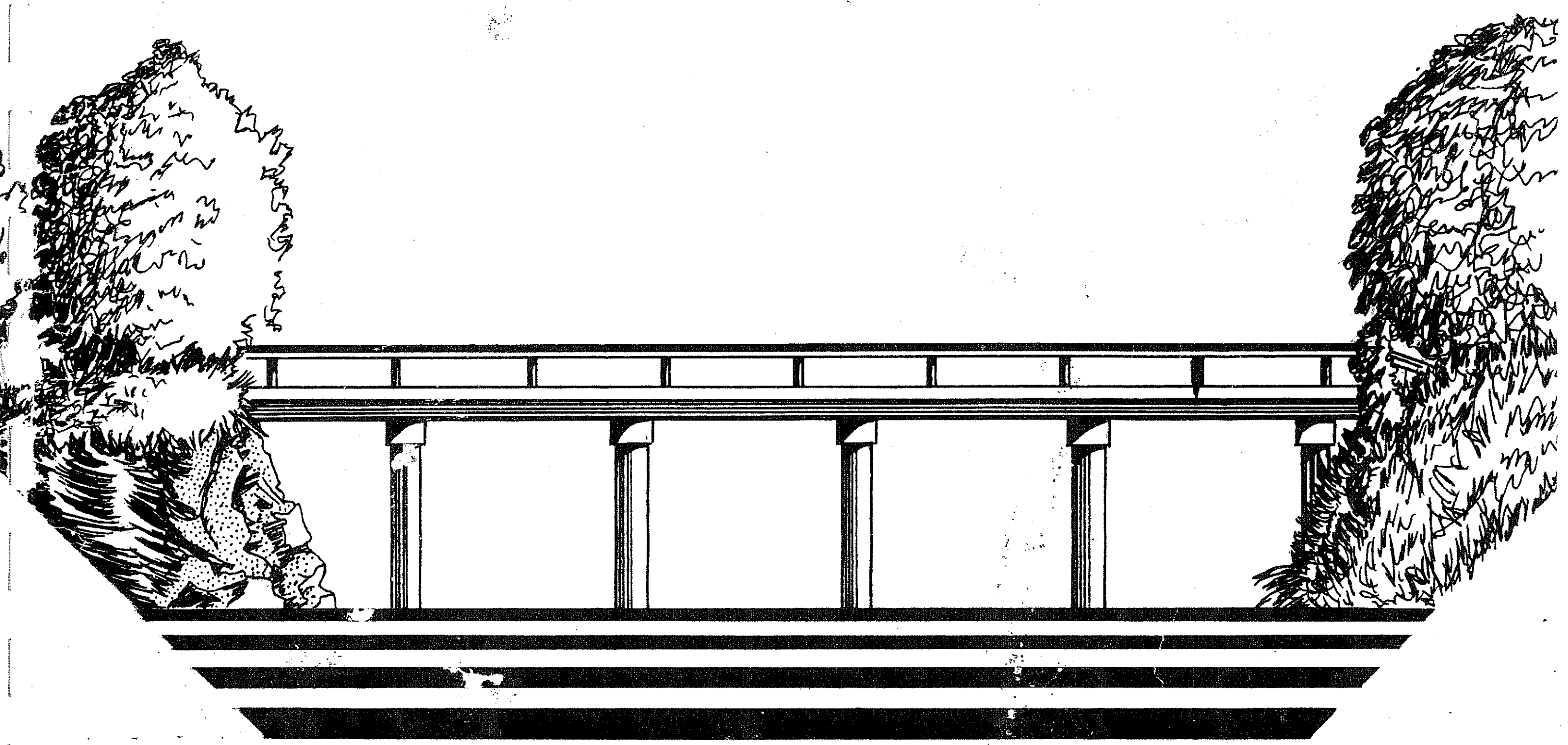
FLORIDA
BRIDGE LOAD RATING
MANUAL



BUREAU OF MAINTENANCE

Structure Maintenance Operations Section

SEPTEMBER 1982

FLORIDA BRIDGE LOAD RATING MANUAL

for the

Structural Rating of Superstructures

Developed by

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Forward

The Florida Bridge Load Rating Manual presents methods to determine the safe load ratings of four basic bridge superstructures. These methods reflect a combination of structural theory and compliance with the latest codes.

The contents of this manual are arranged with a descriptive text concurrently with detailed examples. The manual serves as a self-explanatory reference intended for the use of technical personnel who have a general knowledge of engineering principles. Although the examples within this manual cover a large portion of common bridge examples, this manual by no means is intended to serve as an all-encompassing text. The analysis of a deteriorated bridge combines the art of judgment with the science of structural theory. Often circumstances arise where the guidance and expertise of a registered structural engineer is required.

Extensive effort has been devoted to this manual to assure compliance with the design methods within the AASHTO codes. However, since design and analysis are not always similar, there are circumstances within this manual that require judgmental decisions. Consequently, each individual agency may establish its own standard format using this manuas as its basis.

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KISINGER CAMPO AND ASSOCIATES

FLORIDA BRIDGE LOAD RATING PROGRAM

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INTRODUCTION

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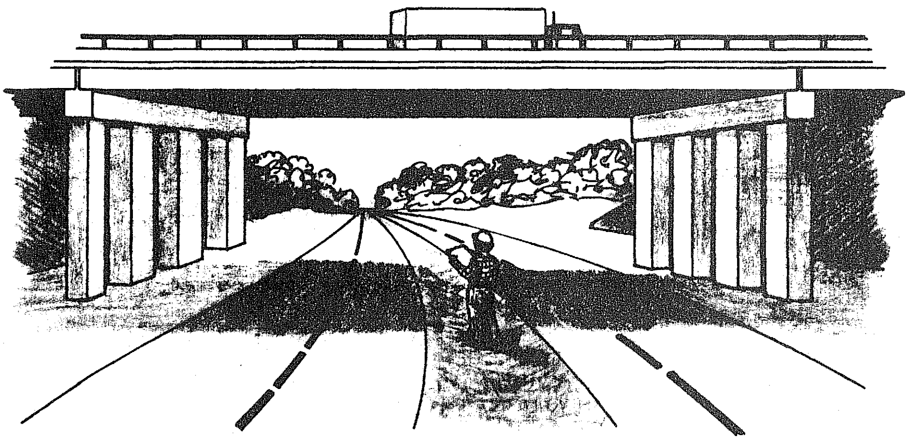


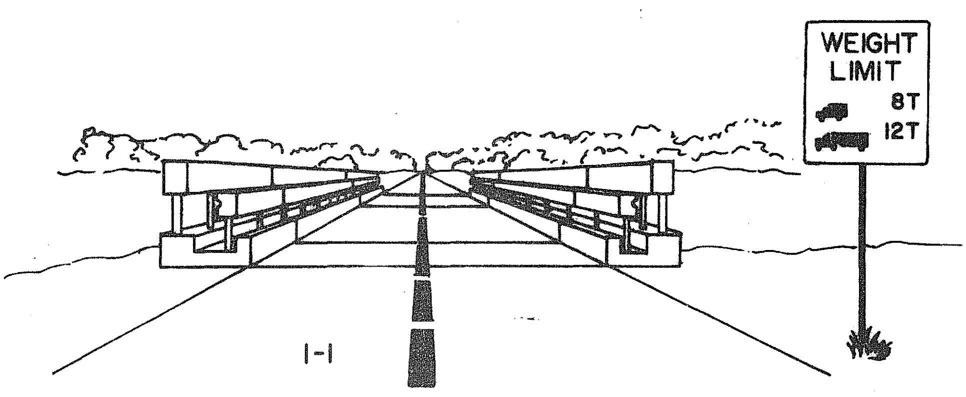
INSPECTION

LOAD RATING

EVALUATION

AND

RECOMMENDATION



INTRODUCTION

SECTION 1.1 - OVERVIEW

The primary function of the Bridge Inspection Program is to maintain highway bridges in a condition to provide safe and uninterrupted traffic flow. A secondary function is to provide the input data for the coordination of well programmed repairs and preventive maintenance. These functions are accomplished through the execution of three distinct tasks: the bridge inspection, where the physical condition of the structure is determined and recorded in the Bridge Inspection Report; the bridge load rating, where the structural condition of the bridge is assessed through the computation of maximum safe loads; and the evaluation and recommendation stage, where a determination of the required maintenance, load restrictions, or replacement (or possibly no action) is made.

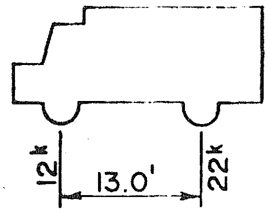
The Bridge Inspection program attained national importance primarily as the result of the collapse of the Silver Bridge linking Point Pleasant, West Virginia with Kanauga, Ohio. On December 15, 1967 the main span of the 40-year-old, 1,750-ft suspension bridge plunged into the Ohio River during rush hour traffic, killing 46 people. It was eventually determined that a crack in a crucial eye-bar had apparently developed over the 40-year life and finally reached the point where it became critical as a result of stress and fatigue and triggered the collapse. As a result of this disaster, the Federal Aid Highway Act was passed by the United States Congress in 1968, requiring the Secretary of Transportation to establish a National Bridge Inspection Standard, including the establishment of training programs. Also, federal regulations were established that requires that all bridge structures located on federal routes be inspected at least once every two years by a registered Professional Engineer or personnel having completed specialized training in the area of maintenance inspection of bridges.

SECTION 1.2 - PURPOSE

The purpose of this manual is to provide a means of accomplishing the second task of the Bridge Inspection Program: assessing the structural condition of the bridge through the computation of maximum safe loads. It should be noted that this manual provides a method for assessing the structural condition of the SUPERSTRUCTURE ONLY. No discussion or rating procedures are provided concerning the substructure. Therefore, this manual provides only for a portion of the computations necessary for the load rating calculations, the rating of the superstructure. The other portion of the load rating calculations, the rating of the substructure, is to be accomplished by a qualified engineer.

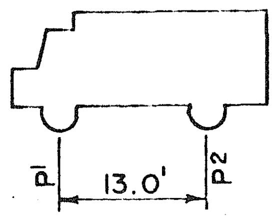
The safe loads to be completed will be tabulated in the form of rating values where the numerical rating value for a particular load case represents the maximum Gross Vehicle Weight (GVW) for the axle spacing of the particular load case, and is proportioned so that the

EXAMPLE OF RATING VALUE INTERPRETATION:



SU2 Loading Case
(GVW - 34.0 kips)

For a rating of 30 kips:



SU2 Loading Case
with rating = 30 kips
(so GVW = 30 kips)

Front axle (P_1):

$$\frac{P_1}{12} = \frac{30}{34}$$

$P_1 = 30/34\ (12) = 10.59$ kips

Rear axle (P_2):

$$\frac{P_2}{22} = \frac{30}{34}$$

$P_2 = 30/34(22) = 19.41$ kips

axle loads are in similar ratios with one another (See example on previous page). In accordance with the AASHTO "Manual for Maintenance Inspection of Bridges"[1] (called AASHTO Bridge Manual in this Manual), each highway bridge shall be rated at two load levels. "At the first or upper load level, the capacity rating will be referred to as the Operating Rating. The Operating Rating will result in the absolute maximum permissible load level to which the structure may be subjected..... At the second or lower load level, the capacity rating will be referred to as the Inventory Rating. The Inventory Rating will result in a load level which can safely utilize an existing structure over an indefinite period of time."[2] Therefore, the final outcome of this manual will be the Inventory and Operating Rating for each loading case. Loading cases will be discussed in detail in a later section of this manual.

This manual presents a method for computing the rating values for four basic superstructure types which represent the bridge types most commonly found under the local maintenance jurisdiction of Counties and Cities. The four basic superstructure types are as follows

1. Timber Deck and Stringer
2. Reinforced Concrete Slab
3. Reinforced Concrete T-Beam
4. Prestressed Concrete Channel and Slab System

Each of these superstructure types are described in Section 2.1.

SECTION 1.3 - STEPS FOR RATING PROCEDURE:

STEP 1: OBTAIN DATA

Obtain a copy of the Bridge Inspection Report. Based on this, a determination of whether or not the bridge needs to be structurally rated can be made. If a rating is necessary, the Bridge Inspection Report will supply much of the required information for the analysis. If possible, secure a copy of the construction plans for the bridge (or as built plans, if available). This is particularly important for concrete bridges since it is necessary to accurately know the size and location of the reinforcing steel in order to arrive at an accurate rating for the structure. In such cases where the information from the Bridge Inspection Report conflicts with the information shown on the construction plans, a field investigation will be needed to determine which data is correct.

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1 Manual for Maintenance Inspection of Bridges, 1978, prepared by the Highway Subcommittee on Bridges and Structures, published by the American Association of State Highway and Transportation Officials, Washington, D.C.

2 Manual for Maintenance Inspection of Bridges, 1978, p. 24.

Obtain the current legal load cases. These may be secured from:

Mr. Chuck Bradshaw
Chief Bureau of Weights
605 Suwannee Street
Mail Station #99
Tallahassee, Florida 32301

STEP 2: DETERMINE THE TYPE OF SUPERSTRUCTURE

Section 2.1 describes the four types of superstructures that are included in this manual. The superstructure to be rated should be compared with those described in Section 2.1 and the proper superstructure type selected. In the event that the superstructure type differs from one of the four types analyzed here, it may either be analyzed by computer (the four superstructure types included herein and several additional configurations have been programmed by FDOT) or analyzed longhand by a qualified engineer. The Florida Department of Transportation may be contacted concerning availability of computer programs.

STEP 3: COMPILE EXISTING DATA

Select from Chapter 2 of this manual the appropriate set of Inspection Data Sheets (Sections 2.2, 2.3, 2.4 or 2.5) and obtain a copy from Appendix C. Complete the data sheets in accordance with the directions presented therein. All spaces on the data sheets should be completed before proceeding with the rating calculations. When there is missing or questionable data a field investigation may be in order.

Accuracy is very important at this stage. It is essential that the existing data be recorded accurately and completely in order for the rating calculations to yield realistic values. All deteriorated areas should be clearly noted on the Inspection Data Sheets. Where spaces or provisions have not been provided to record deteriorated areas, the deterioration should be noted at the end of the Inspection Data Sheets. Remember the case of the Silver Bridge where a small crack went unnoticed and eventually led to the collapse of the structure and the loss of many lives.

