16.3.1 Soil-reinforced concrete structure interaction systems shall be designed to have design strengths of all sections at least equal to the required strengths calculated for the factored loads as stipulated in Article 3.22, except as modified herein.

16.3.2 For precast reinforced concrete circular pipe, elliptical pipe, and arch pipe, the results of three edge-bearing tests made in accordance with AASHTO materials specifications may be used in lieu of load factor design.

16.4 REINFORCED CONCRETE PIPE

16.4.1 Application

This Specification is intended for use in design for precast reinforced concrete circular pipe, elliptical pipe, and arch pipe. Standard dimensions are shown in AASHTO material specifications M 170, M 206, M 207, and M 242. Design wall thicknesses other than the standard wall dimensions may be used, provided the design complies with all applicable requirements of Section 16.

16.4.2 Materials

16.4.2.1 Concrete

Concrete shall conform to Article 8.2 except that evaluation of f'_c may be based on cores.

16.4.2.2 Reinforcement

Reinforcement shall meet the requirements of Articles 8.3.1 through 8.3.3 only, and shall conform to one of the following AASHTO material specifications M 31, M 32, M 55, M 221, or M 255. For smooth wire and smooth

TABLE 16.4A Standard Embankment Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	B_c/24″ (600 mm) minimum, not less than 3″ (75 mm). If rock foundation, use B_c/12″ (300 mm) minimum, not less than 6″ (150 mm).	95% SW	90% SW, 95% ML, or 100% CL
Type 2 (See Note 3.)	B_c/24″ (600 mm) minimum, not less than 3″ (75 mm). If rock foundation, use B_c/12″ (300 mm) minimum, not less than 6″ (150 mm).	90% SW or 95% ML	85% SW, 90% ML, or 95% CL
Type 3 (See Note 3.)	B_c/24″ (600 mm) minimum, not less than 3″ (75 mm). If rock foundation, use B_c/12″ (300 mm) minimum, not less than 6″ (150 mm).	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL
Type 4	No bedding required, except if rock foundation, use B_c/12″ (300 mm) minimum, not less than 6″ (150 mm).	No compaction required, except if CL, use 85% CL	No compaction required, except if CL, use 85% CL

NOTES:

1. Compaction and soil symbols -i.e. "95% SW" refer to SW soil material with a minimum standard proctor compaction of 95%. See Table 16.4C for equivalent modified proctor values.
2. Soil in the outer bedding, haunch, and lower side zones, except within B_c/3 from the pipe springline, shall be compacted to at least the same compaction as the majority of soil in the overfill zone.
3. Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical and arch pipe.
4. *SUBTRENCHES*

4.1 A subtrench is defined as a trench with its top below finished grade by more than 0.1H or, for roadways, its top is at an elevation lower than 1′ (0.3 m) below the bottom of the pavement base material.

4.2 The minimum width of a subtrench shall be 1.33 B_c, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.

4.3 For subtrenches with walls of natural soil, any portion of the lower side zone in the subtrench wall shall be at least as firm as an equivalent soil placed to the compaction requirements specified for the lower side zone and as firm as the majority of soil in the overfill zone, or shall be removed and replaced with soil compacted to the specified level.

welded wire fabric, a yield stress of 65,000 psi and for deformed welded wire fabric, a yield stress of 70,000 psi may be used.

16.4.2.3 Concrete Cover for Reinforcement

The minimum concrete cover for the reinforcement in precast concrete pipe shall be 1 inch in pipe having a wall thickness of 2½ inches or greater and ¾ inch in pipe having a wall thickness of less than 2½ inches.

16.4.3 Installations

16.4.3.1 Standard Installations

Standard Embankment Installations are presented in Figure 16.4B and Standard Trench Installations are presented in Figure 16.4C; these figures define soil areas and critical dimensions. Generic soil types, minimum compaction requirements, and minimum bedding thicknesses are listed in Table 16.4A for four Standard Embankment Installation Types and in Table 16.4B for four Standard Trench Installation Types.

16.4.3.2 Soils

The AASHTO Soil Classifications and the USCS Soil Classifications equivalent to the generic soil types in the Standard Installations are presented in Table 16.4C.

16.4.4 Design

16.4.4.1 General Requirements

Design shall conform to applicable sections of these specifications except as provided otherwise in this article. For design loads, see Article 16.1.3; for standard installation, see Article 16.4.3.1; and for bedding conditions, see Section 27, Division II—Construction and the Soil-Structure Interaction Modifications that follow. Live loads, W_L, shall be included as part of the total load, W_T, and shall be distributed through the earth cover as specified in Article 6.4, except that the 2-foot minimum in the first paragraph of Article 6.4 does not apply. Other methods for determining total load and pressure distribution may be used, if they are based on successful design

TABLE 16.4B Standard Trench Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	$B_c/24''$ (600 mm) minimum, not less than 3″ (75 mm). If rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6″ (150 mm).	95% SW	90% SW, 95% ML, 100% CL, or natural soils of equal firmness
Type 2 (See Note 3.)	$B_c/24''$ (600 mm) minimum, not less than 3″ (75 mm). If rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6″ (150 mm).	90% SW or 95% ML	85% SW, 90% ML, 95% CL, or natural soils of equal firmness
Type 3 (See Note 3.)	$B_c/24''$ (600 mm) minimum, not less than 3″ (75 mm). If rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6″ (150 mm).	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, 95% CL, or natural soils of equal firmness
Type 4	No bedding required, except if rock foundation, use $B_c/12''$ (300 mm) minimum, not less than 6″ (150 mm).	No compaction required, except if CL, use 85% CL	85% SW, 90% ML 95% CL, or natural soils of equal firmness

NOTES:

1. Compaction and soil symbols -i.e. "95% SW"-refers to SW soil material with minimum standard Proctor compaction of 95%. See Table 16.4C for equivalent modified Proctor values.
2. The trench top elevation shall be no lower than 0.1H below finished grade or, for roadways, its top shall be no lower than an elevation of 1′ (0.3 m) below the bottom of the pavement base material.
3. Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical and arch pipe.
4. Soil in bedding and haunch zones shall be compacted to at least the same compaction as specified for the majority of soil in the backfill zone.
5. The trench width shall be wider than shown if required for adequate space to attain the specified compaction in the haunch and bedding zones.
6. For trench walls that are within 10 degrees of vertical, the compaction or firmness of the soil in the trench walls and lower side zone need not be considered.
7. For trench walls with greater than 10-degree slopes that consist of embankment, the lower side shall be compacted to at least the same compaction as specified for the soil in the backfill zone.

practice or tests that reflect the appropriate design conditions.

16.4.4.2 Loads

16.4.4.2.1 Earth Loads and Pressure Distribution

The effects of soil-structure interaction shall be taken into account and shall be based on the design earth cover, sidefill compaction, and bedding characteristics of the pipe-soil installations.

16.4.4.2.1.1 Standard Installations

For the Standard Installations given in Article 16.4.3.1, the earth load, W_E, may be determined by multiplying the prism load (weight of the column of earth) over the pipes outside diameter by the soil-structure interaction factor, F_e, for the specified installation type.

$$W_E = F_e w B_c H \qquad (16\text{-}1)$$

Standard Installations for both embankments and trenches shall be designed for positive projection, embankment loading conditions where F_e = VAF given, in Figure 16.4A for each Type of Standard Installation.

For Standard Installations the earth pressure distribution shall be the Heger pressure distribution shown in Figure 16.4A for each type of Standard Installation.

The unit weight of soil used to calculate earth load shall be the estimated unit weight for the soils specified for the pipe-soil installation and shall not be less than 110 lbs/cu ft.

16.4.4.2.1.2 Nonstandard Installations

When nonstandard installations are used, the earth load and pressure distribution shall be determined by an appropriate soil-structure interaction analysis.

16.4.4.2.2 Pipe Fluid Weight

The weight of fluid, W_f, in the pipe shall be considered in design based on a fluid weight of 62.4 lbs/ft^3, unless otherwise specified. For Standard Installations, the fluid weight shall be supported by vertical earth pressure that is assumed to have the same distribution over the lower part of the pipe as given in Figure 16.4A for earth load.

TABLE 16.4C Equivalent USCS and AASHTO Soil Classifications For SIDD Soil Designations

	Representative Soil Types		Percent Compaction	
SIDD Soil	USCS	AASHTO	Standard Proctor	Modified Proctor
Gravelly Sand (SW)	SW, SP GW, GP	A1, A3	100	95
			95	90
			90	85
			85	80
			80	75
			61	59
Sandy Silt (ML)	GM, SM, ML Also GC, SC with less than 20% passing No. 200 sieve	A2, A4	100	95
			95	90
			90	85
			85	80
			80	75
			49	46
Silty Clay (CL)	GL, MH, GC, SC	A5, A6	100	90
			95	85
			90	80
			85	75
			80	70
			45	40
	CH	A7	100	90
			95	85
			90	80
			45	40

16.4.4.2.3 Live Loads

Live loads shall be either the AASHTO HS-Series or the AASHTO Interstate Design truck loads. Live loads shall be distributed through the earth cover as specified in Article 6.4, except that the 2-foot minimum in the first paragraph of Article 6.4 does not apply. For Standard Installations the live load on the pipe shall be assumed to have a uniform vertical distribution across the top of the pipe and the same distribution across the bottom of the pipe as given in Figure 16.4A for earth load.

16.4.4.3 Minimum Fill

For unpaved areas and under flexible pavements, the minimum fill over precast reinforced concrete pipe shall be 1 foot or ⅛ of the diameter or rise, whichever is greater. Under rigid pavements, the distance between the top of the pipe and the bottom of the pavement slab shall be a minimum of 9 inches of compacted granular fill.

16.4.4.4 Design Methods

The structural design requirements of installed precast reinforced concrete pipe may be determined by either the Indirect or Direct Method.

16.4.5 Indirect Design Method Based on Pipe Strength and Load-Carrying Capacity

16.4.5.1 Loads

The design load-carrying capacity of a reinforced concrete pipe must equal the design load determined for the pipe as installed, or

$$D = \left[\frac{12}{S_i}\right]\left[\frac{W_E + W_F}{B_{fe}} + \frac{W_L}{B_{fLL}}\right] \qquad (16\text{-}2)$$

where

- D = D-load of the pipe (three edge-bearing test load expressed in pounds per linear foot per foot of diameter) to produce a 0.01-inch crack. For Type 1 installations, D-load as calculated above shall be modified by multiplying by an installation factor of 1.10;
- S_i = internal diameter or horizontal span of the pipe in inches;
- B_f = bedding factor, see Article 16.4.5.2;
- B_{Fe} = earth load bedding factor;
- B_{FLL} = live load bedding factor;

16.5.3.2 Minimum Cover

The minimum fill over reinforced concrete arches shall be 12 inches or Span/8.

16.5.3.3 Strength-Reduction Factors

Strength-reduction factors for load factor design of cast-in-place arches may be taken as 0.90 for flexure and 0.85 for shear.

16.5.3.4 Splices of Reinforcement

Reinforcement shall be in conformity with Article 8.32.1.1. If lap splicing is used, laps shall be staggered with a minimum of 1 foot measured along the circumference of the arch. Ties shall be provided connecting the intrados and extrados reinforcement. Ties shall be at 12-inch maximum spacing, in both longitudinal and circumferential directions, except as modified by shear.

16.5.3.5 Footing Design

Design shall include consideration of differential horizontal and vertical movements and footing rotations. Footing design shall conform to Article 4.4.

16.6 REINFORCED CONCRETE BOX, CAST-IN-PLACE

16.6.1 Application

This specification is intended for use in the design of cast-in-place reinforced concrete box culverts.

16.6.2 Materials

16.6.2.1 Concrete

Concrete shall conform to Article 8.2 except that evaluation of f'_c may be based on test beams.

16.6.2.2 Reinforcement

Reinforcement shall meet the requirements of Article 8.3 except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8 inches.