STEP 4: CALCULATE LIVE LOAD SHEARS AND MOMENTS

Live load shears and moments are the forces in the bridge members that result from the loadings of the rating trucks. The calculation of the live load shears and moments is independent of the type of superstructure, which is why their calculation has been included as a separate section.

Chapter 3 of this manual describes and illustrates a method for computing the live load shears and moments. The Engineering Overview (Sec. 3.1) should be reviewed and then

Section 3.2 completed to compute the live load moments and Section 3.3 completed to compute the live load shears. Sections 3.2 and 3.3 follow the format of providing a discussion and directions, and then the equations to be used. Since no forms are given, the equations will need to be evaluated on separate sheets of paper. These calculations should be neat, clear and not overcrowded. Standard 8-1/2" x 11" paper should be used so the calculations may be neatly bound with the forms once the rating procedure is complete.

Section 3.4 is an example of the computations required for Sections 3.2 and 3.3. When a question arises review of the examples will often answer the question.

STEP 5: CALCULATE RATING VALUES

Calculations of the Rating Values is presented in Chapters 4, 5, 6 and 7. Each section describes the necessary calculation of ratings for one of the four superstructure types. The section should be chosen for the superstructure type that is being rated, and the other three disregarded.

Before beginning the calculation of the rating values, a copy of all sheets headed "RATING OF ..." should be secured from Appendix C. The first Section (Sec. 4.1, 5.1, 6.1 and 7.1) gives a general overview of the rating calculations for the particular superstructure type and presents the conditions and assumptions that affect the analysis.

Since the specific procedure to be followed for the rating calculations differ for the four superstructure types, it will not be discussed here. However, the second section of each Chapter (Sec. 4.2, 5.2, 6.2 and 7.2) describes the specific procedure to be followed for the rating calculations for each superstructure type.

STEP 6: SUMMARY OF RATINGS

As the Rating Values are calculated in step 5, specific instructions are given as to how the values should be entered in the Summary of Ratings table. The ratings are summarized by completing Chapter 8 of this manual.

The Inventory Rating represents the load levels which can safely utilize the bridge for an indefinite period of time. Whenever the GVW of a legal load case exceeds the Inventory Rating, posting the structure for maximum load or the implementation of remedial repairs must be considered.

The Operating Rating represents the load levels that stress the bridge to its highest level consistent with a reasonable degree of safety and represents the maximum permissible load level to which the structure may be subjected.

The specifications for rating bridge structures allows bridges to be utilized by an occasional vehicle (specifically a vehicle traveling under a special overweight permit) that produces stresses greater than the values obtained for the Inventory Rating. However, special permits will only be issued if such loads are distributed so as not to exceed the structural capacity determined by the Operating Rating.

Based on a comparison of the actual vehicle weights with the Inventory and Operating Ratings, the Engineer will recommend the posting, repair, replacement, or any combination thereof for the bridge.

Posting of bridge structures for a given load is often confused with the ultimate collapse load of the bridge. This is not the intent of the posted load. The posted load is generally that load which would be consistent with sound engineering practice and would insure the continued integrity of a bridge structure over the service life expected of it.

Among the factors to consider when establishing the extent of and priority for repairs to a structure are:

1) Safety - Any condition that is considered unsafe from a structural or user viewpoint should receive immediate attention.

2) Density of Traffic - The greater volume of traffic served by the bridge, the more important it is to make needed repairs in a timely manner.

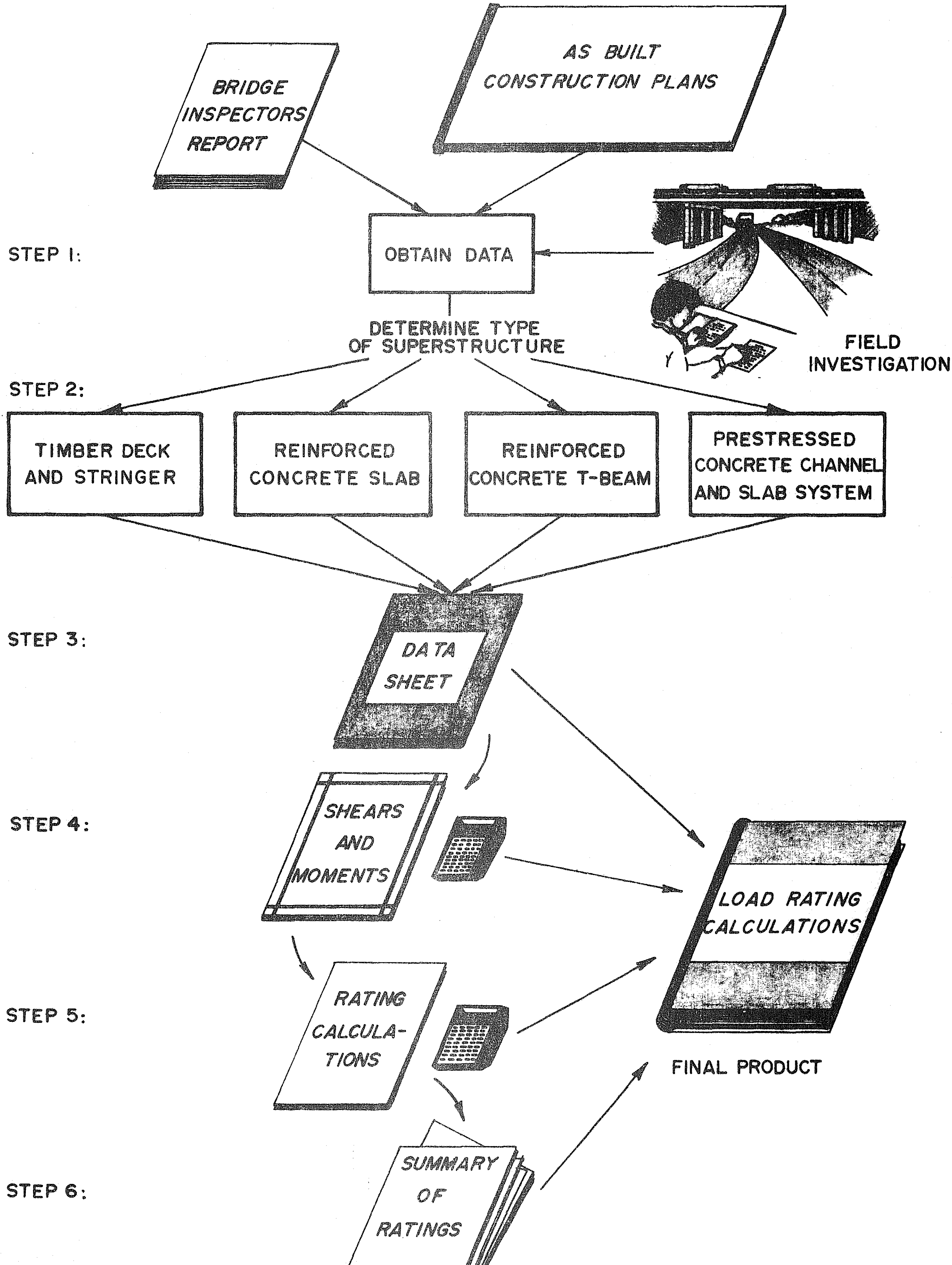
3) Alternate Bridges - The lack of convenient alternate bridges increases the importance of the repair to the structure.

4) Weather - Defects that could be aggravated by adverse weather should be corrected prior to the onset of the extreme weather conditions.

5) Replacement - Any decision for making bridge repairs should consider whether the bridge in question is scheduled for replacement and when closure or restricted access, if any, is to be implemented. A bridge scheduled for replacement in the near future may warrant only temporary repair or, possibly, none at all depending on the nature of the repairs.

The Engineer's recommendation of replacement of a bridge structure must be based on the practicality and feasibility of making the needed repairs, and of an economic comparison of the cost of the necessary repairs with the cost of the replacement bridge. This comparison must consider the expected life of the bridge in question, supposing the required repairs are made, and costs of subsequent repairs made during the remaining life of the bridge.

STEPS IN RATING PROCEDURE:



SECTION 1-4 - A FINAL NOTE:

When performing the calculations in accordance with this Manual, clarity and accuracy are of great importance.

Do not crowd your calculations. Paper is cheap compared to the man-hours spent in trying to interpret results, and crowded work usually leads to errors. Where insufficient space has been given to perform a calculation, add additional sheets as required and keep the computations neat.

Accuracy is a must. Accuracy does not mean carrying work to an excessive number of decimal places. For all calculations in this manual, two or three decimal places are sufficient. Accuracy does mean watching the decimal point, making sure the proper values are used in the proper places in the equations and the value has the proper units, and making sure the directions given are followed. Carelessness in any one of these points may lead to significant errors. If questions arise, a qualified engineer should be consulted. "Assumptions" should not be made about an area in which you are not thoroughly familiar.

With the advent of modern electronic calculators has come the ability to process a great deal of computations quite rapidly. The amount of computations involved in the rating process is sizable and it is common to fall into the habit of calculating each equation once, getting the result and going on to the next equation. Instead take advantage of the calculator's ability to perform calculations rapidly by solving each equation twice and making sure the results are the same each time. This simple check will save a great deal of time later, as a simple error in calculations will often lead to results that are obviously erroneous. It is very time consuming to go back through all the computations looking for an error.

As a final check, compare your results to other structures with similar superstructure types and member layouts and see if they look reasonable. Many times calculations have been turned in with results that are obviously in error because the person making the computations got so involved in the mechanics of the calculations that they did not take time to see if their conclusions looked reasonable.

REMEMBER:

- Keep calculations neat.
- Watch the decimal point.
- Make sure the proper values are used in the proper places.
- Make sure the values have the proper units as indicated in the equations.
- Perform all calculations twice to check the math.
- See if final results look reasonable.

CHAPTER 2

TABULATION OF EXISTING DATA

2.1 Overview

2.2 Timber Deck and Stringer Superstructure Inspection Data Sheet

2.3 Reinforced Concrete Slab Superstructure Inspection Data Sheet

2.4 Reinforced Concrete T-Beam Superstructure Inspection Data Sheet

2.5 Prestressed Concrete Channel and Slab System Inspection Data Sheet

SECTION 2.1 - OVERVIEW

The purpose of this section is twofold: to determine if the superstructure under evaluation fits one of the four types presented in this manual, and to compile the applicable data from the inspection report.

Methods for evaluating four types of superstructures are presented in this manual: Timber Deck and Stringer, Reinforced Concrete Slab, Reinforced Concrete T-Beam and Prestressed Concrete Channel and Slab System. The superstructure to be evaluated should be compared with the descriptions given below to determine if the type of construction is the same. Where minor differences exist, a qualified engineer should be consulted as to the applicability of this manual. Where significant differences in construction between the superstructure under consideration and those presented in this manual exist (such as hybrid sections where a timber deck is supported by steel stringers), a qualified engineer will need to perform the rating calculations.

TIMBER DECK AND STRINGER SUPERSTRUCTURE: Included under this superstructure classification are bridges comprised of a timber deck with planks placed perpendicular to the flow of traffic and supported by timber stringers placed parallel to the flow of traffic. The flooring may have a concrete or asphalt wearing surface. Sidewalks may be included on either side of the bridge. Glued and laminated timber members are not covered. A general plan and section for this type of superstructure is shown on Figure 2-1 and 2-2.

REINFORCED CONCRETE SLAB SUPERSTRUCTURE: Included under this superstructure classificaiton are bridges comprised of conventionally reinforced slab spans where the main reinforcement is parallel to the direction of traffic. Prestressed or post-tensioned type construction is not covered. The slab may have an asphalt wearing surface. A general plan and section for this type of superstructure is shown on Figure 2-3 and 2-4.

REINFORCED CONCRETE T-BEAM SUPERSTRUCTURE: Included under this superstructure classification are bridges comprised of monolithically cast, conventionally reinforced T-section superstructures and a conventionally reinforced concrete slab supported by conventionally reinforced rectangular beams where sufficient bond between the two members has been established to assure monolithic action. This bond may be supplied by stirrups extending from the rectangular beam section to the slab. Where significant cracking has developed between the slab and the rectangular beam portion, monolithic action is questionable and the analysis presented in this manual does not apply. In such cases, the ratings should be computed by a qualified engineer.

Members must be conventionally reinforced as pretensioned and post-tensioned construction is not covered. The slab may be covered with an asphalt wearing surface. A general plan and section for this type of superstructure is shown on Figures 2-5 and 2-6.

PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM SUPERSTRUCTURE: Included under this superstructure classification are bridges comprised of precast-prestressed concrete channel sections, placed side-by-side and topped with a conventionally reinforced pour-in-place slab. This slab may have an additional asphalt wearing surface. A general plan and section for this type superstructure is shown on Figures 2-7 and 2-8.

SECTION 2.2 - TIMBER DECK AND STRINGER SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE:

1. These sheets should be completed in their entirety before continuing with the rating computations.
2. Sketches of the bridge superstructure under consideration should be made and dimensions, spacings, and direction of inspection should be shown thereon. These sketches should include a section and plan similar to Figures 2-1 and 2-2.

By: ______________________________ Date: __________

Bridge Name: ______________________________

Bridge No.: __________ Road No.: __________ County: __________

Bridge Length = ____________ ft. Number of Spans = __________

Span Length ($\mathbb{C}_L$ support to $\mathbb{C}_L$) = ________ ft. (if different list each)

Overall Bridge Width (out-to-out) = ______________ ft.

Roadway Width (curb to curb) = ______________ ft.

Sidewalks:

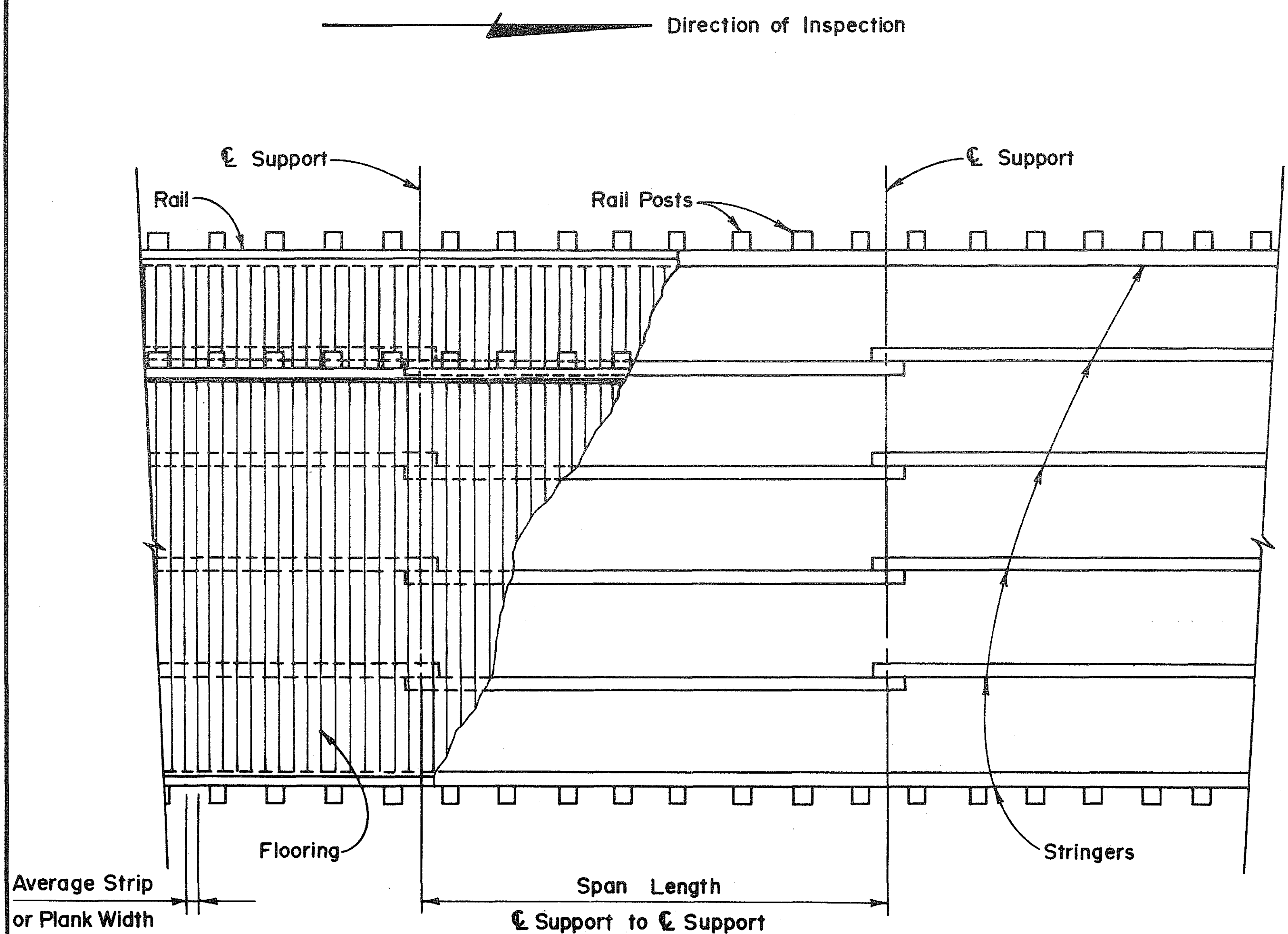
LEFT SIDE (looking in direction of inspection):

Width = ____________ ft.

Rail Size = _________in. x ________ in.

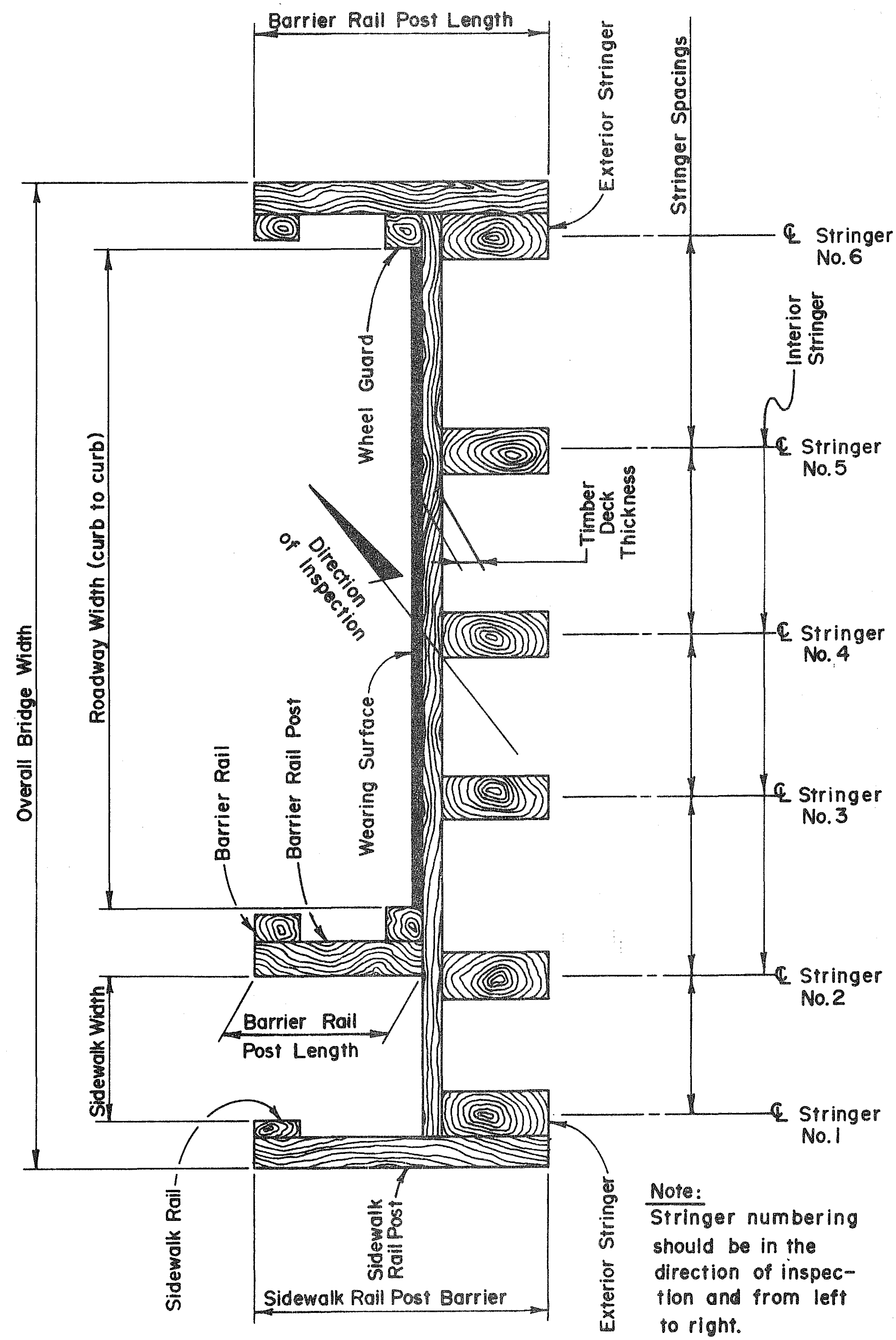
Rail Post Size & Length = ________ in. x _______ in. x ________ ft.

Average Post Spacing = ___________ ft.



GENERAL PLAN - TIMBER DECK AND STRINGER

Figure 2-1



GENERAL SECTION - TIMBER DECK AND STRINGER

Figure 2-2

RIGHT SIDE (looking in direction of inspection):

Width = ____________ ft.

Rail Size = ________in. x _______ in.

Rail Post Size & Length = _______ in. x ______ in. x _______ ft.

Average Post Spacing = __________ ft.

Barrier Rails: Rail Size = _______ in. x ______ in.

Rail Post Size & Length = ______ in. x ______ in. x ______ ft.

Average Post Spacing = __________ ft.

Wheel Guard: Width = ___________ in.

Height = ___________ in.

Wearing Surface:

asphalt; thickness = ___________ in.

concrete; thickness = ___________ in.

Deck:

Type; Strip (tongue & groove or dowled) or Plank

Continuous over three or more stringers/or simple span between two stringers

thickness = _______ in. (average thickness not considering deterioration)

thickness = _______ in. (average thickness taking into account deterioration.)

Width of Strip or Plank = _________ in.

SECTION 2.2 - TIMBER DECK AND STRINGER SUPERSTRUCTURE INSPECTION DATA SHEET (CONT.)

STRINGER DATA:

Stringer Number	Stringer Cross-Sectional Dimensions (Height and Width) (inches)	Spacing Between Adjacent Stringers (ft.)

STRINGER DATA NOTES:

1. Deterioration of the stringer should be accounted for by using an average height x width of the sound material. If interior deterioration has occurred (the stringer is a "shell") the average exterior height x width along with the approximate interior height x width of deterioration and dimensions from center of deteriorated area to bottom of stringer should be recorded under "stringer cross-sectional dimensions".

2. The location of deteriorated sections as measured from the beginning of stringer (as defined by the direction of inspection) should be noted.

3. It should be specifically noted if a loss of stringer section has occurred near the end of the stringer. If no specific comments were noted in the Bridge Inspectors Report, it should be verified that the stringer has suffered no loss of section due to deterioration or crushing, as this condition will be significant in the horizontal shear rating of the stringer.

Material Strengths:

Allowable stress in bending (f_b) as determined by the Bridge Engineer* = __________ psi.

Allowable stress in shear (f_v) as determined by the Bridge Engineer* = __________ psi.

Allowable bending stress at Inventory Rating level (f_{bi})

$= f_b = \boxed{}$ psi.

Allowable bending stress at Operating Rating level (f_{bo})

$= 1.33\ f_b = \boxed{}$ psi.

Allowable shear stress at Inventory Rating level (f_{vi})

$= f_v = \boxed{}$ psi

Allowable shear stress at Operating Rating level (f_{vo})

$= 1.33\ f_v = \boxed{}$ psi.

---

*Standard Specifications for Highway Bridges, Twelfth Edition, 1977; Interim Specifications Bridges 1978, 1979 and 1980; Section 1.10.1 and Table 1.10.1A will be used in determining the allowable stresses for timber members when, in the judgement of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the allowable stresses shall be fixed by a Registered Professional Engineer.

SECTION 2.3 - REINFORCED CONCRETE SLAB SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE:

1. These sheets should be completed in their entirety before continuing with the rating computations.
2. Sketches of the bridge superstructure under consideration should be made and dimensions, spacings, and direction of inspection should be shown thereon. These sketches should include a section and plan similar to Figures 2-3 and 2-4.

By: ______________________ Date: ____________ ft.

Bridge Name: ______________________

Bridge No.: __________ Road No.: __________ County: __________

Bridge Length: __________ ft. Number of Spans: __________

Span Length ($\mathbb{C}_L$ Support to $\mathbb{C}_L$ Support) = __________ (if different, list each)

Overall Bridge Width = __________ ft.

Roadway Width (curb to curb) = __________ ft.

Sidewalk: Width = ________ ft.
Rail Size = ________ in. x ________ in.
Rail Post Size & Length = ________ in. x ________ in. x ________ ft.
Average Post Spacing = ________ ft.

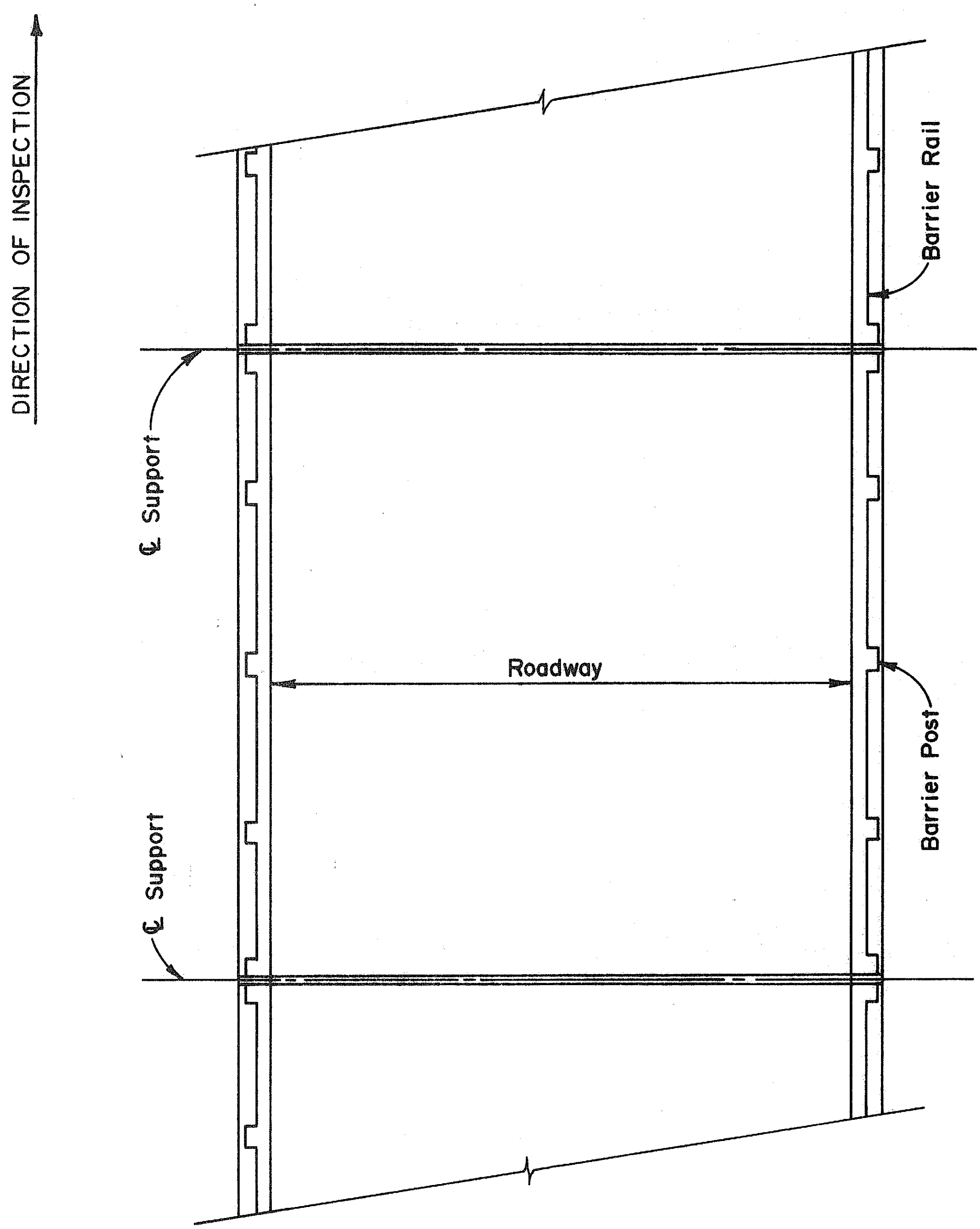
Barrier Rails: Rail Size = ________ in. x ________ in.
Rail Post Size & Length = ________ in. x ________ in. x ________ ft.
Average Post Spacing = ________ ft.