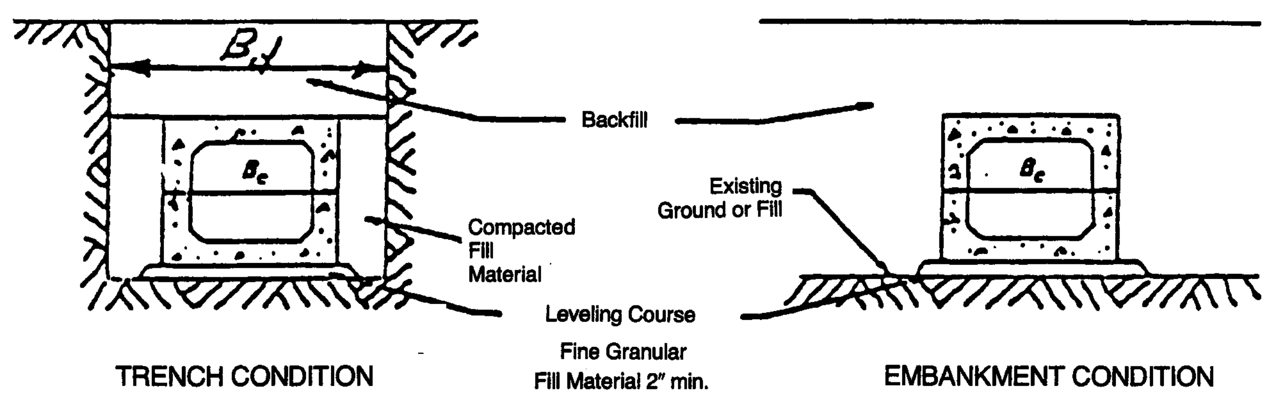
16.6.3 Concrete Cover for Reinforcement

The minimum concrete cover for reinforcement shall conform to Article 8.22. The top slab shall be considered a bridge slab for concrete cover considerations.

16.6.4 Design

16.6.4.1 General Requirements

Designs shall conform to applicable sections of these specifications except as provided otherwise in this article. For design loads and loading conditions see Section 3. For distribution of concentrated loads through earth for culverts with less than 2 feet of cover, see Article 3.24.3, Case B, and for requirements for bottom distribution reinforcement in top slabs of such culverts see Article 3.24.10. For distribution of wheel loads to culverts with 2 feet or more of cover see Article 6.4. For reinforced concrete design requirements, see Section 8.



CONCRETE BOX SECTIONS

FIGURE 16.6A

16.6.4.2 Modification of Earth Loads for Soil Structure Interaction

The effects of soil structure interaction shall be taken into account and shall be based on the design earth cover, sidefill compaction, and bedding characteristics. These parameters may be determined by a soil-structure interaction analysis of the system. The loads given in Article 6.2 may be used, if they are multiplied by a soil-structure interaction factor, F_e, that accounts for the type and conditions of installation as defined in Figure 16.6A, so that the total earth load, W_E on the box section is

$$W_E = F_e w B_c H \qquad (16\text{-}16)$$

F_e may be determined by the Marston-Spangler Theory of earth loads, as follows

16.6.4.2.1 Embankment Installations

$$F_{e1} = 1 + 0.20 \frac{H}{B_c} \qquad (16\text{-}17)$$

F_{e1} need not be greater than 1.15 for installations with compacted fill at the sides of the box section, and need not be greater than 1.4 for installations with uncompacted fill at the sides of the box section.

16.6.4.2.2 Trench Installations

$$F_{e2} = \frac{C_d B_d^2}{H B_c} \qquad (16\text{-}18)$$

Values of C_d can be obtained from Figure 16.4B for normally encountered soils. The maximum value of F_{e2} need not exceed F_{e1}.

The soil-structure interaction factor, F_e, is not applicable if the Service Load Design Method is used.

16.6.4.3 Distribution of Concentrated Load Effects to Bottom Slab

The width of top slab strip used for distribution of concentrated wheel loads may be increased by twice the box height and used for the distribution of loads to the bottom slab.

16.6.4.4 Distribution of Concentrated Loads in Skewed Culverts

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

16.6.4.5 Span Length

For span length, see Article 8.8, except when monolithic haunches included at 45° are considered in the design, negative moment reinforcement in walls and slabs may be proportioned based on the bending moment at the intersection of haunch and the uniform depth member.

16.6.4.6 Strength-Reduction Factors

Strength-reduction factors for load factor design may be taken at 0.9 for combined flexure and thrust and as 0.85 for shear.

16.6.4.7 Crack Control

The maximum service load stress in the reinforcing steel for crack control shall be

$$f_s = \frac{155}{\beta \sqrt[3]{d_c A}} \leq 0.6\ fy\ \text{ksi} \qquad (16\text{-}19)$$

$$\beta = \left[1 + \frac{d_c}{0.7d}\right]$$

β = approximate ratio of distance from neutral axis to location of crack width at the concrete surface divided by distance from neutral axis to centroid of tensile reinforcing

d_c = distance measured from extreme tension fiber to center of the closest bar or wire in inches. For calculation purposes, the thickness of clear concrete cover used to compute d_c shall not be taken greater than 2 inches.

The service load stress should be computed considering the effects of both bending moment and thrust using:

$$f_s = \frac{M_s + N_s(d - h/2)}{(A_s jid)} \qquad (16\text{-}20)$$

where

f_s = stress in reinforcement under service load conditions, psi

e = $M_s/N_s + d - h/2$

e/d min. = 1.15

i = $1/(1-(jd/e)$

j = $0.74 + 0.1(e/d) \leq 0.9$

16.6.4.8 Minimum Reinforcement

Minimum reinforcement shall be provided in accordance with Article 8.17.1 at all cross sections subject to flexural tension, including the inside face of walls. Shrinkage and temperature reinforcement shall be provided near the inside surfaces of walls and slabs in accordance with Article 8.20.

16.7 REINFORCED CONCRETE BOX, PRECAST

16.7.1 Application

This specification is intended for use in design for precast reinforced concrete box sections. Boxes may be manufactured using conventional structural concrete and forms (formed) or with dry concrete and vibrating form pipe-making methods (machine made). Standard dimensions are shown in AASHTO materials specifications M 259 and M 273.

16.7.2 Materials

16.7.2.1 Concrete

Concrete shall conform to Article 8.2 except that evaluation of f'_c may be based on cores.

16.7.2.2 Reinforcement

Reinforcement shall meet the requirements of Article 8.3 except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8 inches.

16.7.3 Concrete Cover for Reinforcement

The minimum concrete cover for reinforcement in boxes reinforced with wire fabric shall be three times the wire diameter but not less than 1 inch. For boxes covered by less than 2 feet of fill, the minimum cover for reinforcement in the top of the slab shall be 2 inches.

16.7.4 Design

16.7.4.1 General Requirements

Design shall conform to applicable sections of these specifications except as provided otherwise in this article. For design loads and loading conditions see Section 3. For distribution of wheel loads to culvert slabs under less than 2 feet of cover see Article 3.24.3, Case B, and for requirements for bottom reinforcement in top slabs of such culverts see Article 3.24.10. For distribution of wheel loads to culvert slabs with 2 feet or more of cover, see Article 6.4.

For reinforced concrete design requirements see Section 8. For span length see Article 8.8, except as noted in Article 16.7.4.6.

16.7.4.2 Modification of Earth Loads for Soil-Structure Interaction

The effects of soil-structure interaction shall be taken into account and shall be based on the design earth cover, sidefill compaction, and bedding characteristics. These parameters may be determined by a soil-structure interaction analysis of the system. The loads given in Article 6.2 may be used, if they are multiplied by a soil-structure interaction factor, F_e, that accounts for the type and conditions of installation as defined in Figure 16.6A, so that the total earth load, W_E, on the box section is:

$$W_E = F_e w B_c H \tag{16-21}$$

F_e may be determined by the Marston-Spangler Theory of earth loads as follows:

16.7.4.2.1 Embankment Installations:

$$F_{e1} = 1 + 0.20\frac{H}{B_c} \tag{16-22}$$

F_{e1} need not be greater than 1.15 for installations with compacted fill at the sides of the box section, and need not be greater than 1.4 for installations with uncompacted fill at the sides of the box section.

16.7.4.2.2 Trench Installations:

$$F_{e2} = \frac{C_d B_d^2}{HB_c} \tag{16-23}$$

Values of C_d can be obtained from Figure 16.4B for normally encountered soils. The maximum value of F_{e2} need not exceed F_{e1}.

The soil-structure interaction factor F_e, is not applicable if the Service Load Design Method is used.

16.7.4.3 Distribution of Concentrated Load Effects in Sides and Bottoms

The width of the top slab strip used for distribution of concentrated wheel loads shall also be used for determination of bending moments, shears, and thrusts in the sides and bottoms.

16.7.4.4 Distribution of Concentrated Loads in Skewed Culverts

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

16.7.4.5 Span Length

When monolithic haunches inclined at 45° are taken into account, negative reinforcement in walls and slabs may be proportioned based on the bending moment at the intersection of haunch and uniform depth member.

16.7.4.6 Strength-Reduction Factors

Strength-reduction factors for load factor design of machine-made boxes may be taken as 1.0 for moment and 0.9 for shear.

16.7.4.7 Crack Control

The maximum service load stress in the reinforcing steel for crack control shall be:

$$f_s = \frac{98}{\sqrt[3]{d_c A}} \text{ ksi} \qquad (16\text{-}24)$$

The service load stress should be computed considering the effects of both bending moment and thrust using:

$$f_s = \frac{M_s + N_s(d - h/2)}{(A_s jid)} \qquad (16\text{-}25)$$

where

f_s = stress in reinforcement under service load conditions, psi
e = $M_s/N_s + d–h/2$
e/d min. = 1.15
i = $1/(1–(jd/e)$
j = $0.74 + 0.1(e/d) \leq 0.9$

16.7.4.8 Minimum Reinforcement

The primary flexural reinforcement in the direction of the span shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.002. Such minimum reinforcement shall be provided at all cross sections subject to flexural tension, at the inside face of walls, and in each direction at the top of slabs of box sections with less than 2 feet of fill. The provisions of Article 8.20 do not apply to precast concrete box sections, except if units of unusual length (over 16 ft) are fabricated, the minimum longitudinal reinforcement for shrinkage and temperature should be as provided in Article 8.20.

16.8 PRECAST REINFORCED CONCRETE THREE-SIDED STRUCTURES

16.8.1 Application

This specification is intended for use in design for precast reinforced concrete three-sided structures supported on a concrete footing foundation. Units may be manufactured using conventional structural concrete and forms (formed) or machine made using low slump concrete and vibrating forms.

16.8.2 Materials

16.8.2.1 Concrete

Concrete shall conform to Article 8.2 except that evaluation of f'_c may also be based on cores.

16.8.2.2 Reinforcement

Reinforcement shall meet the requirements of Article 8.3 except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8 inches. Circumferential welded wire fabric spacing shall not exceed a 4-inch maximum and 2-inch minimum. Prestressing if used, shall be in accordance with Section 9.

16.8.3 Concrete Cover for Reinforcement

The minimum concrete cover for reinforcement in precast three-sided structures reinforced with welded wire fabric shall be three times the wire diameter but not less than 1 inch. For precast three-sided structures covered by less than 2 feet of fill, the minimum cover for the reinforcement in the top of the top slab shall be 2 inches.

16.8.4 Geometric Properties

The shape of the precast three-sided structures may vary in span, rise, wall thickness, haunch dimensions and curvature. Specific geometric properties shall be specified by the manufacturer. Wall thicknesses, however, shall be a minimum of 8 inches for spans under 24 feet and 10 inches for 24-foot spans and larger.

Bridge Inspection Manual

Texas
Department
of Transportation

Revised July 2002

Section 4
Load Ratings

Definition of Load Ratings

The Load Rating is a measure of bridge live load capacity and has two commonly used categories:

- Inventory Rating, as defined by the current AASHTO Manual for Condition Evaluation of Bridges,[4] is that load, including loads in multiple lanes, that can safely utilize the bridge for an indefinite period of time.
- Operating Rating, defined by the same manual, is the maximum permissible live load that can be placed on the bridge. This load rating also includes the same load in multiple lanes. Allowing unlimited usage at the Operating Rating level will reduce the life of the bridge.

Determination of Load Ratings

Currently, all Inventory and Operating Ratings are expressed in terms of an equivalent HS-truck as shown in the *Manual for Condition Evaluation of Bridges.*[5] Prior to about 1995, many ratings were for an equivalent H-truck, shown in *Manual for Condition Evaluation of Bridges.*[6] The H-truck directly corresponds to single-unit trucks, which used to be common on rural highways. Today, even rural Farm- or Ranch-to-Market highways and many off-system highways are exposed to much larger semi-trucks; therefore, the HS-truck is more realistic.

Inventory or Operating Ratings are usually determined using either Load Factor (LF) or Allowable Stress (AS) methods. Since 2000, LF is to be used for all on-system bridges. Either AS or LF may be used for all off-system bridges. Timber bridges on both systems are rated using only AS methods because as of 2001, there are no well-documented LF timber rating procedures. Timber is a structural material for which it is difficult to assign an ultimate strength.

Inventory Rating and Design Load Considerations

The Inventory Rating (Item 66) can usually be initially estimated to be at least equal to the design loading if no damage or deterioration exists and the original design was made using an HS load pattern. Many old plans have a design loading shown as H-20 S-16 which some raters have misinterpreted as meaning H-20. AASHTO replaced the H-20 S-16 designation in 1965 with the HS-20 designation. Re-rating these bridges using LF procedures will usually increase the Inventory Rating above HS-20. Some newer bridges have been designed on a case-by-case basis for higher design loads, but as of 2001, TxDOT bridge design practice is still to design to the HS-20 loading.

Rating bridges designed between 1946 and about 1958 by current LF procedures may result in significantly different values than the original design loading. Even though the plans may say designed to H-20 S-16 and THD Supplement No. 1, the bridge may rate significantly less than HS-20 loading. This difference is due to the more liberal effects of THD Design Supplement No. 1 described below.

THD Design Supplement No. 1

In 1946, the Bridge Division of TxDOT (then called THD) issued what is commonly called THD Supplement No. 1.[7] Texas at that time was influential in the development of the AASHTO Bridge Design Specifications. However, not all the Texas opinions were immediately accepted by the AASHTO Bridge Committee, which includes all states. As a result, TxDOT used the supplement for a number of years to amend portions of the 1944 and 1949 AASHTO Standard Specifications for Highway Bridges[8,9] for use in Texas.

The first version of Supplement No. 1 was dated June 1946.[10] The second version of Supplement No. 1 was dated September 1953[11] and included only those items of the 1946 version that had not been incorporated into the 1949 AASHTO Standard Specifications for Highway Bridges.[12] The primary subjects of the supplement that affected bridge design can be summarized as follows:

- ***Crown Width Bridges.*** The 1944 AASHTO Bridge Specifications[13] required curbs on all bridges. Texas initiated the concept of "crown width" bridges with the following: "On non-restrictive bridges the curbs may be omitted provided the guard fence or an equivalent member is carried continuously through the structure." The 1949 AASHTO Bridge Specifications[14] allowed the condition of no curbs with certain additional width limitations. Texas continued the crown-width, no-curb concept with the retention of the provision in the second version of Supplement No. 1 dated September 1953.[15]
- ***Design Overload.*** The 1944 AASHTO Bridge Specifications[16] required an overload to be considered for all bridges designed for less than an H-20 (40,000 lbs) or H-20 S-16 (72,000 lbs) loading, now called HS-20 loading. The overload was to be the design truck (usually H-15) increased by 100 percent, but without concurrent loading of adjacent lanes, thus allowing single-lane load distribution. The allowable stress was to also be increased to 150 percent of the basic allowable. Texas modified this provision specifically to apply the same overload to truss counter members for all design loadings. Truss counters are those members that, for some positions of live load, will change from tension to compression. If a truss was designed H-15, H-20, or H-20 S-16, the overload was applied in determining the size of counter member.
- ***Lane Load Negative Moments.*** The 1944 AASHTO Bridge Specifications[17] required for H-10, H-15, or H-20 lane loads an additional concentrated load in one other span in a continuous unit positioned to produce maximum positive and negative moments. Texas limited the distance between the concentrated loads for the lane load to a maximum of 30 feet. This is probably based on the fact that the AASHTO 1944 Bridge Specifications[18] did not require an additional concentrated load for H-20 S-16 lane loadings. The H-20 S-16 truck loadings have a second axle spaced from 14 to 30 feet from the first heavy axle. This is probably the rationale for the limit of 30 feet in THD Supplement No. 1.[19] The 1949 AASHTO Bridge Specifications[20] made the lane loading negative moment requirement the same for HS-trucks. However, the 1953 THD Supplement No. 1[21] continued modifying the provision for continuous spans subjected

to lane load by limiting the spacing between the additional concentrated load to 30 feet. This limit had the effect of reducing the lane load negative moment maximums for some continuous spans. The 30-ft limit may also have been in recognition that the second large axle for an HS-load pattern is spaced at a maximum of 30 feet from the first large axle, or it might have been because the lane load approximately represents a train of trucks with a headway distance of 30 feet between trucks. It would have been more logical for the second concentrated load to be placed a minimum of 30 feet from the first instead of a maximum of 30 ft. Current specifications do not limit the distance between the two loads for negative moment lane loadings.

- ***Impact Load Provision.*** The 1944 AASHTO Bridge Specifications[22] required that the shortest length of adjacent spans in a continuous unit be used for the negative moment impact value. In 1949, AASHTO changed this to the current provision of using the average length of the adjacent spans. Both versions of THD Supplement No. 1[23,24] changed the impact provision for continuous units or other structures where discontinuous lane loadings are applied to be the loaded length as indicated by the influence line for the section of member considered. This change had the effect of slightly increasing the impact value.
- ***Special Axle Loads.*** The 1946 THD Supplement No. 1[25] added a provision that no axle load in excess of 24,000 lbs should be considered in the design of floor slabs. It further stated that either a single 24,000-lb axle or two 16,000-lb axles spaced four feet apart must be used for the design of H-20 and H-20 S-16 bridge floors (slabs, grids, timber) instead of the 32,000 lb axle. The provision was dropped in the 1953 THD Supplement No. 1[26] because the 1949 AASHTO Bridge Specifications[27] included the provision specifically for concrete bridge slabs. The AASHTO Bridge Specifications further limited the 24,000-lb axle to slab spans under 18 feet and the two 16,000 lb axles for slab spans over 18 feet. This provision had the effect of reducing the design load for many slab spans designed during that time. It has been found that some beams have been designed in Texas using the single 24,000-lb axle. It is believed to be an error for beams to have been designed this way. For this reason, any plans prepared during the period between approximately 1949 and 1961 with a design load of H20 or H20 S-16 that also had the THD Supplement No. 1[28] notation should be carefully evaluated.

Customary Rating Procedures

The initial load rating should always be re-calculated; the design loading should not be used as the final Inventory Rating. When a bridge was originally designed, the designer often had to select the next size of reinforcing bar, size of steel beam, or thickness of cover plate to meet the design stress criteria. Sizes that were larger than the theoretically perfect size of member result in Inventory Ratings significantly higher than the design loading. However, the design loading and date of original construction is an important part of the bridge data since they often provide a basis for determining initial routing of overload permits.

If the original design was made using an H-load, such as H-15, or H-20, then the equivalent HS Inventory Rating will usually be significantly less numerically. For example, an H-15 design might rate at HS-12. However, this difference means that the total inventory HS-load capacity is 43,200 pounds (two 19,200 lb axles and one 4,800 lb axle totaling 21.6 tons) as compared to the H-15 design of 30,000 pounds (15 tons).

The original design load should be determined from a review of the bridge plans if available. If the structure essentially matches an old TxDOT standard bridge, then the design load for that standard can be used for the Design Load (Item 31). Appropriate notation about this should be entered in the Bridge Folder and the electronic Bridge Inventory File should also be updated. However, caution should be used in accepting the design load in plans that used the THD Design Supplement No. 1[29,30] due to circumstances described above.

TxDOT bridge designs during the 1950s and 1960s sometimes intentionally were made using AS procedures with an allowable of 5 percent overstress for some components. Therefore, re-analysis using LF procedures is necessary for these bridges.

AS rating procedures are usually set at 55 percent of the material yield stress for steel structures and 50 percent of the material yield stress for Grade 40 reinforcing steel in concrete structures. When AASHTO first introduced the use of Grade 60 reinforcing steel in the 1970 Interim Bridge Design Specifications,[31] the allowable of 24 ksi for Grade 60 was assigned based approximately on the ratio of the Grade 60 ultimate strength to that of Grade 40. Thus the AS procedures were still compatible in factor of safety for concrete members.

LF rating procedures usually assign a dead load factor of 1.3 and live load factors of 2.17 (when computing Inventory Ratings) and 1.3 (when computing Operating Ratings). The resulting stresses or bending moments are compared to the yield of steel members or the ultimate capacity of concrete members also considering appropriate phi strength reduction factors.

Note that the value of 2.17 is the dead load value of 1.3 times 1.67. The load factor of 1.3 accounts for a 30 percent increase in all loadings, either dead or live, so as to provide a uniform safety factor. The factor of 1.67 accounts for the variability of live load configurations other than a standard HS-load pattern and further provides for potential overloads or loads in excess of the State Legal Loads.

Specific analysis of structures for overweight loads, particularly superheavy permits over about 280,000 pounds, is usually done with a load multiplier consistent with the restricted speed of the vehicle. Commonly this factor is about 1.1, with total stresses compared to an allowable of 75 percent of the yield for steel bridges or 75 percent of the ultimate capacity for concrete bridges including prestressed beam bridges. This procedure is explained more fully in Chapter 6, Routing and Permits.

Temporary repairs must not be considered for Inventory or Operating Ratings. However, temporary repairs are taken into account when assigning the operational status code of Item 41 to the structure. Temporary repairs are to be considered for the operational status code only until a more permanent repair is made and should not be used for more than four years. The Inventory Rating directly affects the Sufficiency Rating, and therefore temporary repairs must not be assigned any weight in the Load Rating calculations.

When the design loading is unknown or deterioration exists, load rating calculations must use all field information and conventional analysis techniques. Even when the design loading is known, the only acceptable method for accurate load rating is to do calculations based on the plans and known field measurements.

Rating Concrete Bridges with No Plans

A concrete bridge with unknown reinforcing details (no plans) can be rated for the State Legal Load (HS-20) at the Operating Level, which is currently defined for load rating purposes as an HS-20 design load, provided that the following two considerations are met:

- It has been carrying unrestricted traffic for many years.
- There are no signs of significant distress.

Notation that the ratings are assumed must be inserted in the permanent Bridge Record described in Chapter 8, and the bridge should be inspected at more frequent intervals, usually each year, until an inspection history of at least four years is developed. This procedure is summarized in detail by Figure 5-2.

Three additional considerations for rating concrete bridges with unknown reinforcing are:

- Bridge must exhibit proper span-to-depth ratios of the main members, which indicates that the original design was by competent engineers. In general, this consideration means that for simple span structures the span-to-depth ratio of main members should not exceed approximately 20. Span-to-depth ratios exceeding this ratio may indicate that the designer did not properly consider reasonable design truck loadings.
- Construction details, such as slab thickness and reinforcement cover over any exposed reinforcing, should conform to specifications current at the time of the estimated construction date.
- Appearance should show that construction was done by a competent builder.

A comparative original design rating can be used to estimate the amount of reinforcing in the main members. Normally, if the design was done prior to about 1950 and the above five considerations exist, then the amount of reinforcing can be estimated based on a percentage of the gross concrete area of the main beams (if tee-beam construction), or depth of slab (if slab construction). Two of the examples below describe this method, and a third example describes a method that can be used for prestressed beam bridges with no plans or other documentation.