Wearing Surface:

Thickness = ________ in.

Concrete Slab:

Thickness = ________ in.
Longitudinal Rein. (Bottom) Size ______ Avg. Spacing = ________ in.
Longitudinal Rein. (Top) Size ______ Avg. Spacing = ________ in.
Transverse Rein. (Bottom) Size ______ Avg. Spacing = ________ in
Transverse Rein. (Top) Size ______ Avg. Spacing = ________ in.
d = ________ in. (distance from top of slab to centerline of bottom longitudinal reinforcing).



GENERAL PLAN - REINFORCED CONCRETE SLAB

Figure 2-3

GENERAL SECTION - REINFORCED CONCRETE SLAB

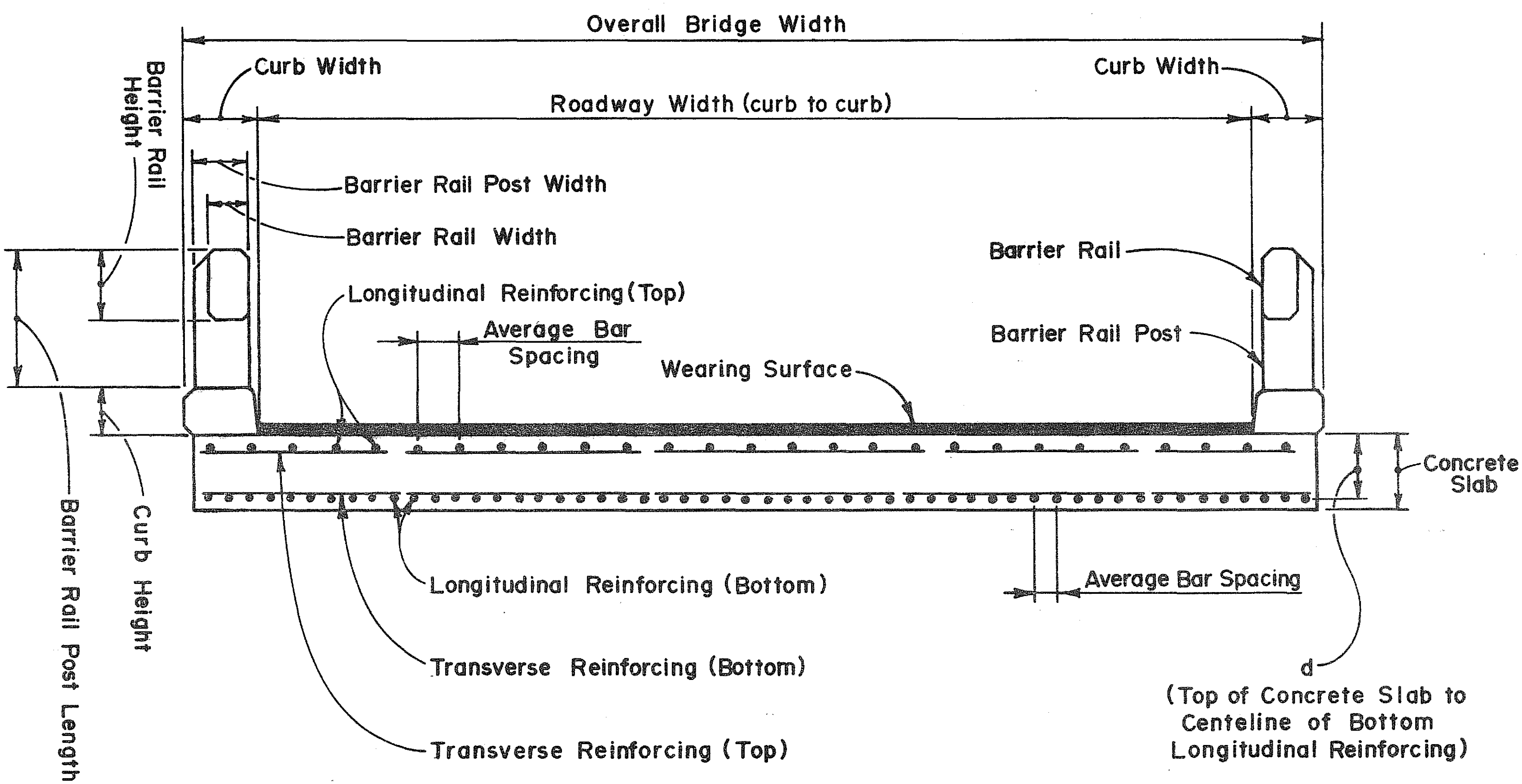


Figure 2-4

Material Strengths:

f'c (compressive strength of concrete) = __________ psi.
(from contract plans*)

fy (yield strength of reinforcing steel) = __________ psi.
(from contract plans**)

These unit stresses shall be used when, in the judgement of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the maximum yield stresses and compressive strength of concrete shall be fixed by the Registered Professional Engineer, based on the field investigation, and shall be substituted for the previously given stresses. These stresses shall in no case be greater than the previously given stresses.

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* When the compressive strength of the concrete is unknown, f'c may be taken as 3,000 psi., subject to the above paragraph.

**When the Grade is given, or the steel is unknown, use the following yield stresses for reinforcing steel:

Reinforcing Steel	Yield Point (fy) psi
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Intermediate Grade and unknown after 1954 (GRADE 40)	40,000
Hard grade (GRADE 50)	50,000
(GRADE 60)	60,000

SECTION 2.4 - REINFORCED CONCRETE T-BEAM SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE:

1. These sheets should be completed in their entirety before continuing with the rating computations.
2. Sketches of the bridge superstructure under consideration should be made and dimensions, spacings, and direction of inspection should be shown thereon. These sketches should include a section and plan similar to Figures 2-5 and 2-6.

By: ______________________________ Date: ______________

Bridge Name: ______________________________

Bridge No.: __________ Road No.: ________ County: ______________

Bridge Length = ____________ ft. Number of Spans = ______________

Span length ($\mathbb{C}_L$ to $\mathbb{C}_L$ of support = ____________ft.
(if different, list for each)

Overall Bridge Width = ________________ ft.

Bridge Width (curb to curb) = ________________ ft.

Sidewalk: Width = ____________ ft.
Rail Size = ________ in. x ________ in.
Rail Post Size & Length = ________ in. x ________ in. x ______ ft.
Average Post Spacing = ______________ ft.

Barrier Rails: Rail Size = ________ in. x ________ in.
Rail Post Size & Length = ________ in. x ________ in. x ________ ft.
Average Post Spacing = ____________ ft.

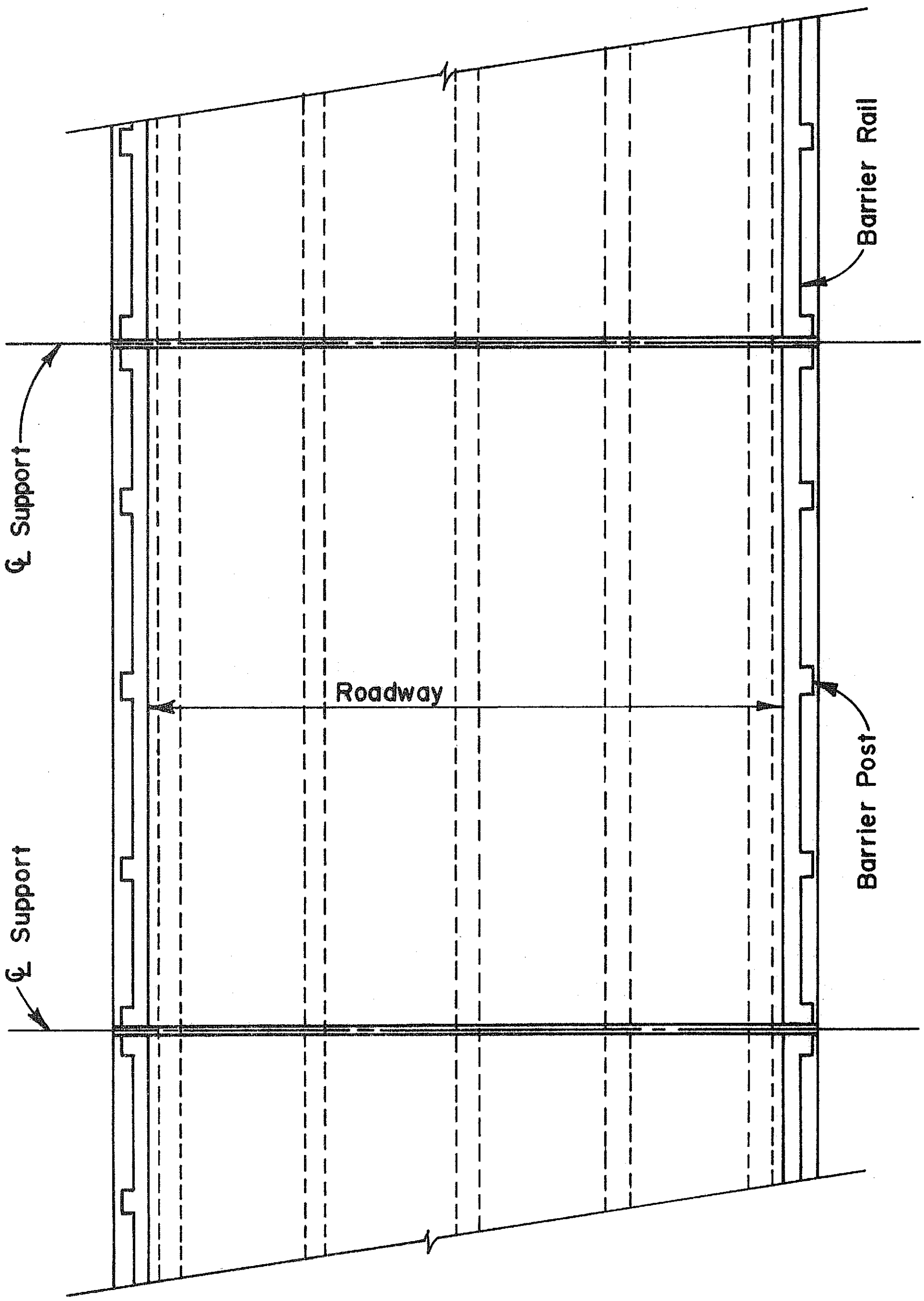
Wheel Guard: width = ________ in.
height = ________ in.

Asphalt Wearing Surface:

Thickness = ________ in.

Concrete Slab:

Thickness = __________ in.
Longitudinal Rein. (Bottom) Size ______ Avg. Spacing = ________ in.
Longitudinal Rein. (Top) Size ______ Avg. Spacing = ________ in.
Transverse Rein. (Bottom) Size ______ Avg. Spacing = ________ in.
Transverse Rein. (Top) Size ________ Avg. Spacing = ________ in.
d = ________ in. (distance from top of slab to centerline of longitudinal reinforcing).



GENERAL PLAN - REINFORCED CONCRETE T-BEAM

Figure 2-5

GENERAL SECTION-REINFORCED CONCRETE T-BEAM

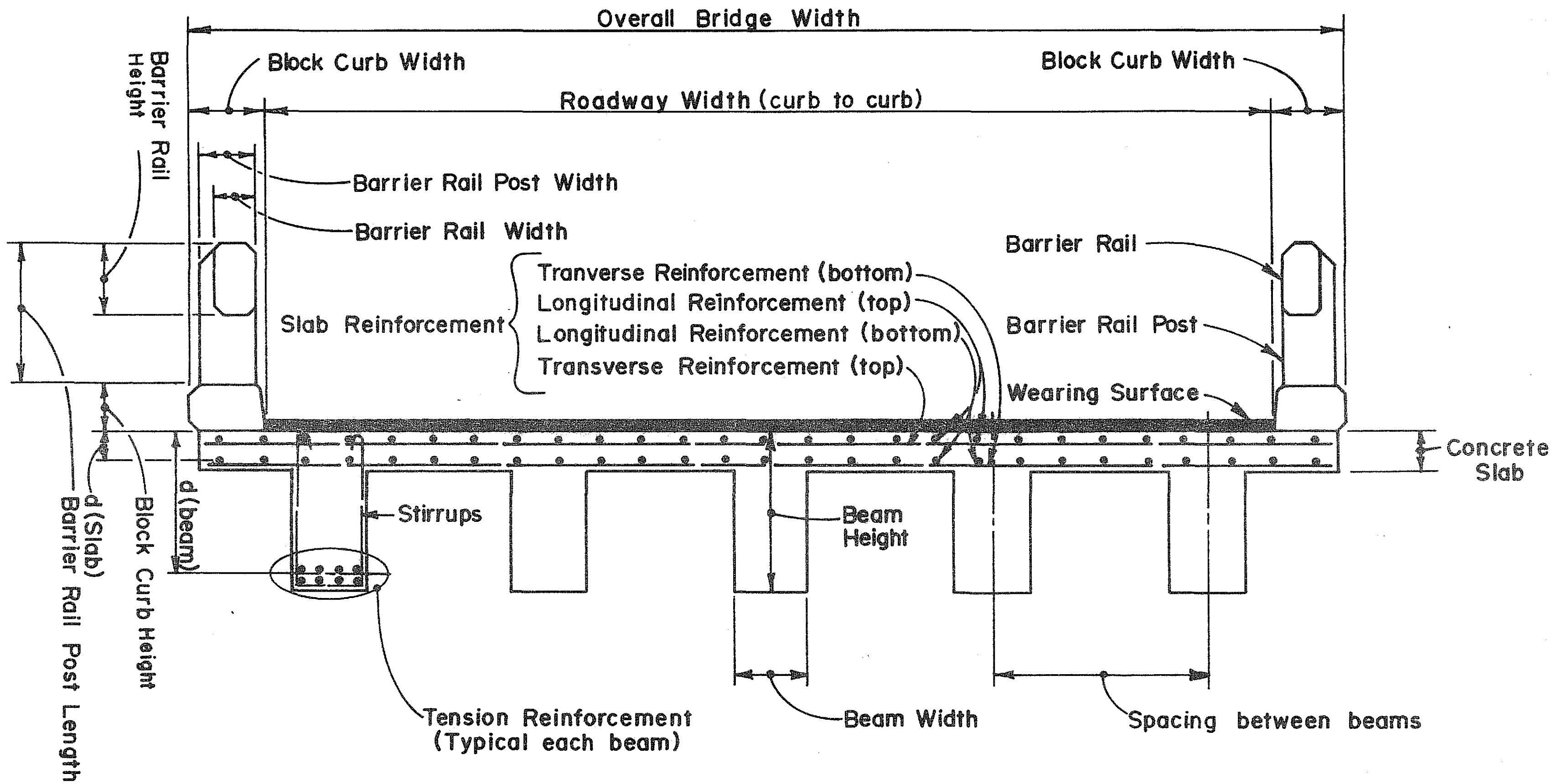


Figure 2-6

Concrete Beam:

Height = __________ in.
Width = __________ in.
Spacing Between Beams __________ ft.
Tension Reinforcement (reinforcement in bottom of beam)

size __________
number of bars = __________
d (distance from centroid of reinforcement to top of concrete slab) = __________ in.