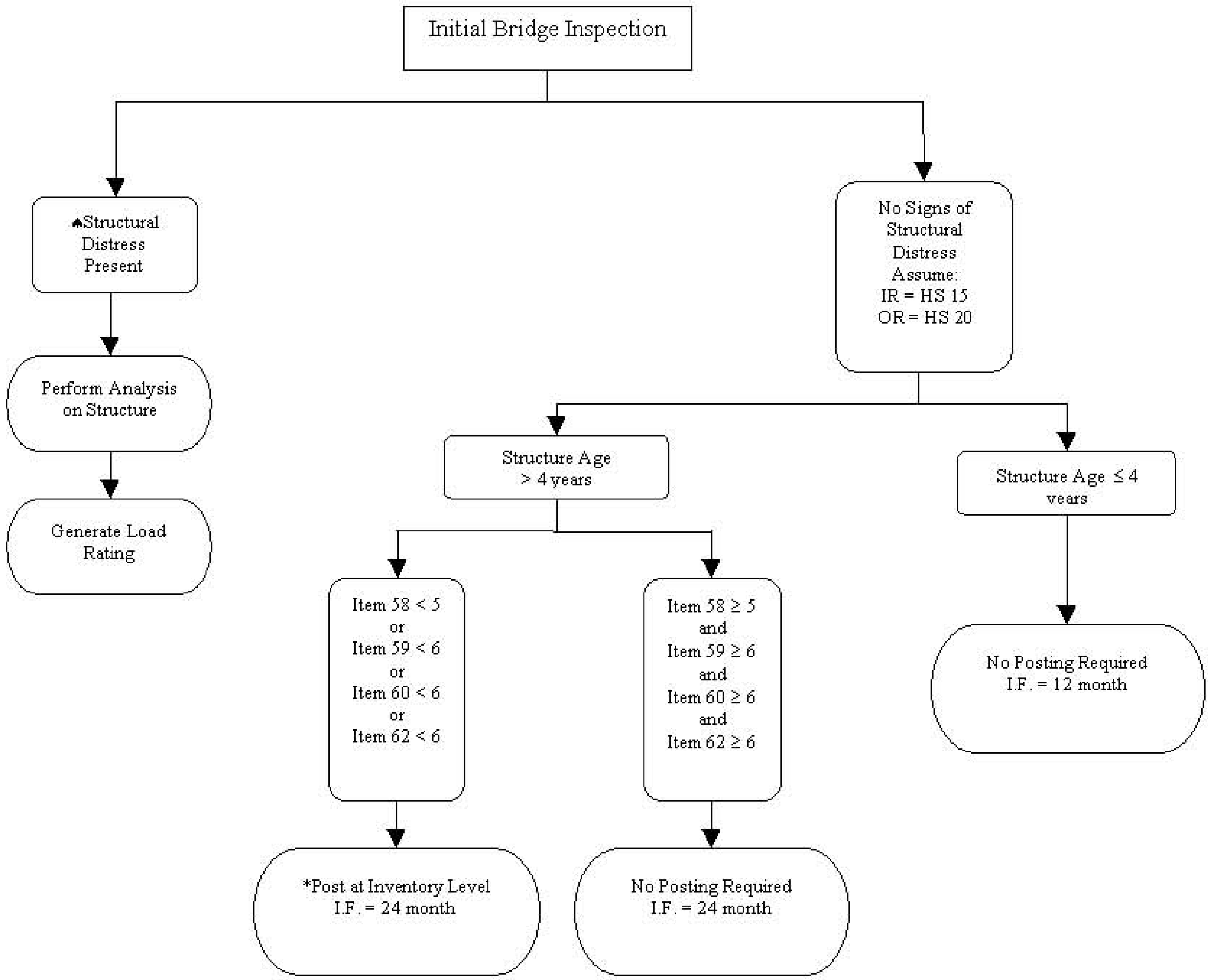


Figure 5-2: Load Ratings for Concrete Bridges without Plans
**Permit Trucks with gross or axle weights that exceed the state legal load limits will not be allowed to use these bridges.*
I.F. – Inspection Frequency.
♠Refer to AASHTO Manual for Condition Evaluation of Bridges, Chapter 7, Section 7.4.

Examples of Rating Concrete Bridges with no Plans

Example 1. A flat-slab bridge designed between about 1930 and 1960 can be assumed to have approximately 0.7 percent tension steel based on the total slab depth. Calculations with this amount of steel using AS procedures with stresses, materials, covers, and live load distribution appropriate to the AASHTO Bridge Specifications for the estimated date of construction should give at or very near an H-10, H-15, or perhaps an H-20 theoretical rating. Any other value would make the assumptions suspect. After this analysis is made, an analysis using LF procedures, HS loading, and current load distributions should give an acceptable rating. Flat-slab bridges constructed off-system can also often be rated by this procedure providing the above five considerations are also met. This method is not suitable for evaluation of FS slabs, which may be recognized as those with narrow roadways and tall integral curbs.

Example 2. A multi-beam concrete bridge built between about 1940 and 1965 can be estimated to have approximately 0.3 percent tension steel based on beam spacing and an estimated depth to the center of the steel group of 0.9 D where D is the total depth of the tee-beam. As in Example 1, an "old" AS rating can first be calculated for comparison. If reasonable, then a modern LF rating can be made with HS loading and the estimated amount of reinforcing steel. The amount of steel can be adjusted slightly so the AS design exactly matches an H-rating of H-10, H-15, or H-20.

Example 3. Some bridges built since about 1955 are composed of prestressed beams and no plans exist. This condition is often found for off-system bridges. The ratings should be done using conservative assumptions and good engineering judgment. One procedure would be to assume that the beams were designed to an H-15 loading in conformance with the estimated date of specifications. Using this assumption, an AS calculation can be made to estimate the even number of 7/16-in. 250 K strands. An LF rating using the HS-loading can then be performed based on this number and size of strand. In Texas, prestressed beams were probably never designed to less than H-15. Most beams have been designed to H-20 or HS-20. Texas prestressed beam fabricators keep good records of their products, and identification of the design loading may sometimes be tracked down.

All three of these examples should give H-ratings using AS procedures that are close to a realistic design load. For instance, a calculated value of H-14.4 could reasonably be assumed to verify that the original design was H-15. A calculated AS value of H-13 would be suspect, and further investigation will be required.

Ratings for Unusual Bridges

Unusual bridges, such as those composed of old railroad flat cars, can be rated, but care must be taken to ensure that the critical rating component is considered. For instance, flat cars were originally designed for a maximum point load combined with a uniform load over the whole car. When used for traffic loadings, even though the main two-girder members may give a good equivalent HS load rating, the transverse stiffening members and floor beams often control the live load capacity.

Another type of unusual bridge in Texas is the continuous cast-in-place (CIP) flat slab. Most of these bridges were designed in the 1940s and 1950s to an H-15 or H-20 load pattern. Unfortunately, the design negative moments were from the single truck load in one span. Current design procedures use a lane load with two concentrated loads in adjacent spans for the controlling negative moment case for longer continuous bridges or with two heavy axles of the HS-20 load pattern at variable spacing in adjacent spans for shorter continuous bridges. These bridges are thus under designed for HS-loadings and as a consequence many should actually be load posted. However, the current AASHTO Bridge Specifications[32] (Ref 5-2) do not differentiate between single-and multiple-lane distribution factors for slab bridges. As a result, this type of bridge has greater strength for multiple trucks positioned in the middle of the bridge span. Some structural evaluators will make live load distribution adjustments based on the number of lanes loaded for flat slab bridges. However, this must be done with care and properly correlated to two- or three-dimensional methods of analysis.

H- and HS-Load Ratings

Previously, all ratings were done with the equivalent H-truck, shown in Figure 5-1, or the HS-truck shown in Figure 5-1. Currently all ratings are only with the HS-truck. A moment equivalency conversion from H- to HS-ratings is not recommended since this process would assume that the structure was exactly designed for the given H-loading. In addition, continuous spans cannot be converted by this process. Most structures have a degree of capacity past the design H-load, particularly since load distribution assumptions of the AASHTO Bridge Specifications[33] have been made more liberal since the time many structures were commonly designed using H-loads. However, as previously explained, some bridges were intentionally designed with AS methods to a 5 percent overstress for some components.

It is not acceptable to ratio the design live load moments for an H-truck to the same moment for an equivalent HS-truck. For instance, if a 48-ft simple-span bridge has a design load of H-15, the design load for moment equivalency would be HS-10.8. However, due to the above reasons, the actual rating based on LF methods might easily be HS-9 or HS-13. A LF rating must be made.

Section 5
Legal Loads and Load Posting

Definition of State Legal Loads

State Legal Loads are those that may safely use any of our highways and bridges. Some routes and many bridges must be load-posted to protect them from possible damage. At this time, a load capacity of HS-20 is considered to best represent the State Legal Load for evaluation of the need for load posting.

Truck loads in Texas are considered "legal" if the gross load, axle load, axle configuration, length, and width are within the current size and weight laws or rules. The applicable laws are contained in the current volume of Texas Transportation Code.[34] See Section 623.0111 of the Texas Transportation Code for permit fees for selected numbers of counties, and see Section 201.8035 for requirements related to the notification of off-system municipalities and counties of deficient bridges.

The laws also provide for additional rules and regulations regarding truck weights and configurations as may be formulated by the Texas Transportation Commission.

In general, the laws require that the maximum gross load on any truck cannot exceed 80,000 lbs, the maximum load on any pair of tandem axles cannot exceed 34,000 lbs, and the maximum load on any single axle cannot exceed 20,000 lbs. Total length must not exceed 65 feet and total width must not exceed 96 inches. However, in 1989 the Texas Legislature enabled truck owners to pay an annual fee to allow their gross legal loads to be increased by 5 percent with any individual maximum axle load increased by 10 percent.[35] The bill was considered controversial because it allowed travel on any bridge, on- or off-system, even if it is load restricted. This portion of the Transportation Code was amended during the 77th Legislative Session to restrict vehicles possessing a permit of this type from crossing load restricted bridges unless the bridge is the only vehicular access.

There are other so-called legal loads, sometimes referred to as Bonded Trucks, such as ready-mix trucks, utility-pole trucks, garbage trucks, mobile cranes, oil well servicing equipment, etc., that have special rules passed by the legislature allowing special categories of loads and lengths exceeding the normal limits for trucks.

Most State Legal Loads do not have a greater effect on bridges than the current HS-20 design total gross load of 72,000 lbs even though they may have a total "legal" weight of 84,000 lbs.[36] This apparent contradiction is due to the different axle load configurations and numbers of axles.

Load Posting

Load posting is often required for structures that, due to their original design or condition, do not have the structural capacity to safely carry the State Legal Loads. Posting is usually necessary for bridges designed at a time when the design truck for the particular stretch of roadway was only H-10 or H-15, meaning gross truck loads of 20,000 or 30,000 lbs. Posting may be at Operating Rating levels provided that the Condition Ratings exceed those defined in Figure 5-3 and Figure 5-4 and other requirements are met. Otherwise, if the Condition Ratings are less than those defined, the Posting must be at Inventory Rating levels.

All load postings of a given truck size actually mean that two trucks of the posted capacity can safely pass on the bridge. This concept is often misinterpreted by those doing load ratings and making load posting recommendations. It is recognized that a bridge posted for an HS-5 (18,000 lbs gross load) can safely carry a single truck of significantly more than 18,000 lbs. No method ensures that only a single truck is on the bridge. Therefore, assume that two trucks of the same size could be passing on the bridge simultaneously.

However, some bridges, particularly off-system, are load posted assuming only one rating truck even though they may be wider than 18 feet. This condition usually occurs due to the volume of truck traffic, structure width or approach roadway width, striping, runners, etc. making them functionally one-lane bridges for trucks.

It is important to recognize that even though a bridge may have been designed to an H-15 loading, it may not need to be load posted due to considerations discussed previously, such as reinforcement or member size in excess of the theoretical amount, more liberal load distribution now used in analysis, and LF analysis methods which usually increase Inventory Ratings significantly more than the original design loading.

Senate Bill 220, 77th Legislature, 2001, amended Transportation Code, Section 621.301 to provide that a county may establish load limits for a county road or bridge only with the concurrence of the department. If a county determines that the load limit of a county bridge should be different than the load limit supported by a department inspection, the county must submit the proposed load limit to the district engineer. A request for a load limit must be accompanied by supporting documentation that is sealed by an engineer and that includes at a minimum: calculations supporting the proposed limit and a structural evaluation report documenting the condition of the bridge. The district engineer will give concurrence to a county's proposal in writing. If the department does not indicate concurrence or non-concurrence in writing within 30 calendar days of receipt by the department of a request that included all required documentation, the proposed load limit must be deemed concurred with by the department. The department may review the load limit and withdraw this concurrence at any time by providing written notification to the county. A county may appeal the decision of the district engineer by submitting a written request, along with the required documentation, to the executive director. The executive director will review the request and determine if department concurrence will be granted. The executive director's decision is final.

The recommended load posting of all off-system bridges must be supplied to the affected municipalities and counties. TxDOT provides the necessary posting signs and placement hardware. Should the local jurisdiction elect not to post the bridge, there is the possibility that all federal funds could be jeopardized or delayed for all transportation-related projects, on- or off-system, in that county.

A list of off-system bridges that are recommended for load posting must be sent by certified mail to the owner of the bridges. A signed copy of the cover letter is returned to TxDOT from the local jurisdiction official. Subsequently, after the appropriate load zone signs have been prepared by TxDOT, a letter is sent notifying the local jurisdiction as to where the signs, posts, and hardware may be picked up along with installation instructions. After the signs are installed, the local jurisdiction returns a statement of compliance to TxDOT.