Shear reinforcement in outer quarters of T-beam.

stirrup size = __________
stirrup spacing = __________ in.

Material Strengths:

f'c (compressive strength of concrete) = __________ psi.
(from contract plans*)

fy (yield strength of reinforcing steel) = __________ psi.
(from contract plans**)

These unit stresses shall be used when, in the judgement of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the maximum yield stresses and compressive strength of concrete shall be fixed by the Registered Professional Engineer, based on the field investigation, and shall be substituted for the previously given stresses.

---

* When the compressive strength of the concrete is unknown, f'c may be taken as 3,000 psi, subject to the above paragraph.

** When the Grade is given, or the steel is unknown, use the following yield stresses for reinforcing steel:

Reinforcing Steel	Yield Point (fy) psi
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Intermediate Grade and Unknown after 1954 (GRADE 40)	40,000
Hard grade (GRADE 50)	50,000
(GRADE 60)	60,000

SECTION 2.5 - PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM
Inspection Data Sheet

NOTE:

1. These sheets should be completed in their entirety before continuing with the rating computations.
2. Sketches of the bridge superstructure under consideration should be made and dimensions, spacings, and direction of inspection should beshown thereon. These sketches should include a section and plan similar to Figures 2-7 and 2-8.

By: ______________________________ Date: ______________

Bridge Name: ______________________________________

Bridge No.: __________ Road No.: _______ County: ______________

Bridge Length = ___________ ft. Number of Spans = ______________

Span length ($\mathbb{C}_L$ to $\mathbb{C}_L$ of support = __________ft.
(if different, list for each)

Overall Bridge Width = ________________ ft.

Bridge Width (curb to curb) = _________________ ft.

Sidewalk: Width = ___________ ft.
Rail Size = ________ in. x _______ in.
Rail Post Size & Length = _______ in. x ______ in. x _____ ft.
Average Post Spacing = _____________ ft.

Barrier Rails: Rail Size = _______ in. x ______ in.
Rail Post Size & Length = ______ in. x ______ in. x ______ ft.
Average Post Spacing = ___________ ft.

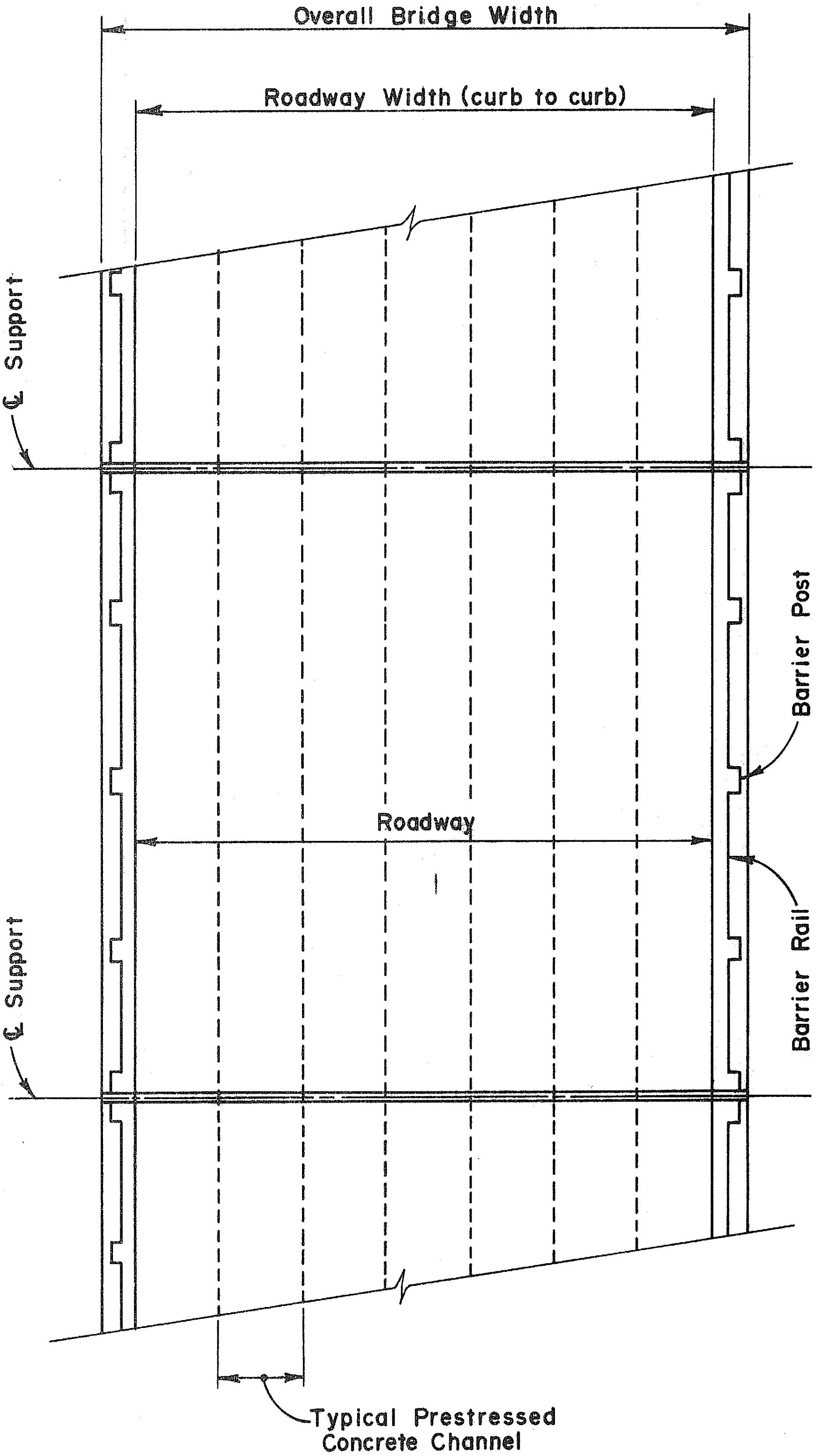
Curb Block: height x width = _______ in. x ______ in.

Wearing Surface: asphalt; thickness = ________ in.
concrete; thickness = ________ in.(not deck thickness)

Cast in place concrete deck surface: thickness= ________ in.

transverse reinf. = #_______ at _______ in. spacing.
longitudinal reinf. = #_______ at ________ in. spacing.

or, WWF = __________



GENERAL PLAN - CHANNEL AND SLAB SYSTEM

Figure 2 - 7

GENERAL SECTION-CHANNEL AND SLAB SYSTEM

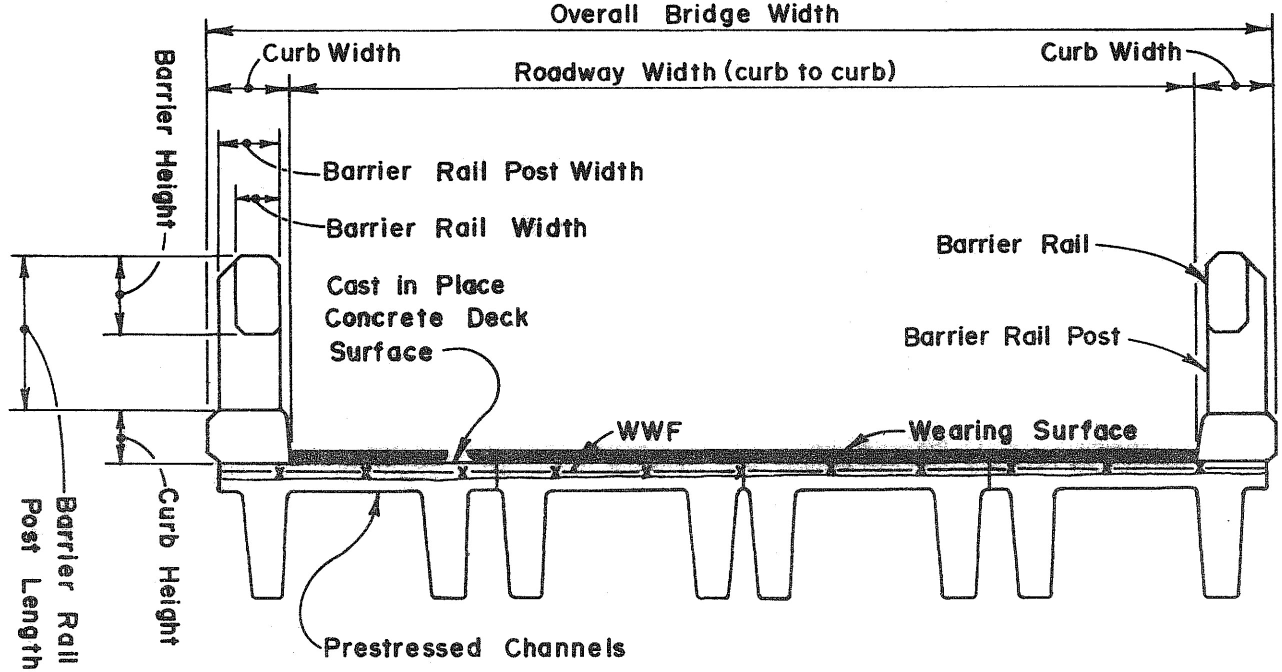
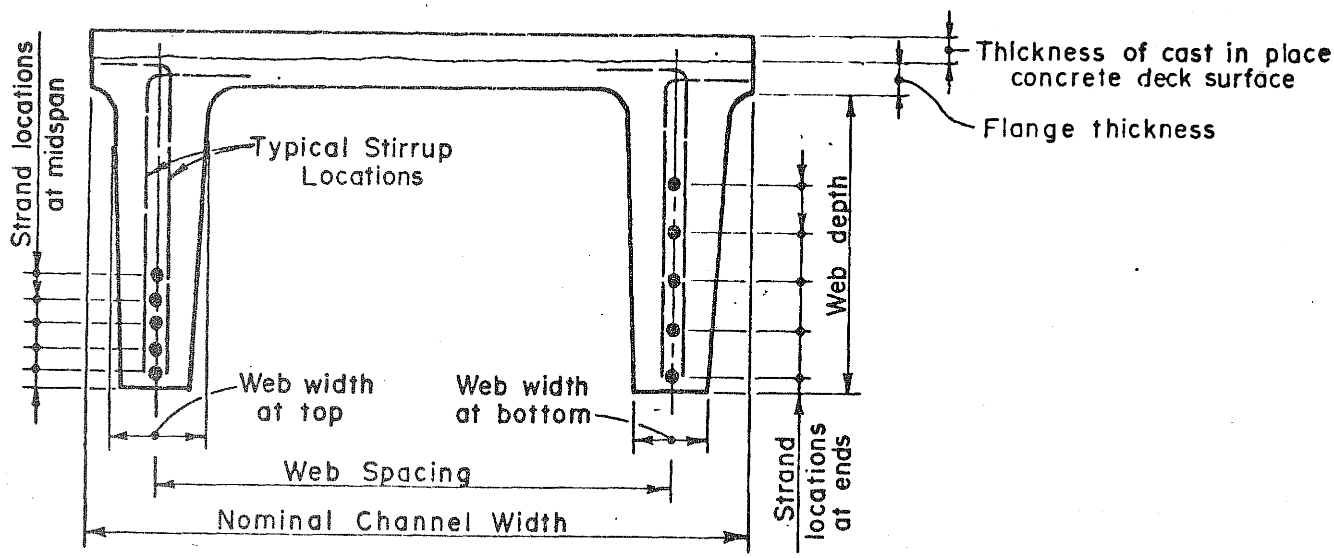


Figure 2-8

Prestressed Channel Section:



Number of Channels = ____________

Thickness of cast in place concrete deck surface = __________ in.

Flange thickness = ____________ in.

Web depth = ____________ in.

Web width at top = _____________ in.

Web width at bottom = ____________ in.

Web spacing = ___________ in.

Nominal channel width = __________ in.

Strand locations at ends:

Distance from bottom of web to 1st strand = ___________ in.
Distance from bottom of web to 2nd strand = ___________ in.
Distance from bottom of web to 3rd strand = ___________ in.
Distance from bottom of web to 4th strand = ___________ in.
Distance from bottom of web to 5th strand = ___________ in.

Strand locations at mid span:

Distance from bottom of web to 1st strand = ___________ in.
Distance from bottom of web to 2nd strand = ___________ in.
Distance from bottom of web to 3rd strand = ___________ in.
Distance from bottom of web to 4th strand = ___________ in.
Distance from bottom of web to 5th strand = ___________ in.