Typical load posting signs are shown in Figure 5-5.[37] Texas must comply with posting time limits, which are set by the Code of Federal Regulations. The time limit for initial or revised posting after bridge inspection is 90 days after the change in status for on-system bridges. This time limit is extended to 180 days for off-system bridges.[38]

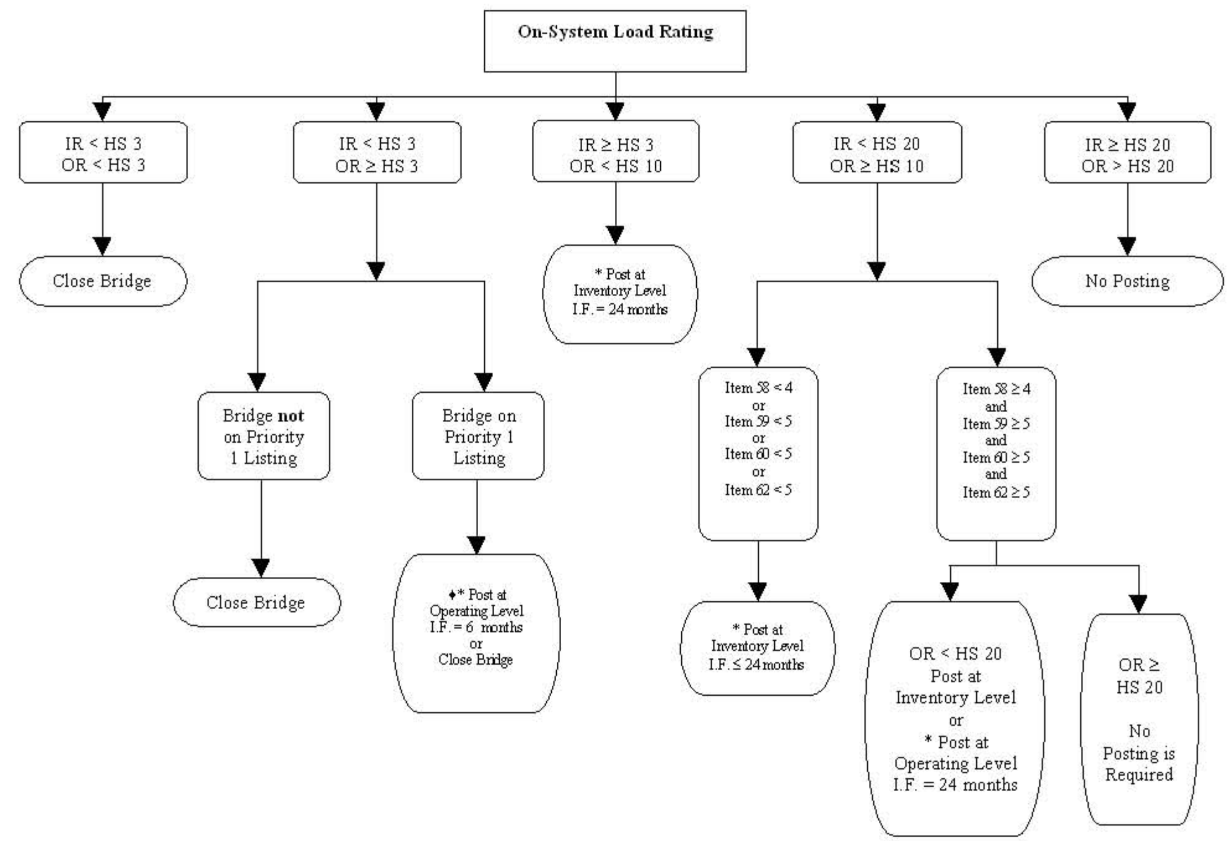


Figure 5-3: On-System Load Posting Guidelines
** Permit Loads will not be allowed on bridges that are load posted.*
♦ If the bridge has not been rehabilitated or replaced in 24 months then the structure shall be closed.
I.F. – Inspection Frequency.
OR – Operating Rating (Item 64)
IR – Inventory Rating (Item 66)

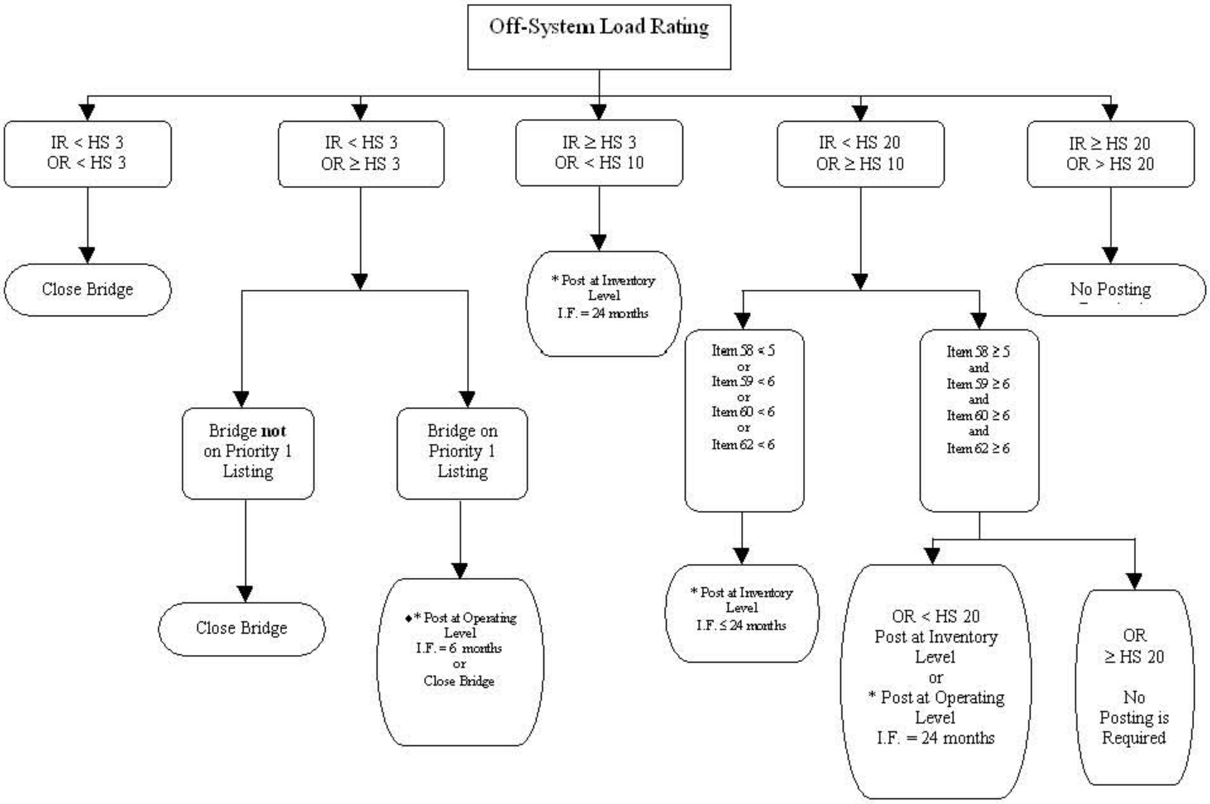


Figure 5-4: Off-System Load Posting Guidelines
** Permit Loads will not be allowed on bridges that are load posted.*
♦ If the bridge has not been rehabilitated or replaced in 24 months then the structure shall be closed.
I.F. – Inspection Frequency.
OR – Operating Rating (Item 64)
IR – Inventory Rating (Item 66)

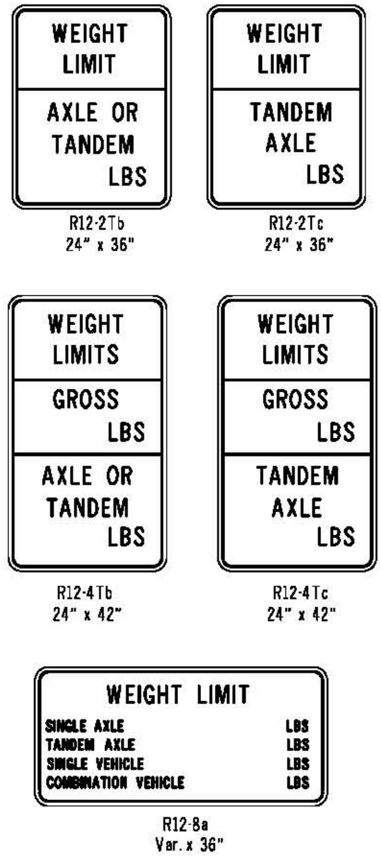


Figure 5-5: Typical Load Posting Signs

Procedures for Changing On-System Bridge Load Posting

The following table outlines the procedure for changing the load posting of an on-system bridge.

Changing Load Posting of an On-System Bridge

Step	Responsible Party	Action
1	District	Complete Form 1083R, "Recommended Change in Bridge Load Zoning," and send it to the Inspection Branch of the Bridge Division. If the request involves a new limit or a reduction of a current load limit, attach the most recent inspection report, plans (layouts and structural details), and any load ratings that support the recommended change.
2	Bridge Division	Review the request and supporting documents, and if review supports the recommended change, prepare a Minute Order for Commission approval based on the review.
3	Bridge Division	Provide approval notification and a copy of the approved Minute Order to the District.
4	Bridge Division	Notify the Motor Carrier Division of any bridge load restriction Minute Orders approved by the Commission.
5	District	On receipt of an approved Commission Minute Order, immediately erect signs indicating the proper load limit.
6	District	Notify the Bridge Division of the date that the sign was erected.

Under the following conditions, the District should submit a completed Form 1083R showing reasons for a restriction removal to the Bridge Division's Inspection Branch:

- ♦ Repair or rehabilitation of a bridge that increases load capacity and eliminates a load restriction.
- ♦ Construction of a new bridge that replaces one with a load restriction.

Procedures for Emergency On-System Bridge Load Posting

The following table outlines the procedure for changing the load posting of an on-system bridge in an emergency.

Changing Load Posting of an On-System Bridge in an Emergency

Step	Responsible Party	Action
1	District	Notify the Bridge Division's Inspection Branch by telephone that an emergency load restriction is required. Identify deficiencies that justify the placement of an emergency load limit.
2	Bridge Division	Work with the District to determine the load limit, if required, and verbally authorize an emergency load restriction for a period not to exceed 60 days if necessary.
3	Bridge Division	Prepare a letter to the District for signature by the Director of the Bridge Division authorizing the temporary load limits and specifying the duration of the temporary limit.
4	Bridge Division	Verbally notify the District of official approval of the emergency load limit.
5	Bridge Division	Notify the Motor Carrier Division of any bridge load restriction.
6	District	On receipt of verbal approval by the Bridge Division, immediately erect signs indicating the emergency load limit.

If the emergency load limit is required for a period longer than 60 days, the District should submit a request to the Bridge Division for the emergency load restriction to remain in place for another 60 days. If the bridge is not replaced or repaired before the emergency load restriction extension expires, the District should submit a request to the Bridge Division for a permanent load restriction following the procedures for changing on-system bridge load postings.

Closure of Weak Bridges

A memo to all District Engineers, titled "Closing of Weak Bridges,"[39] from C.W. Heald, dated Feb 12, 1999, (Ref 5-14) contains procedures to be followed in Texas for the closure of bridges. Bridges with less than an HS-3 Operating Rating capacity must be closed according to the Texas Load Posting Guidelines presented in Figure 5-3 and Figure 5-4. These policies must be followed for on-system bridges and are strongly recommended for the municipalities and counties with jurisdiction over off-system bridges. Bridges with Inventory Ratings less than HS-3 but with Operating Ratings greater than HS-3 may remain open for a limited amount of time. If it is desired to leave a bridge in this category open, then the inspection frequency must not exceed six months and the bridge must be categorized for Priority 1 rehabilitation or replacement funding. Categorized Priority 1 means that funds are allocated for rehabilitation or replacement. This categorization is explained more fully in the section of Chapter 7 titled Texas Eligible Bridge Selection System. If after 24 months the bridge has not been rehabilitated or replaced, then it should be closed.

Off-System Bridge Closure Procedures

If inspection reveals deterioration that affects an off-system bridge's ability to safely carry vehicular traffic, the department may use the following procedure to recommend that it be closed for safety reasons:

Recommending Off-System Bridge Closures

Step	Responsible Party	Action
1	Bridge Division	The Bridge Division will immediately notify the district if it determines that a bridge should be closed based on the results of an inspection conducted by the Bridge Division.
2	District	The district will verify as soon as possible the condition of a bridge recommended for closure by a consultant.
3	District	The district will immediately notify the local entity of a valid closure recommendation. The district will inform the local entity that its participation in the TxDOT Participation Waived and Equivalent Match Program depends on full compliance with departmental closure and posting recommendations and that failure to follow closure recommendations could result in the loss of federal funds. The district will promptly update the Bridge Inspection database to reflect the closure recommendation. (See Item 41 in the coding_guide.) Note: The department will not conduct another formal inspection of the bridge until it is repaired or replaced.
4	Local Entity	The local entity will close the bridge and notify the district when the bridge is closed to traffic.
5	District	The district will verify closure of the bridge when it receives notification and will include a photo or certified documentation verifying the closure in the bridge inspection file. The district will promptly update the Bridge Inspection database to reflect the closure status of the bridge. (See Item 41 in the coding_guide.)
6	District	If the bridge will remain closed for an extended period of time, the district will verify and document with a photo that the bridge is still closed to traffic as part of the regular inspection cycle.