SECTION 2.5 - PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM INSPECTION DATA SHEET (Continued)

Prestressing Strand Size: ________ in. diameter; Grade ____________

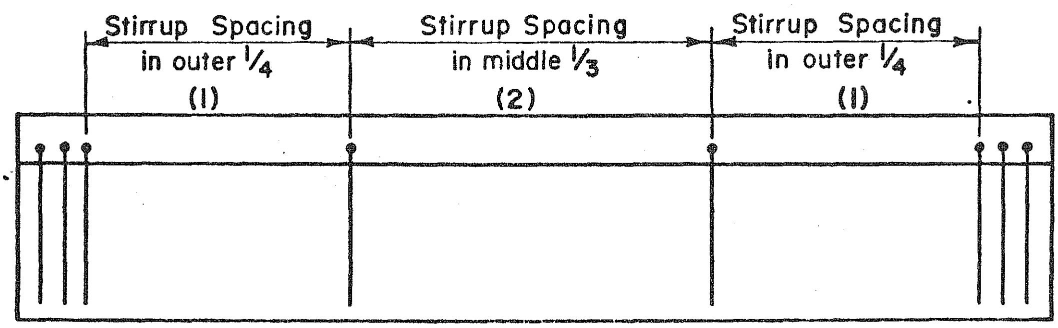
(270k high-strength or 250k ASTM designation)

Initial Tensioning Load ____________ lbs. each

Concrete Strength = __________ psi. (for prestressed channels)

Concrete Strength = __________ psi. (for cast in place concrete deck)

Shear Reinforcing (Stirrups):



Side View of Channel

outer 1/4 (1)	Stirrup Spacing = __________ in. Stirrup Bar Size = __________ fy (yield strength of steel) = __________ psi. (From contract plans[1])
middle 1/3 (2)	Stirrup Spacing = __________ in. Stirrup Bar Size = __________ fy (yield strength of steel) = __________ psi. (from contract plans)

---

[1]When the Grade is given, or the steel is unknown, use the following yield stresses for reinforcing steel:

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

CHAPTER 3

CALCULATION OF LIVE LOAD SHEARS AND MOMENTS

3.1 Overview for the calculation of Live Load Shears and Moments

3.2 Computation of Maximum Live Load Moment

3.3 Computation of Maximum Live Load Shear

3.4 Example Computations

APPLIED LIVE LOAD SHEARS and APPLIED LIVE LOAD MOMENTS are the forces resulting from the weight or loads of the trucks.

A LOAD CASE represents a truck and its specific set of axle loads and spacings designed to produce the greatest stresses in the bridge members for that given number of axles.

SECTION 3.1 - OVERVIEW FOR THE CALCULATION OF LIVE LOAD SHEARS AND MOMENTS

This chapter provides the mechanics necessary for computing the maximum applied live load shears and moments for various loading conditions. In subsequent chapters the allowable shears and moments for the bridge members will be computed and compared with these applied live load shears and moments to arrive at the load rating for the structure. The format of this chapter provides the shear and moment equations along with the necessary directions and discussion. The equations are to be evaluated on separate sheets of paper in a neat and systematic manner, following the example presented in Section 3.4. It is emphasized that accuracy is a must when computing the maximum live load shears and moments (as well as all computations in this manual) since computational errors will directly affect the final Rating Values.

STEPS FOR CALCULATION OF LIVE LOAD SHEARS AND MOMENTS

STEP 1: DEFINE PROBLEM

Compile the data necessary to perform the calculations for determining the maximum Live Load Moment and maximum Live Load Shears. The necessary data is as follows:

1. Type of Superstructure (timber deck and stringer, reinforced concrete slab, etc.)
2. Span Length (Ft.)
3. Design Loading (if known)

STEP 2: DETERMINE LOADING CASES

Illustrated in Appendix A on Plate III are six (6) current Florida Legal Load Cases and two (2) AASHTO[1] Load Cases. These cases can be changed periodically and the user should confirm their status with F.D.O.T. prior to useage. Current AASHTO specs.[2] should be consulted to determine

---

[1]American Association of State Highway and Transportation Officials.

[2]Standard Specifications for Highway Bridges, Twelfth Edition, 1977, adopted by the American Association of State Highway and Transportation Officials, published by the Association General Offices, Washington, D. C. and approved Interm Specifications (or later edition if available.

the current AASHTO Load Cases, and then compared with the AASHTO Load Cases shown on Plate III to arrive at the design load cases to be considered. When the design loading is known (from construction plans) only that AASHTO Load Case will need to be evaluated for maximum Live Load Moment and Shear. When design loading is unknown, both AASHTO Load Cases should be evaluated for maximum Live Load Moment and Shear. Each Legal and AASHTO Load Case that is deemed applicable will need to be evaluated for maximum Moment and Shear. If the current Load Cases are identical to those shown on Plate III and design loading is unknown, eight Load Cases will need to be evaluated for maximum Moment and Shear.

STEP 3: COMPUTE MAXIMUM LIVE LOAD MOMENTS FOR EACH LOADING CASE

In the event that some (or all) of the current loading cases are identical to the Load Cases illustrated on Plate III, the maximum applied Live Load Moments (LLM) for those Load Cases may be determined from the table[3] on Plate I in the appendix. When using the table and the span length falls between the given span lengths, the reader should interpolate to get the maximum applied Live Load Moment.

In the event that some (or all) of the current loading cases are different from the Load Cases illustrated on Plate III, the maximum applied Live Load Moments for those Load Cases should be determined by the methods presented in Section 3.2.

STEP 4: COMPUTE MAXIMUM LIVE LOAD SHEAR FOR EACH LOADING CASE

The maximum applied Live Load Shear for each loading case should be determined by the methods presented in Section 3.3.

---

[3]This table is provided for the convenience of the user of this manual, and it is emphasized that the tables were calculated based on the load cases shown on Plate III. If a current load case does not match a load case on Plate III <u>identically</u>, <u>do not use the table on Plate I</u>.

EXAMPLE OF INTERPOLATION IN THE TABLE ON PLATE I

We want the maximum applied Live Load Moment due to C4 loading on a 44.2 foot span.

On Plate I notice there is no 44.2 ft. span length, but for a 44.0 ft. span length the maximum applied Live Load Moment = 453.0 k-ft. and for a 44.5 ft. span length the maximum applied Live Load Moment = 461.1 k-ft.;

so: $$\frac{44.2 - 44.0}{44.5 - 44.0} = \frac{x - 453.0}{461.1 - 453.0}$$

$$\frac{.2}{.5} = \frac{x - 453.0}{8.1}$$

$$x - 453.0 = 3.24$$
$$x = 3.24 + 453.0 = \underline{456.2\ \text{k-ft.}}$$

SECTION 3.2 - COMPUTATION OF MAXIMUM LIVE LOAD MOMENT

This section is to be completed for each current loading case that differs from those shown on Plate III in the Appendix.

Begin by defining the problem. State the span length and sketch the loading configuration showing the axle loads and dimensions. Next, it must be determined how to locate the loads on the span in order to yield maximum moment. It is not always obvious which combination of axle loads will accomplish this so usually more than one case will need to be checked and the controlling condition determined. On the following pages are axle placements, among which will be the controlling condition for the span lengths we are concerned with as part of this manual. The user of this manual should examine the loading case-span length combination under consideration and determine which axle placements are possible.*

Then the live load moment for each possible axle placement should be calculated in accordance with the equation of the applicable axle placement, the live load moment for each axle placement should be compared with the others, and the greatest moment for each particular load case is the one to be used in the rating calculations. Some of the axle placements shown have several conditions. The General Case may be used for any load/axle spacing combination. The other conditions are special cases of load/axle spacing combinations which have simpler equations and have been included for convenience. It is emphasized that the load conditions and their corresponding equations are intended to represent truck loadings only. The general and specific cases were developed considering the basic model of a truck as single unit or combination type vehicles. Variable loads and axle spacings were provided in order to allow for minor differences in loads and axle spacings between future loading conditions and those shown on Plate III. Where major differences in loads, axle spacings or number of axles exist between future loading conditions and those shown on Plate III, the equations do not apply and the maximum live load moment equations should be examined by a qualified engineer to determine their applicability.

The following data is necessary in order to evaluate the live load moment (LLM) equations:

1. The number of axles on the span.
2. The span length in feet.
3. The distance(s) between axles in feet.
4. The axle loads in kips.

A worked example problem is included in Section 3.4

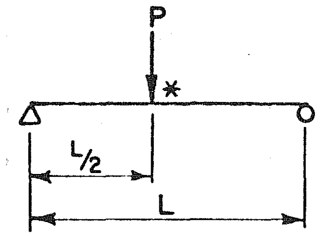
---

*It may be helpful to examine this procedure in the example computations given in Section 3.4.

SECTION 3.2 - COMPUTATION OF MAXIMUM LIVE LOAD MOMENT

Placement I: One Axle:

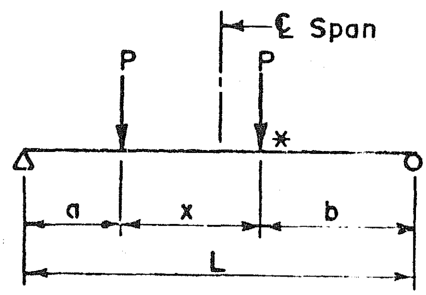
General Case:



$$LLM = \frac{PL}{4}$$

Placement II: Two Axles:

Condition A: Loads are equal

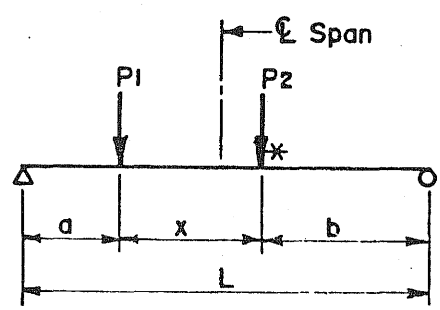


$$b = \frac{L}{2} - \frac{x}{4}$$

$$a = L - x - b$$

$$LLM = \frac{Pb}{L}(2a + x)$$

Condition B: General Case ($P_2 > P_1$)



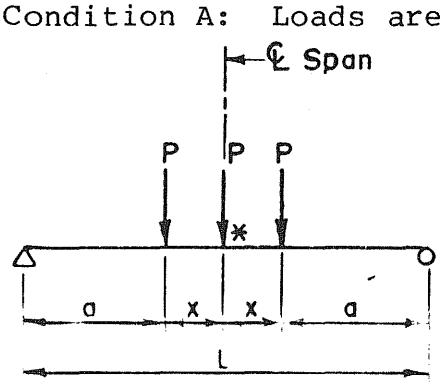
$$b = 1/2\left(L - \frac{P_1 x}{(P_1 + P_2)}\right)$$

$$a = L - x - b$$

$$LLM = \frac{ab}{L}\left(P_1 + P_2 + \frac{P_2 x}{a}\right)$$

Placement III: Three Axles

Condition A: Loads are equal, spacing is equal



$$a = \frac{L}{2} - x$$

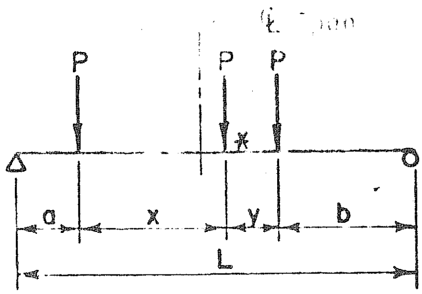
$$LLM = P\left(\frac{L}{4} + a\right)$$

---

*Maximum moment occurs at this load.

Placement III (Continued):

Condition B: Loads are equal, spacing is unequal ($x > y$)

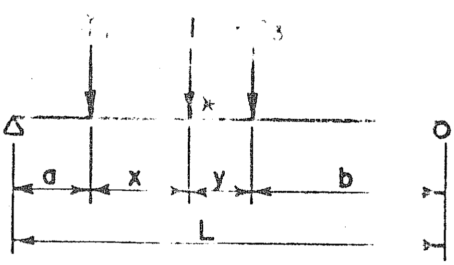


$$a = 1/6(3L - 5x - y)$$

$$b = L - x - y - a$$

$$LLM = \frac{P}{L}(3ay + 3ab + 2xy + 2xb + y^2 + yb - yL)$$

Condition C: General Case ($P_3 \geq P_2 \geq P_1$)



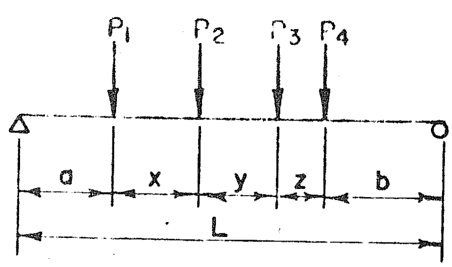
$$a = \frac{L}{2} - \frac{1}{2}\left(\frac{P_1x + 2P_2x + 2P_3x + P_3y}{P_1 + P_2 + P_3}\right)$$

$$b = L - x - y - a$$

$$LLM = \frac{a(b + y)}{L}(P_1+P_2+P_3)+\left(\frac{b+y}{L}\right)(P_2x+P_3x+P_3y)-P_3y$$

Placement IV: Four Axles ($P_4 \geq P_3 \geq P_2 \geq P_1$)

General Case:



$$a = \frac{L}{2} - \frac{1}{2}\left[\frac{P_1(x + y) + P_2(2x + y) + P_3(2x + 2y) + P_4(2x + 2y + z)}{P_1 + P_2 + P_3 + P_4}\right]$$

$$b = L - x - y - z - a$$

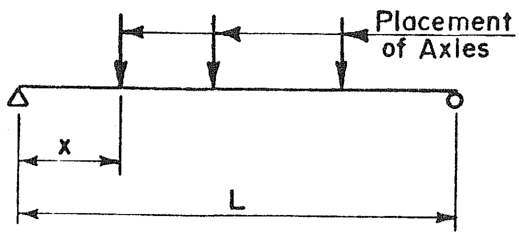
$$LLM = \frac{a(z + b)}{L}(P_1+P_2+P_3+P_4) + \left(\frac{z+b}{L}\right)[x(P_2+P_3+P_4)+y(P_3+P_4)+z(P_4)] - P_4z$$

---

*Maximum moment occurs at this load.