[1] Manual for Condition Evaluation of Bridges, AASHTO, 1994
[2] Standard Specifications for Highway Bridges, AASHTO, 1994
[3] Guide for Selecting, Locating, and Designing Traffic Barriers, AASHTO, 1977
[4] Manual for Condition Evaluation of Bridges, AASHTO, 1994
[5] Manual for Condition Evaluation of Bridges, AASHTO, 1994
[6] Manual for Condition Evaluation of Bridges, AASHTO, 1994
[7] THD Supplement No. 1, TxDOT, September 1953
[8] Standard Specifications for Highway Bridges, AASHTO, 1944
[9] Standard Specifications for Highway Bridges, AASHTO, 1949
[10] THD Supplement No. 1, TxDOT, June 1946
[11] THD Supplement No. 1, TxDOT, September 1953
[12] Standard Specifications for Highway Bridges, AASHTO, 1949
[13] Standard Specifications for Highway Bridges, AASHTO, 1944
[14] Standard Specifications for Highway Bridges, AASHTO, 1949
[15] THD Supplement No. 1, TxDOT, September 1953
[16] Standard Specifications for Highway Bridges, AASHTO, 1944
[17] Standard Specifications for Highway Bridges, AASHTO, 1944
[18] Standard Specifications for Highway Bridges, AASHTO, 1944
[19] THD Supplement No. 1, TxDOT, June 1946
[20] Standard Specifications for Highway Bridges, AASHTO, 1949
[21] THD Supplement No. 1, TxDOT, September 1953
[22] Standard Specifications for Highway Bridges, AASHTO, 1944
[23] THD Supplement No. 1, TxDOT, June 1946
[24] THD Supplement No. 1, TxDOT, September 1953
[25] THD Supplement No. 1, TxDOT, June 1946
[26] THD Supplement No. 1, TxDOT, September 1953
[27] Standard Specifications for Highway Bridges, AASHTO, 1949
[28] THD Supplement No. 1, TxDOT, September 1953
[29] THD Supplement No. 1, TxDOT, June 1946
[30] THD Supplement No. 1, TxDOT, September 1953
[31] Interim Specifications for Highway Bridges, AASHTO, 1970
[32] Standard Specifications for Highway Bridges, AASHTO, 1994
[33] Standard Specifications for Highway Bridges, AASHTO, 1994
[34] Texas Transportation Code, Title 7, Chapter 621
[35] Texas Transportation Code, Section 623.011.
[36] Texas Transportation Code, Section 623.011.
[37] Texas Manual on Uniform Traffic Control Devices, 1980
[38] Closing and Posting Recommendations for Off-System Structures, Memo from Robert L. Wilson, P.E., TxDOT, October 1997
[39] Closing of Weak Bridges, Memo from C.W. Heald, P.E., TxDOT, February 1999

Bridge Design Manual

Texas
Department
of Transportation

December 2001

Chapter 6
General Design Controls

Contents:

Section 1
Specifications

AASHTO Specifications

The *Standard Specifications for Highway Bridges*[1] adopted by the American Association of State Highway and Transportation Officials (AASHTO) controls the design of bridges and culverts under highway traffic. These specifications were first published in 1931. They have been revised and republished approximately every four years since then. In the past 40 years, tentative or interim revisions have been published annually that carry the full force of the specifications. Guide specifications have been published that have the status of suggested or trial specifications. After a few years of use and necessary revisions, they are generally incorporated into the regular specifications. Separate specifications and manuals have also been published for unusual types of structures or particular areas of bridge management.

A supplement[2] to the 1944 *AASHO Design Specifications* entitled THD Supplement No. 1 was issued originally on May 24, 1945. It contained 18 items that had approval dates between October 14, 1944, and the issue date. The supplement was revised and reissued on June 13, 1946, this time with 17 items. The items pertaining to live load did not appear to change during this time frame.

THD Supplement No. 1 called for reduced axle loads in the design of concrete slabs. Many structures designed during the era will not have an acceptable rating and are usually replaced rather than rehabilitated or widened. Careful consideration should be given to structures where THD Supplement No. 1 is referenced in the plan notes.

AASHTO Specifications are proposed, discussed, and approved by the AASHTO Highway Subcommittee on Bridges and Structures, which is composed of the 50 state bridge engineers and representatives from the Federal Highway Administration, Puerto Rico, Guam, Mariana Islands, and seven Canadian provinces. The membership is divided among approximately 20 technical committees, each responsible for a certain area of the specifications. These committees continually monitor their specification areas, consider suggestions for change from other sources, and present any needed revisions to the full subcommittee for consideration and possible approval for AASHTO publication. Revisions may be suggested by users of the specifications, researchers, or organizations representing industries that supply the various bridge materials or components. Industry proposals are usually based on the latest research in the area.

FHWA and Industry Participation

The Federal Highway Administration (FHWA) sits in review of state practice involving the use of federal funds. They are often in the position of strongly advocating specification revisions, usually perceived to respond to national safety concerns. FHWA has been known to enforce its own specifications on the states in sensitive areas but currently appears to be following AASHTO-approved specifications and manuals.

Industry participation in maintenance of the AASHTO Specifications is very important. Research sponsored by organizations representing suppliers and fabricators of concrete products, reinforcing steel, prestressing systems, structural steel, culvert pipe, timber, and aluminum is the basis for much of the current specifications. Research sponsored by AASHTO through the National Cooperative Highway Research Program (NCHRP) has also furnished valuable background. The FHWA directly funds research that often finds its way into the specification. Research sponsored by individual states using Highway Planning and Research funds has been useful. Texas conducts a Cooperative Highway Research Program with state universities using these funds.

Recent Changes

Although research drives many of the specifications, there are often compromises worked out during committee deliberations. The rationale behind many provisions is lost due to lack of written commentary. Some of the early provisions, reflecting the wisdom of dominant bridge engineers of that time, still remain in the specifications. Lately, through an NCHRP project, the AASHTO Specifications have been completely rewritten using current knowledge and specification logic. A complete commentary is provided in the new version as a historical record of specification changes. This commentary is found in the "Load and Resistance Factor Specification," which has the status of a guide specification in Texas.

While generally bound to compliance with the AASHTO Specifications, Texas design practice departs from it in a few areas. Such departures are based on proven experience or local research. Some of these areas will be identified in the following sections of the manual.

AREMA Specifications

Design of structures to carry railroad traffic is controlled by the American Railway Engineering and Maintenance-of-Way Association (AREMA), *Manual for Railway Engineering.*[3] Additionally, some railroad companies have expanded interpretations or provisions that must be followed for structures supporting their trains. These exceptions will be addressed in Chapter 9, Section 5 of this manual.

Wind-Sensitive Structures

Wind-sensitive structures are subject to the AASHTO *Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.* These structures will be discussed in Chapter 9, Sections 19, 20, and 21 of this manual.

Section 2
Loading

Overview

Most short span bridges can be adequately designed using only dead and live loads. Dead load is simply the weight of the structure. Live load is whatever the governing specification requires for service loads to be resisted by the structure throughout its life.

The loading of bridges and structures associated with bridges, such as sign supports, is discussed in several mandatory specifications, as discussed in Chapter 3 of this manual. The most commonly applicable specifications include the following:

- AASHTO *Standard Specifications for Highway Bridges*
- AREMA Specifications
- AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*

The loading criteria presented in each of these specifications are mandatory for the appropriate structures covered by each. This section provides some additional information concerning the Texas Department of Transportation (TxDOT) policy for each of these specifications.

Additionally, loads on bridges are thoroughly discussed by the American Society of Civil Engineers[4] (ASCE). Although not directly applicable to Texas bridge design, this reference can be used as further guidance.

AASHTO Standard Specifications for Highway Bridges

Early Texas specifications required structures to safely carry 125 pounds per square foot as a live load or a 20 ton roller, whichever required the greater strength. The 1931 American Association of State Highway Officials (AASHO) specifications established the H10, H15, and H20 truck loadings, truck trains, and equivalent lane loadings that remain in effect today. The 1941 third edition added the current group of H-S truck trailer loads with an equivalent lane loading heavier than for H loads. The 1944 version made the equivalent lane load the same for H and H-S loading. A military loading for interstate highways was introduced by the Bureau of Public Roads in 1956.

Highway loads have increased in size and frequency during the past 50 years, but the design load has remained virtually the same. The effects of a 36 ton HS20 design load are generally a little more severe than the current 40 ton legal 18-wheeler because of the number and spacing of the rear axles. The trucking industry continually seeks to raise the legal load and size limits. A few states design for an HS25 loading, but the American Association of State Highway and Transportation Officials (AASHTO, formerly AASHO) has not seen fit to require this. Texas has used HS25 for some bridges in the Texas-Mexico border area where heavier loads due to international truck travel are encountered. HS25 is also being considered for new bridges on North American Free Trade Agreement (NAFTA) truck corridors. If justified, other areas in Texas may use HS25 upon approval from the Bridge Division. Some states require their bridge designs to safely carry a family of overload truck configurations permitted over their highways. This practice is recognized by AASHTO.

Application of Live Loads. Texas generally uses only the live loadings prescribed by the AASHTO Specifications, applied as shown in Figure 6-1 and Figure 6-2. As discussed above, there may be instances where alternative loadings are considered. Bridges on all highway systems are currently being designed, at the minimum, for the HS20 loading and also for the military loading. The military loading only controls span lengths up to 37 ft. and does not apply to deck slabs and direct traffic box culverts.

When applying live load, the following guidelines should be followed:

- Specified design live loads are placed in each traffic lane as necessary to cause maximum stress.
- Only one design truck per lane is placed in a span or unit.
- Equivalent lane loads are placed in spans as necessary to produce maximum stress. For continuous spans, the lane loading shall be continuous or discontinuous as to produce a maximum value, and a second concentrated load is used to produce maximum negative moment. Refer to AASHTO's *Standard Specifications for Highway Bridges* for additional information concerning continuous spans.

There has been research and statistical analysis directed toward a realistic mix of vehicle loads for various types of bridges. These deliberations are very complex but it is reported that the AASHTO lane loading may be unrealistically severe for long span bridges. Because of this, revised loadings are occasionally negotiated for long span bridges.

Impact Due to Live Loads. Live loads shall be multiplied by an impact factor to increase the effects of the live load to account for effects due to vibration and impact per AASHTO Specifications. For the analysis of the structure, the effects of impact shall be transferred from superstructure to substructure but shall not be included in loads transferred to structural elements below the ground line for the analysis of those structural elements.

Application of Other Loads. In addition to dead loads and live loads, the following AASHTO Specifications loads are common to bridges:

- *Centrifugal Force.* Centrifugal forces due to live load may be treated as shown in Figure 6-3.
- *Longitudinal Force.* Longitudinal forces due to live load are thoroughly described in AASHTO Specifications.
- *Wind Load.* Wind loads must be considered but will seldom control the design of grade separation or stream crossing structures less than 25 ft. above the ground.

The 1931 specification required bridges to resist a wind load of 30 pounds per square foot on 1½ times the area as seen in elevations, plus all girders in excess of two in the cross section. The origin of this loading is lost in antiquity. The load was changed to 50 pounds per square foot on 1½ times the area in 1953 and then to the current 50 pounds per square foot on the area as seen in elevation in 1957. Trusses and arches are designed for 75 pounds per square foot.

The AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*[5] contains a more refined treatment of wind loads. There is an excellent treatise on wind with a large bibliography in the ASCE Transactions, Paper No. 3269.[6]

If the structure is considered sensitive to wind, the forces are applied according to Figure 6-4 . Wind on the live load is also covered on this figure.

Long span structures may justify more sophisticated analyses, including wind tunnel tests, to investigate the dynamic performance of the design.

Other loads mentioned in the AASHTO Specifications are treated as follows.

- *Electric Railway Loads*. Electric railway loads are a holdover from early specifications, but streetcars are becoming popular again.
- *Sidewalk Loading*. Sidewalk loading shall be applied as described in the specifications.
- *Curb Loading*. Curb loading shall be applied as described in the specifications. Curbs are seldom used on bridges.
- *Railing Loading*. Current railing standards are designed to AASHTO requirements, but the trend is toward crash testing to verify railing details.
- *Stream Current*. Stream current should be considered but rarely controls the design. Designing for drift loads is highly speculative. If significant drift is expected, wall or webbed piers should be used and careful attention given to span lengths and skew angle.
- *Ice Pressure*. Ice pressure does not occur in Texas.
- *Buoyancy*. Buoyancy is important for cofferdams but is seldom a factor in ordinary bridge design.
- *Earth Pressure*. Use an equivalent hydrostatic pressure of 40 pounds per cubic foot unless more exact determinations are justified.
- *Earthquake Motions*. At the present time, TxDOT does not design for earthquakes.
- *Temperature, Shrinkage, and Rib Shortening*. These factors are listed in the combination of loads, but they are actually internal deformations that can result in stress redistribution. Temperature deformations, as defined in the specifications, are considered in the design of substructure for continuous units. Shrinkage is seldom treated analytically. Rib shortening is a secondary effect that should be considered in the design of arches.
  Temperature, shrinkage, and creep have been observed to have a significant effect on large concrete box girders. Research has verified some of the parameters and methods available to analyze their effects. All concrete box girders should undergo such an analysis.