SECTION 3.3 - COMPUTATION OF MAXIMUM LIVE LOAD SHEAR

This Section is to be completed for each current loading case[1]. The point along the length of the stringer or beam where the maximum Live Load Shear is to be determined is specified in AASHTO Standard Specifications[2] and varies for the different superstructure types. For this reason no tables have been computed for maximum Live Load Shear and the Shear must be calculated for each loading case in accordance with the method presented here. The reader should determine how the loading case is to be positioned on the span for the particular superstructure type under examination (i.e., determine x as shown on the sketch below) and then complete the calculation of live load shear (LLV) for each loading case.



Determine Position of Load Case:

1. Timber Deck and Stringer Superstructure:

   x = Smaller of:

   x = 3 (Stringer Height)
   = 3 (in.)(1/12) = ____________ ft.

   or x = 1/4 (Span Length)
   = 1/4 (ft.) = ____________ ft.

   so, x = [   ] ft.

---

[1]The Live Load Shear does not need to be computed for timber deck and stringer type structures under certain conditions described in Section 4.2. Therefore, the reader may wish to omit the maximum Live Load Shear calculations when dealing with timber deck and stringer type superstructures until it is determined whether or not they will be required.

[2]Standard Specifications for Highway Bridges, Twelfth Edition, 1977; p. 110, p. 129, p. 261.

2. Reinforced Concrete Slab Superstructure and Reinforced Concrete Tee-Beam Superstructure:

x = 1/2 (Pier Cap Width) + (d)

= 1/2 (ft.) + (in.)(1/12) = $\boxed{\qquad}$ ft.

3. Prestressed Concrete Channel and Slab System Superstructure:

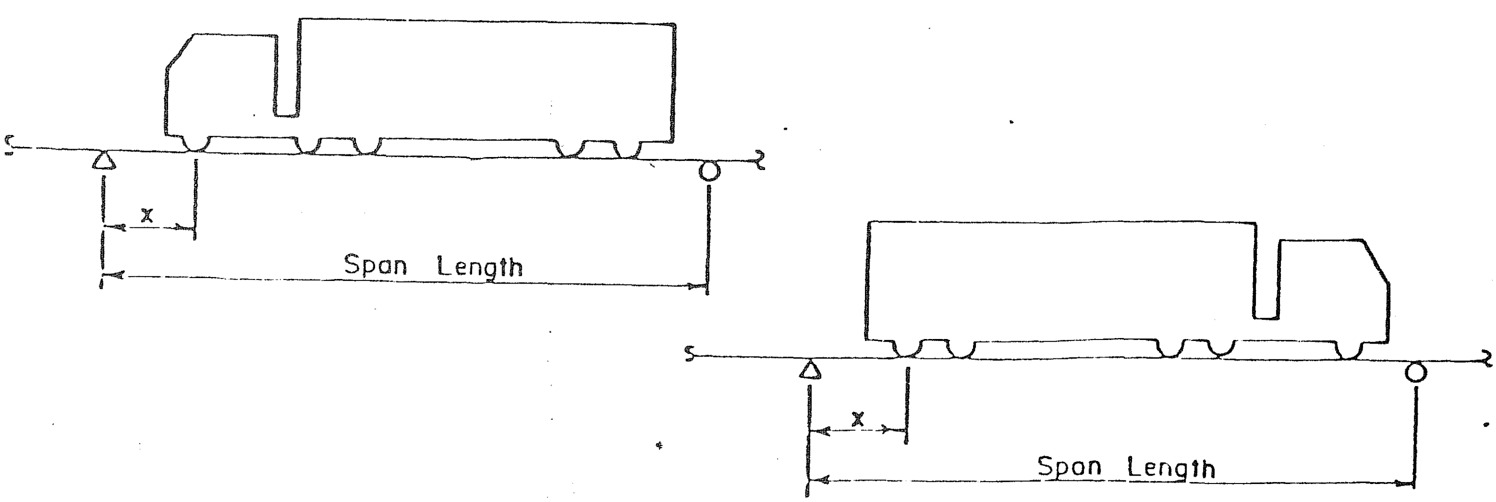
In general, shear is to be checked for the outer quarters of the channel and for the middle one third of the channel. This means that the live load shear must be computed at two points; the quarter point and the third point. The value of "x" at these points is:

At Quarter Point	At Third Point
x = 1/4 (Span Length)	x = 1/3 (Span Length)
x = 1/4 (ft.)	x = 1/3 (ft.)
x = $\boxed{\qquad}$	x = $\boxed{\qquad}$

One should note, however, that if the stirrup spacing is uniform over the length of the member only the shear at the quarter point needs to be checked, therefore only the live load shear at the quarter point need be computed.

Calculate Live Load Shear:

In situations where the bridge span length under consideration exceeds the total axle spacing from front to rear plus the calculated distance x for a particular load case, that load case may be placed on the span in two ways; with the front axle placed x away from the support or with the rear axle placed x away from the support (see sketch below).



 REV. 1181

It is seldom obvious which orientation will produce the maximum Live Load Shear, therefore both cases must be checked.

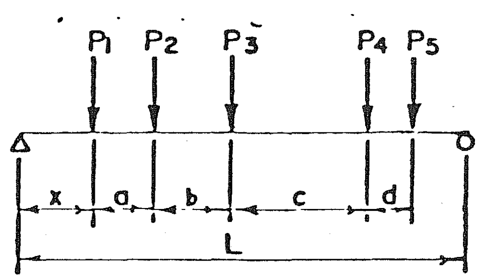
In situations where the total axle spacing from front to rear plus the calculated distance x exceeds the span length under consideration, more than two placements could exist.

The user should position each individual axle a distance x from the support and evaluate Live Load Shear. As the user becomes experienced, he will eventually be able to eliminate placements which would obviously not produce a maximum Live Load Shear. Users of this manual are encouraged to evaluate any placements which have not been included in the examples to assure themselves of the controlling Live Load Shear.

Shown below is a general equation to calculate the maximum Live Load Shear at a distance of x from the support. Up to five axles with different spacings may be specified. If the Load Case being examined has less than five axles on the span, simply delete the parts of the equation pertaining to the unused axles. (ex. if four axles are being used delete the term $P_5(1-\frac{x+a+b+c+d}{L})$).

$$LLV = P_1(1-\frac{x}{L})+P_2(1-\frac{x+a}{L})+P_3(1-\frac{x+a+b}{L})+P_4(1-\frac{x+a+b+c}{L})+P_5(1-\frac{x+a+b+c+d}{L})$$

Where:



A worked example problem has been included in Section 3.4.

STEP I: DEFINE PROBLEM.

Record superstructure type, span length, and design loading.

STEP II: DETERMINE LOADING CASES.

Confirm the status of the Legal Load and AASHTO Load Cases with the governing agencies and construction plans respectively.

STEP III: COMPUTE MAXIMUM LIVE LOAD MOMENTS FOR EACH LOADING CASE.

Since the current loading cases matched those shown on Plate III, the table on Plate I was used to determine the Live Load Moments.

For illustration of the method used to compute the Live Load Moment when a loading case does not match those shown on Plate III, evaluate the C4 loading case.

Timber deck and stringer type superstructure

Span Length = 25.0'

Design Loading = H-15

Loading cases are the same as those shown on Plate III, therefore use the table on Plate I to obtain maximum live load moment for each loading case.

From Plate I at span length = 25.0 ft.

SU2 = 141.7 ft-K

SU3 = 249.1 ft-K

SU4 = 272.7 ft-K

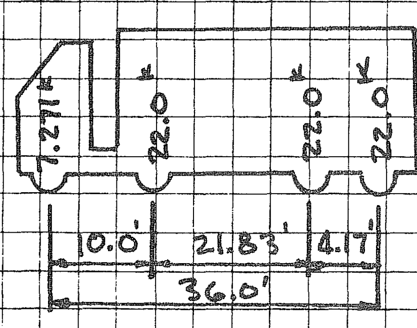
C3 = 156.7 ft-K

C4 = 231.1 ft-K

C5 = 231.1 ft-K

H = 200.0 ft-K

CONSIDER C4 LOADING CASE :



GVW = 73.271^K

Determine the axle placements that are possible for the C4 axle spacings and a 25 foot span:

From Section 3.2 notice that the one axle placement requires the axle to be placed (L/2) from the support (ie. at midspan). By placing any C4 axle at midspan there will always be another axle on the span, therefore the one axle placement is not possible.

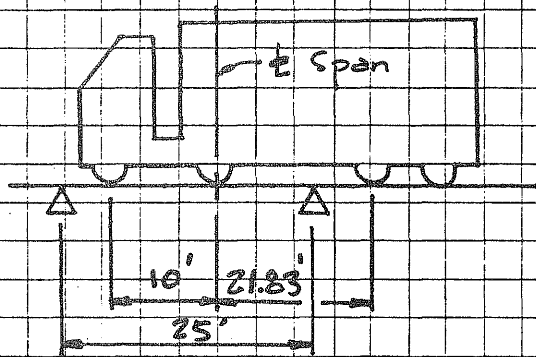
As shown on the next page, there are two possible axle placements for two axles on the span. Since it is not obvious which placement will yield greater moments, both must be evaluated.

In order to have three axles on the span, the minimum span length would have to be the distance between the closest three axles. For the C4 loading case the distance between the closest three axles is 4.17 feet plus 21.83 feet or 26.0 feet. This is greater than the span length so a three axle placement is not possible for a C4 loading case and a 25.0 foot span. Obviously, a four axle placement is not possible either.

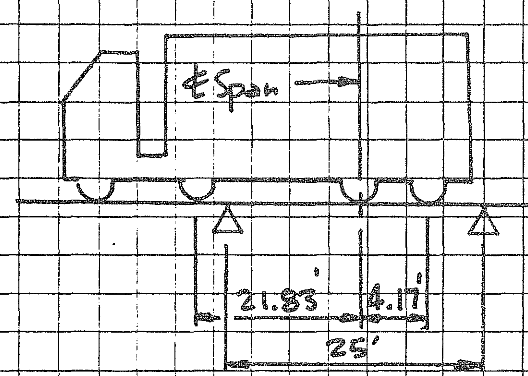
DATE	DESIGN	SEC. 3.4 - EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Try one axle on span:



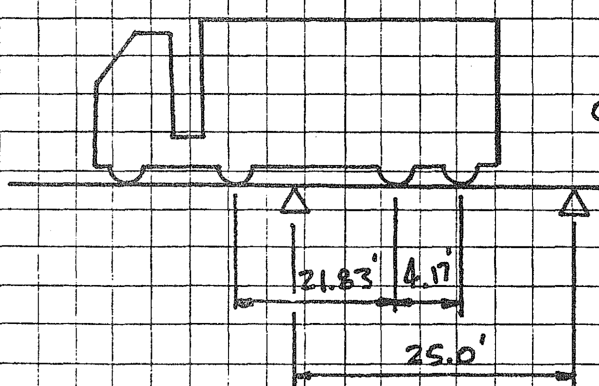
OR



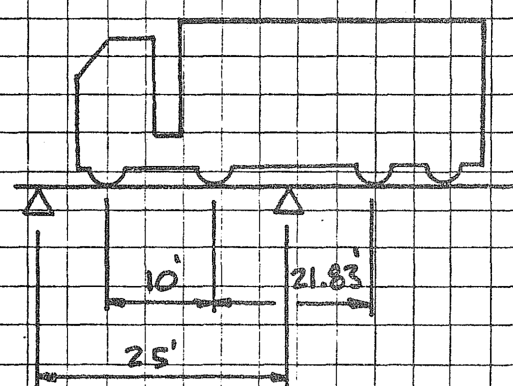
Not Possible

Not Possible

Try two axles on span:



OR



Case I

Case II

Both Possible

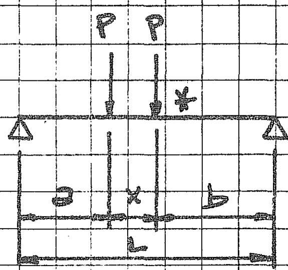
DATE | DESIGN | SHEET OF

CHECK | JOB | FOR | JOB NO.

SUBJECT

Sec. 3.4 - Example Computations Cont.

Examine Case I :



Where: $P = 22.0^k$

$X = 4.167$ ft.

$L = 25.0$ ft.

$b = \frac{L}{2} - \frac{x}{4}$ $\qquad$ $a = L - x - b$

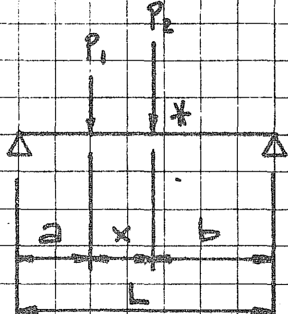
$b = \frac{25}{2} - \frac{4.167}{4}$ $\qquad$ $a = 25 - 4.167 - 11.458$

$b = 11.458$ ft. $\qquad$ $a = 9.375$ ft.

$$LLM = \frac{Pb}{L}(2a + x) = \frac{(22.0)(11.458)}{25.0}\left[(2)(9.375) + 4.167\right]$$

$LLM = 231.07$ ft-K

Examine Case II :



Where : $P_1 = 7.271^k$

$P_2 = 22.0^k$

$X = 10.0$ ft.

$L = 25.0$ ft.

Summarizing, there are two - two axle placement cases that must be checked for moment in order to determine the maximum live load moment for the C4 loading case and a 25.0 foot span.

Compare the loads for Case I with the axle placement conditions shown in Section 3.2. Since axle loads are equal use Condition A to compute maximum Live Load Moment (LLM).

Compare the loads for Case II with the axle placement conditions shown in Section 3.2. Since the axle loads are unequal for Case II use Condition B to compute maximum Live Load Moment (LLM).

Note that if "a" or "b" comes out negative it means the given loads and axle spacing cannot be placed on the span in a manner to achieve maximum moment. Therefore, that axle placement case should be disregarded.

Comparing Case 1 and Case 2, it is found that Case 1 controls and the maximum Live Load Moment for this set of loads, axle placements, and span length is 231.1 ft.-k. Furthermore, notice that the maximum Live Load Moment of 231.1 f-k. as calculated by placing the axle loads and evaluating the appropriate equation matches identically with the maximum Live Load Moment for C4 load case as determined by the table on Plate I, as would be expected.

STEP IV: COMPUTE MAXIMUM LIVE LOAD SHEAR FOR EACH LOADING CASE

Referring to Section 3.3 determine "x" for a timber deck and stringer superstructure. For purposes of this example, let stringer height = 20 inches.