Load combinations. Seven combinations of the above loads (Groups I through VI and X in the AASHTO Specificaitons) must be considered in the design of bridges.

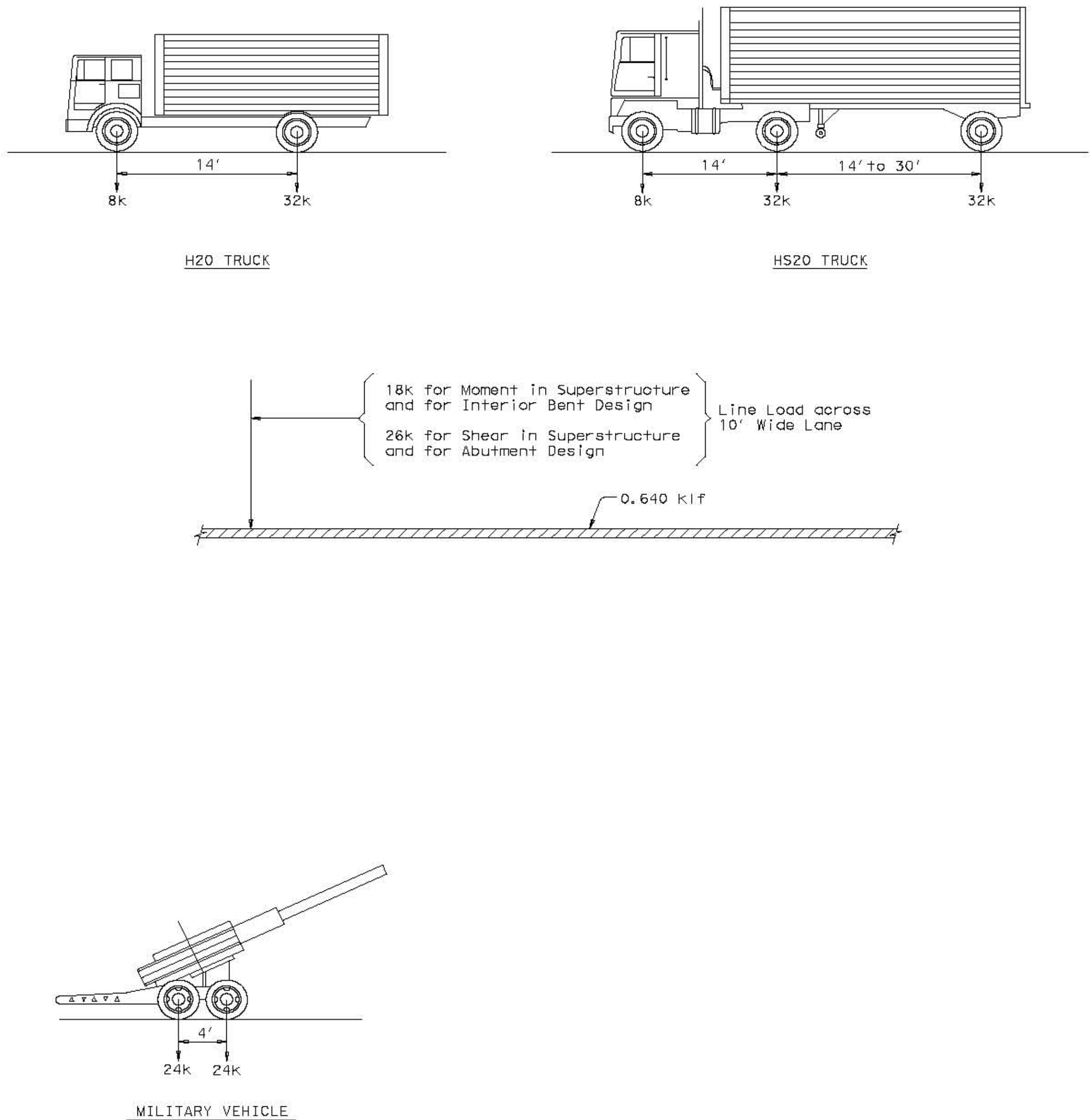


Figure 6-1. AASHTO HS20 Truck, H20 Truck, and Alternate Military Live Loads (see following explanatory notes) (Online users can click here to view this illustration in PDF.)

Explanatory Notes for Figure 6-1

Figure 6.1 shows equivalent lane loading for HS20 and H20 trucks. When applying the live load to the design, remember the following:

- Reduce live load effects by 10 percent if three lanes are loaded.

- Reduce live load effects by 25 percent if four or more lanes are loaded.
- Increase for impact per AASHTO Specifications.

Regarding 18k for moment and 26k for shear notes: An additional concentrated load is used in the design of negative moment regions for continuous spans.

Regarding 0.640 klf notes: In the design of continuous bridges, the uniform load is placed in spans only as necessary to produce maximum stress.

Alternate military loading, developed by the FHWA in 1956, represents heavy military vehicles.

All bridges on the U.S. Interstate Highway System, or any highway bridge that may carry heavy truck traffic, are to be designed using HS20 or the alternate military loading, whichever produces the greatest stresses.

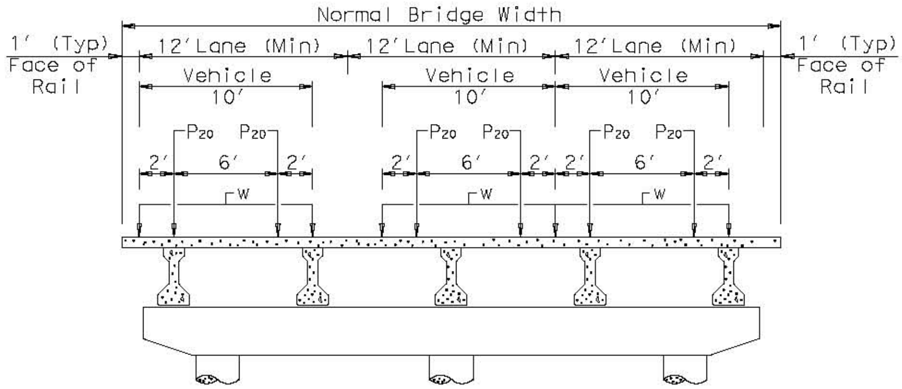


Figure 6-2. Applying Live Load on the Structure for Slab, Beam, and Bent Design (see following explanatory notes) (Online users can click here to view this illustration in PDF.)

Explanatory Notes for Figure 6.2

LL = $2P_{20} + 10w$; Live load reaction per land; controlling between truck and land load, increase for impact if applicable.

P_{20} = 16k; The load on one rear wheel of HS20 or H20 truck, increase 30 percent for impact if applicable ($1.3P_{20}$ = 20.8k).

$$w = \frac{LL - 2P_{20}}{10}$$

The uniform load portion of LL (k/ft).

Slab Design. Specific slab design moments and distribution widths are specific by AASHTO. Uniform load (w) is not applicable and is ignored. Wheel load (P_{20}) shall be increased 30 percent for impact ($1.3P_{20}$ = 20.8k). Exterior P_{20} shall be placed 1 ft. 0 in. from face of rail when designing cantilever.

Beam Design. Use live load distribution factors specified by AASHTO.

Bent Design. The live load is distributed to the stringers assuming the slab is simply supported at each beam. The live load is placed at critical locations, and using combinations of loaded lanes as to produce the maximum stresses. Wheel load (P_{20}) shall be increased 30 percent for impact ($1.3P_{20}$ = 20.8k) for substructure elements above the ground line.

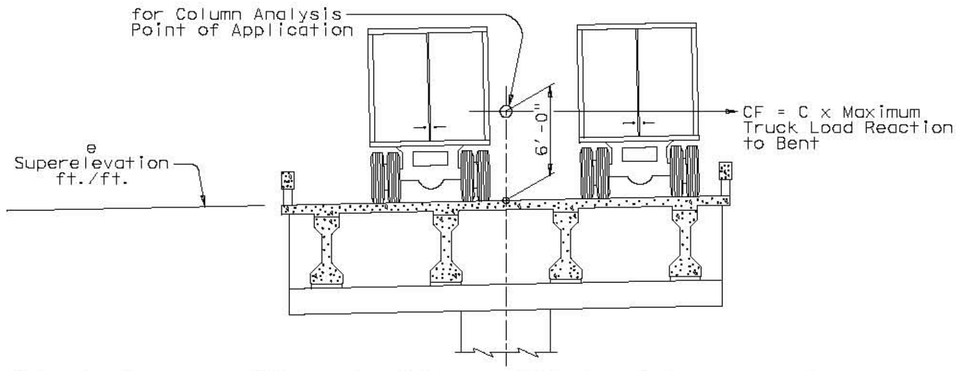


Figure 6-3. Application of Centrifugal Force (CF) (see following explanatory notes)
(Online users can click here to view this illustration in PDF.)

Explanatory Notes for Figure 6-3

Centrifugal Force (CF) = RF (n)(C)LL_{TL})

Where:

RF = Reduction in load intensity factor; applicable only if $n \geq 3$, per AASHTO

N = Number of loaded lanes

C = Centrifugal force in percent of live load

LL_{TL} = Live load due to truck load without impact, kips (k)

The direction of CF is radial. If the bent is skewed, the radial force shall be resolved into parallel and perpendicular components.

The centrifugal force in percent of live load shall be calculated by the following:

$C = 0.0000117S^2D$

Where:

S = Design speed in mph, if no superelevation is present

D = Degree of curvature along the baseline

To account for the effects of superelevation, truck speed (S) may be taken as the following:

$S = \sqrt{\frac{85950}{D}(e + 0.15)}$ where e = superelevation rate

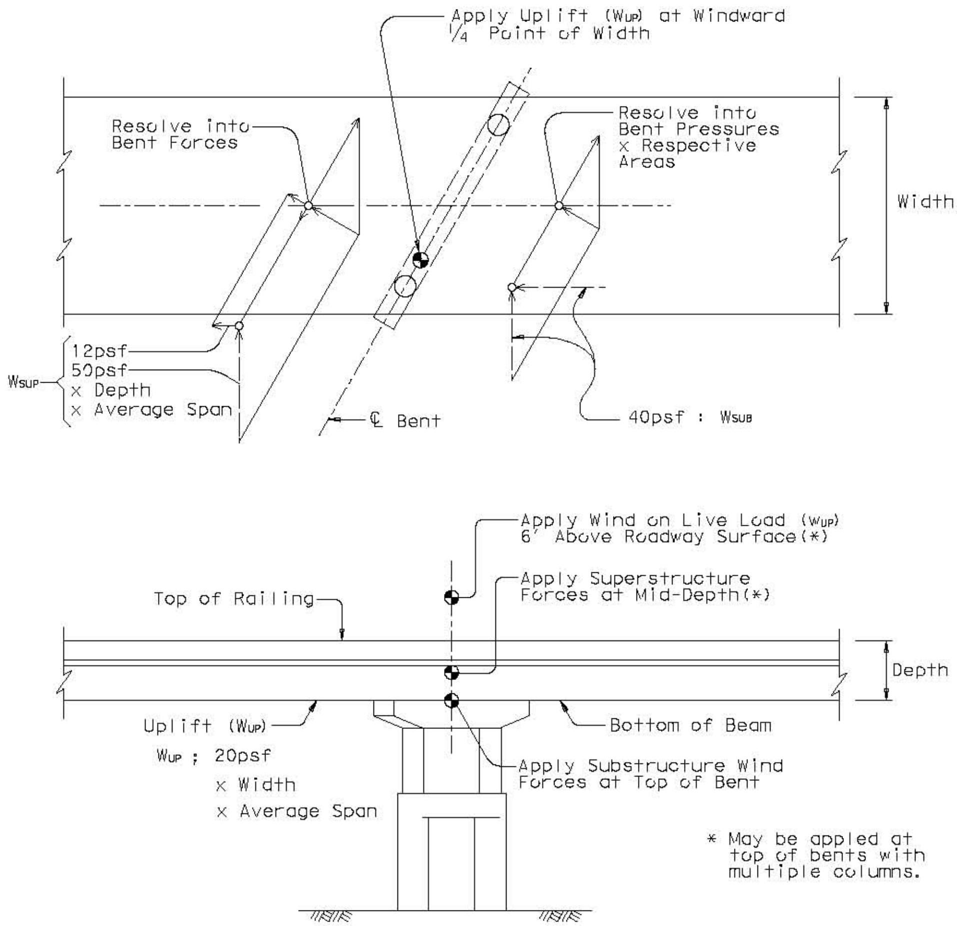


Figure 6-4. Application of Wind Loads, including W_{SUP}, W_{SUB}, W_{UP}, WL (see following explanatory notes) (Online users can click here to view this illustration in PDF.)

Explanatory Notes for Figure 6-4

- Wind on superstructure (W_{SUP}) is 50 psf transverse, applied simultaneously with 12 psf longitudinal and resolved into components parallel and perpendicular to the bent.
- Wind on the substructure (W_{SUB}) is 40 psf transverse, applied simultaneously with 40 psf longitudinal and resolved into components parallel and perpendicular to the bent.
- Uplift (W_{UP}) is 20 psf of deck and sidewalk plan area applied at the windward ¼ point of the transverse superstructure width.
- Wind on live load (WL) is 100 plf transverse, applied simultaneously with 40 plf longitudinal and resolved into components parallel and perpendicular to the bent.

AREMA Specifications

The American Railway Engineering and Maintenance-of-Way Association specifications for loading are strict, but can be followed without any undue expense. Refer to the AREMA specifications for loads and methods of application.

AASHTO Standard Specifications of Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Used for the design of sign supports and poles, these specifications frame a different type of structural design for which wind speed, ice load, and shape factor are important considerations. Refer to the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* for loads and methods of application.

Section 3
Load Distribution

Overview

Truck wheel loads are delivered to a flexible support through compressible tires, which makes it very difficult to define the area of the bridge deck significantly influenced. Computerized grid systems and finite element programs can come close to reality, but they are complicated to apply and are limited by mesh or element size and by the accuracy with which the mechanical properties of the composite materials can be modeled. These two- or three-dimensional problems are reduced to one dimension through various empirical distribution factors given in the AASHTO Specifications.

These distribution factors have been derived from research involving physical testing and/or computerized parameter studies. In order to simplify the design procedure, the number of variables was reduced to a minimum consistent with safety and reasonable economy, according to the judgment of the AASHTO Highway Subcommittee on Bridges and Structures. The factor S/5.5, so developed, has been used for many years to determine the portion of a wheel load to be supported by steel or prestressed concrete stringers under a concrete slab. Other variables, such as span aspect ratio, skew angle and relative stiffness between stringer and slab, are not considered except for occasional special bridges. The conservatism of this approach may account for some of the reserve strength regularly observed when redundant stringer bridges are load tested. Similarly, experience has shown that concrete slab spans and slabs on stringers will invariable support much more load than predicted by empirical analysis.

Load Distribution

Treatment of wheel load distribution to the various bridge components in the AASHTO Specifications is as follows:

- ***Longitudinal Beams (Stringers).*** Distribution factors given in the specifications are used almost exclusively. Occasionally, special conditions will justify the use of a discrete element grid and plate solution.

  For simplicity of calculation and because there is no significant difference, the distribution factor for moment is used also for shear. Composite dead loads are distributed equally to all stringers except for extraordinary conditions of deck width or ratio of overhang to beam spacing. Live load is distributed to all types of outside beams assuming the deck to act as a simple cantilever span supported by the outside and the first inside stringer.

- ***Transverse Beams (Floorbeams).*** For the few cases where floorbeams have been used without stringers on highway bridges, it has appeared proper to calculate reactions assuming the deck slab to act as a continuous beam supported by the floorbeams. No transverse distribution of wheel loads is allowed unless a sophisticated analysis is used.

- ***Concrete Slabs - Reinforced Perpendicular to Traffic (Slab on Stringers).*** For this component, distribution of the wheel load is built into a formula for moment. TxDOT designs are standardized according to the requirements of the current AASHTO Specifications. Span length of slabs on prestressed concrete stringers may be taken as the clear distance between flanges and adjusted to the flange quarter points for steel stringers.
- ***Slab Overhang Design.*** This design is also standardized.
- ***Concrete Slabs - Reinforced Parallel to Traffic (Slab Spans).*** Loads are distributed according to the AASHTO Specifications. The approximate formula for moment is not used.

  For skews up to 30 degrees, main reinforcing is parallel to traffic, and no additional edge beam strength is needed for usual railing conditions. For skews greater than 30 degrees, reinforcing is perpendicular to the bents and edge beam strength is provided and reinforced parallel to traffic.
- ***Concrete Slabs - Reinforced Both Ways.*** Divide the load between transverse and longitudinal spans according to the formulas for slabs supported on four sides. Use the appropriate load distribution in each direction.
- ***Timber Flooring, Composite Wood-Concrete Members, and Glued Laminated Timber Decks.*** Timber is not used in new structures.
- ***Steel Grid Floors.*** The specifications are followed closely. This type of construction is seldom used in Texas.
- ***Spread Box Girders.*** The specifications are followed closely. This type of construction is seldom used in Texas.
- ***Precast Concrete Beams Used in Multibeam Decks (Box Beams).*** The latest standard designs and current special designs comply with the current AASHTO Specifications. The distribution factor is a function of box width, overall bridge width, number of lanes, and span length. The simplified values for K shown in the specifications are usually used for final designs.
- ***Other Structure Types.*** See Chapter 7 for distribution factors for other structure types not listed here.

Horizontal Loads

Horizontal loads on the superstructure distribute to the substructure according to a complicated interaction of bearing and bent stiffness. For continuous steel units, the following method will usually be sufficiently accurate:

- Apply transverse loads times the average adjacent span length.
- Apply longitudinal loads times the unit length to the fixed bents according to their relative stiffness.
- Calculate deformations due to temperature change of 70 degrees and convert to forces according to the stiffness of the fixed bents.

- Centrifugal force is based on the truck load reaction to each bent.
- Friction in expansion bearings can usually be ignored but, if its consideration is desirable, the maximum longitudinal force may be taken as 0.10 times the dead load reaction for rocker shoes and polytetra fluoroethylene (PTFE) sliding bearings.

For prestressed concrete beam spans and units on elastomeric bearings, fixity is superficial and all bearings are approximately the same stiffness. It will usually be sufficiently accurate to distribute horizontal loads in the following manner:

- Apply transverse and longitudinal loads times the average adjacent span length. The concentrated live load for longitudinal force would be located at each bent.
- Centrifugal force is based on the truck load reaction to each bent.
- Forces due to temperature deformations may be ignored except for bearing design. If temperature consideration is desirable, deformations may be based on 40 degree temperature change.
- Bearing stiffness may be based on a shear modulus of 175 psi. For a complete discussion on bearing pad design, refer to the paragraph titled "Elastomeric Bearings" in the Chapter 9 discussion of Design Recommendations for Bearings.

Section 14

Reinforced Concrete Box Culverts

Background

Texas standard box culvert detail sheets can be found dating from 1918. Types of culverts identified are:

- Laminated timber
- Patented creosoted timber
- Stone walls with stone slab
- Stone walls with precast reinforced concrete slab
- Vitrified clay segmental block, round, or flat bottom arch
- Masonry arch
- Interlocking precast concrete u-shaped sections
- Concrete wall with footing and reinforced concrete simple slab
- Cast-in-place single boxes, reinforced for positive moment only
- Cast-in-place single boxes, reinforced as a frame
- Cast-in-place multiple boxes, reinforced as a frame
- Precast single box sections
- Precast two-piece single box sections

The most widely used culverts are the reinforced concrete single and multiple boxes.

Early Designs

From the late 1940s to the middle 1990s, TxDOT maintained an extensive set of culvert and wing wall standard detail sheets. Used directly, or modified and used for higher fill installation, they were the basis for many millions of dollars worth of culvert construction.

Design procedures for these boxes did not exactly conform to AASHTO requirements. The original 1948 standard designs used service load methods and were based on these assumptions:

- Vertical earth pressure 120 pcf times 0.7
- Lateral earth pressure 30 pcf equivalent fluid
- Live load was a 12 kip wheel
- Live load distribution according to a Westergaard article in “Public Roads,” March 1930
- Two feet of surcharge and full lateral pressure used for corner moments

- No lateral pressure used for positive moments
- Live load in one span only for positive moment
- Allowable stresses in concrete, 1,000 psi and steel, 18,000 psi
- Shear in slabs not considered
- Moment distribution by hand calculations

As changes were made to the AASHTO Specifications and local practice, the details were modified but, because of the large number of design combinations and time constraints, the modifications were not exactly accurate for all sections. This makes it difficult to verify these designs according to strict AASHTO requirements. Culverts constructed by these designs have proven adequate by virtue of their performance under traffic.

In spite of their somewhat antiquated design, no significant malfunctions have been observed in the many culverts constructed to these details. This was one reason for the reluctance by TxDOT to redesign according to the latest specifications.

Significant to advancement of culvert technology was acceptance of the precast box culvert. For Texas the industry began in Beaumont in the early 1970s with a fabricator making standard cast-in-place sections vertically in 8 ft. lengths. High-strength dry concrete was required because the inside form was a mandrel that was retracted as soon as concrete placement was complete. Outside forms were also removed soon.

Another fabricator, in Harlingen, also provided TxDOT with standard precast box culvert details before acceptance of an industry standard covered by ASTM C789.

AASHTO acceptance was slow because ASTM C789, rather than the usual material specification, actually gave the live load and fill height that each section could sustain. This usurpation of the bridge engineer's field of authority plus loading research in progress and questionable shear transfer across the joint in direct traffic situations, made it difficult for the industry to gain formal acceptance from AASHTO. Texas allowed the industry standard section prior to acceptance, but established allowable fill heights according to local practice.

ASTM C850 was published to cover direct traffic precast box culverts. Several TxDOT standard detail sheets were developed to allow the use of precast products.

Recent Changes

In the early 1980s, the specifications allowed culverts to be bid by the linear foot, instead of cubic yard. This allowed the contractor to decide whether to build precast or cast-in-place culverts. Virtually all culverts are now bid this way, as shown in the "Reinforced Concrete Box Culvert Usage" table.

Another advance in culvert design began in the early 1980s when the Federal Highway Administration insisted action be taken to protect errant motorists from plunging into the space at the end of cross drainage culverts or running into the headwalls of parallel drainage culverts. The solution involved installing pipe runners across the opening, which created hydraulic concerns because of negative effect on the ability of the culvert to carry storm water. Research was conducted at CTR (Report 301-1F) on the hydraulic aspects and at TTI (Report 280-1 and 280-2F) on the structural aspects. The structural results were fine-tuned and safety end treatment standard detail sheets were prepared.

In the late 1990s it became necessary to produce box culvert standard detail sheets in metric units, and the entire culvert series was redesigned, using load factor design and the current specifications. These metric box culvert standard detail sheets were then converted to English units in 2000. At that time significant changes were made to the wing wall and safety end treatment details.

Current Status

The current TxDOT box culvert standard detail sheets can account for almost any box culvert design need. Culvert standard detail sheets are available on the TxDOT web site for the following box culverts and appurtenances:

- Cast-in-place single boxes (30 ft. max. fill height)
- Cast-in-place multiple boxes (23 ft. max. fill height)
- Precast single boxes (12 ft. to 20 ft. max. fill height depending on span)
- Wing walls (straight, flared, and parallel)
- Safety end treatments

Design Recommendations

The TxDOT box culvert standard detail sheets will significantly reduce the need for special designs for culverts and end treatments. However, if a special design is warranted, some design parameters are as follows:

- Vertical earth pressure 120 pcf
- Lateral earth pressure 40 pcf
- Live load is a 16 kip wheel with impact per AASHTO
- Distribution of a wheel is a square of 1.7 x fill depth, for fills ≥ 2 ft.
- For fill heights less than 2 ft., load is considered a point load. Distribution and design per AASHTO slab design requirements.
- Two feet of surcharge and full lateral pressure used for corner moments
- Half lateral pressure used for positive moments
- Spans loaded according to influence lines for moments
- Class C concrete, f 'c = 3,600 psi; Grade 60 reinforcing steel, fy = 60,000 psi
- Slab thickness based on allowable shear

Additional information, including when and where to use culverts, can be found in the Roadway Design Manual. A discussion on the hydraulic requirements of culverts can be found in the Hydraulic Design Manual.

Reinforced Concrete Box Culvert Usage

Contract Bid Quantities				
Calendar Year	**Concrete for Culverts (C.Y.)**			**[4] Box Culvert (L.F.)**
	[1] Class A	**[2] Class C**	**[3] Class S**	
1966	148,000	10,000	—	—
1971	730	130,000	—	—
1976	39,000	3,000	—	—
1981	23,000	2,000	—	—
1983	7,500	2,000	100	49,000
1986	19,000	300	250	246,700
1988	11,000	2,000	100	226,700
1997	210	548	56	187,612

1. Five sack 3,000 psi
2. Six sack 3,600 psi
3. Special deck concrete for direct traffic culverts
4. Alternate precast or cast-in-place; linear feet of single barrel