DATE	DESIGN	Sec. 3.4 - Example Computations Cont.	SHEET OF
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$b = \frac{1}{2}\left(L - \frac{P_1 x}{P_1 + P_2}\right)$ $\quad a = L - x - b$

$b = \left(\frac{1}{2}\right)\left[25.0 - \frac{(7.271)(10)}{(7.271 + 22)}\right]$ $\quad a = 25.0 - 10.0 - 11.258$

$b = 11.258$ ft. $\quad a = 3.742$

$LLM = \frac{ab}{L}\left(P_1 + P_2 + \frac{P_2 x}{a}\right)$

$LLM = \frac{(3.742)(11.258)}{25}\left[7.271 + 22 + \frac{(22)(10)}{3.742}\right]$

$LLM = 148.4$ ft-K

Therefore LLM for C4 $= \boxed{231.07 \text{ ft-K}}$

Maximum Live Load Shear:

$x = 3$ (Stringer Height)

$x = 3(20 \text{ in.})\left(\frac{1 \text{ ft.}}{12 \text{ in.}}\right) = 5.0$ ft.

OR $x = 1/4$ (Span Length)

$x = 1/4(25.0) = 6.25$ ft.

Use Smaller value, so $x = 5.0$ ft.

Next, determine the maximum Live Load Shear for each Loading Case.

SU2 Load Case: Since both cases have two axles on the span, the general equation reduces to:

$$LLV = P_1(1 - \frac{x}{L}) + P_2(1 - \frac{x + a}{L})$$

by dropping the P_3, P_4, and P_5 terms from the general equation in Section 3.3

SU3 Load Case: Since both cases have three axles on the span, the general equation reduces to:

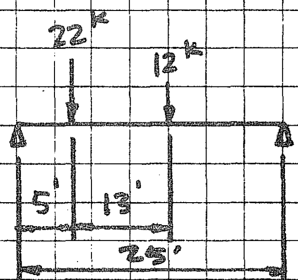
$$LLV = P_1(1 - \frac{x}{L}) + P_2(1 - \frac{x+a}{L}) + P_3(1 - \frac{x+a+b}{L})$$

by dropping the P_4 and P_5 terms from the general equation in Section 3.3.

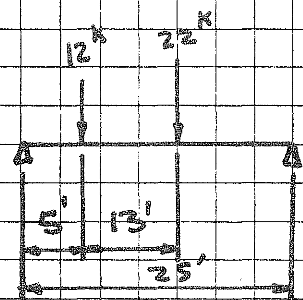
DATE	DESIGN	Sec. 3.4 - Example Computations Cont.		SHEET / OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Examine SU2 Load Case



OR



Case I

Case II

Evaluate case I :

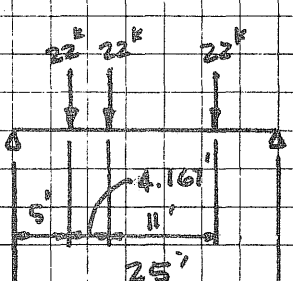
$$LLV = 22\left(1-\frac{5}{25}\right) + 12\left(1-\frac{5+13}{25}\right) = 20.96 \text{ kips}$$

Evaluate Case II :

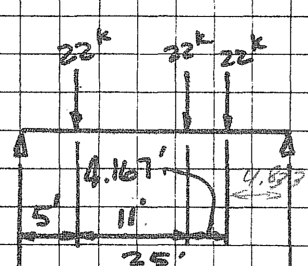
$$LLV = 12\left(1-\frac{5}{25}\right) + 22\left(1-\frac{5+13}{25}\right) = 15.76 \text{ kips}$$

So, Max. LLV for SU2 = $\boxed{20.96^k}$

Examine SU3 Load Case



OR



case I

case II

Evaluate Case I

$$LLV = 22\left(1-\frac{5}{25}\right) + 22\left(1-\frac{5+4.167}{25}\right) + 22\left(1-\frac{5+4.167+11}{25}\right)$$

$LLV = 35.79$ Kips

SU4 Load Case: Since both cases have four axles on the span, the general equation reduces to:

$$LLV = P_1(1 - \frac{x}{L}) + P_2(1 - \frac{x+a}{L}) + P_3(1 - \frac{x+a+b}{L}) + P_4(1 - \frac{x+a+b+c}{L})$$

by dropping the P_5 term from the general equation in Section 3.3.

DATE	DESIGN	SEC. 3.4 - EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

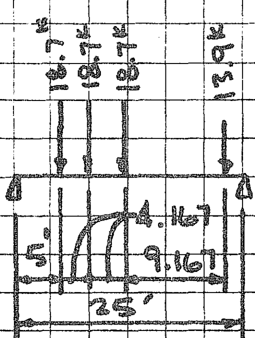
Evaluate Case II

$$LLV = 22\left(1-\frac{5}{25}\right) + 22\left(1-\frac{5+11}{25}\right) + 22\left(1-\frac{5+11+4.167}{25}\right)$$

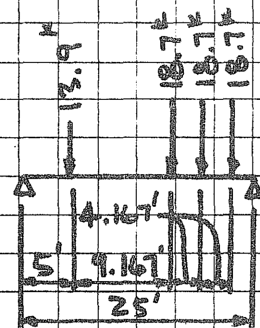
$LLV = 29.77$ Kips

So, Max. LLV for SU3 = $\boxed{35.79^{k}}$

Examine SU4 Load Case



OR



Case I

Case II

Evaluate case I

$$LLV = 18.7\left(1-\frac{5}{25}\right) + 18.7\left(1-\frac{5+4.167}{25}\right) + 18.7\left(1-\frac{5+4.167+4.167}{25}\right)$$
$$+ 13.9\left(1-\frac{5+4.167+4.167+9.167}{25}\right)$$

$LLV = 36.92$ Kips

Evaluate Case II

$$LLV = 13.9\left(1-\frac{5}{25}\right) + 18.7\left(1-\frac{5+9.167}{25}\right) + 18.7\left(1-\frac{5+9.167+4.167}{25}\right)$$
$$+ 18.7\left(1-\frac{5+4.167+4.167+9.167}{25}\right)$$

$LLV = 26.08$ Kips

So, Max. LLV for SU4 = $\boxed{36.92^{k}}$

C3 Load Case: In each case one axle falls completely off the span and will not contribute to the maximum Live Load Shear. Therefore, the general equation reduces to:

$$LLV = P_1(1 - \frac{x}{L}) + P_2(1 - \frac{x+a}{L})$$

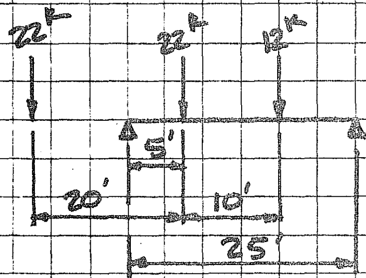
by dropping the P_3, P_4, and P_5 from the general equation in Section 3.3

C4 Load Case: Since both cases have only two axles on the span, the general equation reduces to:

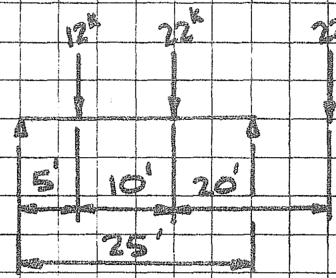
$$LLV = P_1(1 - \frac{x}{L}) + P_2(1 - \frac{x+a}{L})$$

by dropping the P_3, P_4, and P_5 terms from the general equation in Section 3.3

Examine C3 Load Case



OR



Case I

Case II

Evaluate Case I

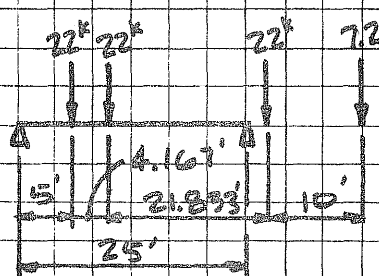
$LLV = 22\left(1-\frac{5}{25}\right) + 12\left(1-\frac{5+10}{25}\right) = 22.40$ Kips

Evaluate Case II

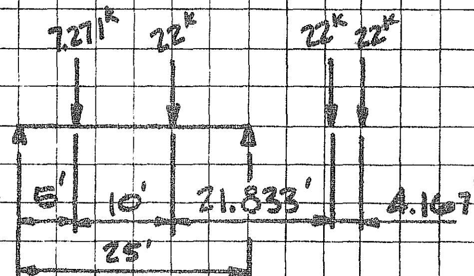
$LLV = 12\left(1-\frac{5}{25}\right) + 22\left(1-\frac{5+10}{25}\right) = 18.40$ Kips

So, Max. LLV for C3 $= \boxed{22.40^K}$

Examine C4 Load Case



OR



Case I

Case II

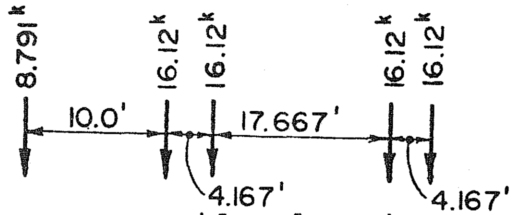
Evaluate Case I

$LLV = 22\left(1-\frac{5}{25}\right) + 22\left(1-\frac{5+4.167}{25}\right) = 31.53$ Kips

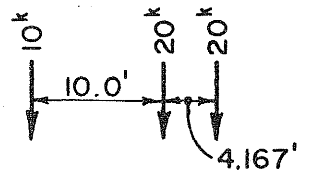
Evaluate Case II

$LLV = 7.271\left(1-\frac{5}{25}\right) + 22\left(1-\frac{5+10}{25}\right) = 14.62$ Kips

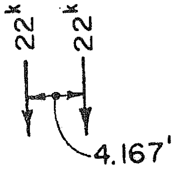
C5 Load Case: For conditions where all five axles are on one span, the C5 load case shall be:



When only the tractor axles are considered to be on one span, the C5 load case shall be taken as:



and when only the trailer axles are considered to be on one span the C5 load case shall be taken as:



Now, since the span under evaluation has a 25 foot span length it would be impossible for all five axles to be on the span at the same time, so we are limited to either the three tractor axles or the two trailer axles on the span. The trailer axles represent one case to be investigated (call it case I) and they may be placed only one way on the span. Since there are two axles on the span the general equation reduces to:

$$LLV = P_1(1 - \frac{x}{L}) + P_2(1 - \frac{x+a}{L})$$

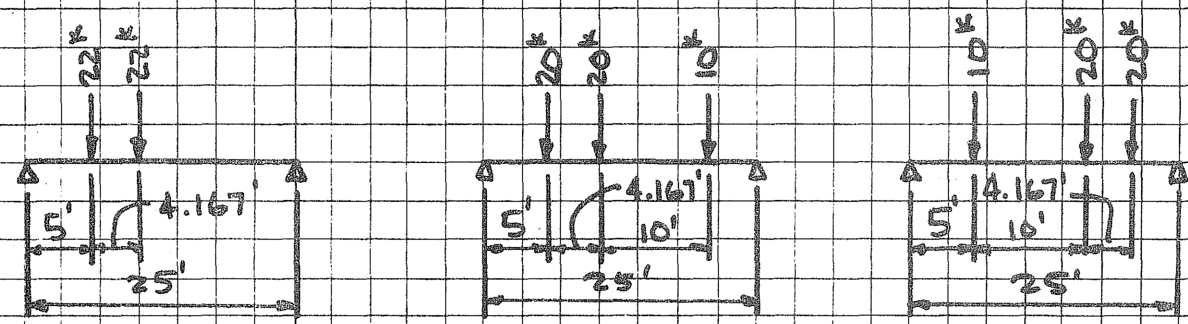
by dropping the P_3, P_4, and P_5 terms from the general equation in Section 3.3. The tractor axles represent two cases to be investigated. Case II and III each have three axles on the span, and the general equation reduces to:

$$LLV = P_1(1 - \frac{x}{L}) + P_2(1 - \frac{x+a}{L}) + P_3(1 - \frac{x+a+b}{L})$$

by dropping the P_4 and P_5 terms from the general equationin Section 3.3.

So, Max. LLV for C4 = $\boxed{31.53^k}$

<u>Examine C5 Load Case</u>



Case I   Case II   Case III

Evaluate Case I

$$LLV = 22\left(1-\frac{5}{25}\right)+22\left(1-\frac{5+4.167}{25}\right) = 31.53 \text{ Kips}$$

Evaluate Case II

$$LLV = 20\left(1-\frac{5}{25}\right)+20\left(1-\frac{5+4.167}{25}\right)+10\left(1-\frac{5+4.167+10}{25}\right)$$

LLV = 31.00 Kips

Evaluate Case III

$$LLV = 10\left(1-\frac{5}{25}\right)+20\left(1-\frac{5+4.167}{25}\right)+20\left(1-\frac{5+4.167+10}{25}\right)$$

LLV = 25.33 Kips

So, Maximum LLV for C5 = $\boxed{31.53 \text{ Kips}}$

H Load Case: Since both cases have two axles on the span, the general equation reduces to:

$$LLV = P_1\left(1 - \frac{x}{L}\right) + P_2\left(1 - \frac{x+a}{L}\right)$$

by dropping the P_3, P_4, and P_5 terms from the general equation in Section 3.3.

And finally, it is good practice to summarize the results so they may be easily found when performing rating calculations.

DATE	DESIGN	SEC. 3.4 - EXAMPLE COMPUTATIONS CONT.		SHEET OF
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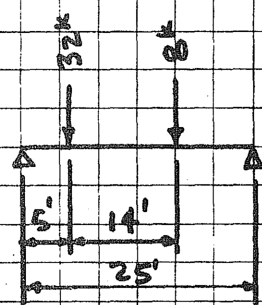
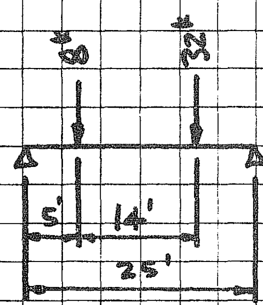
Examine H Loading Case



Case I



Case II

Evaluate Case I

$LLV = 32\left(1-\frac{5}{25}\right) + 8\left(1-\frac{5+14}{25}\right) = 27.52 \text{ kips}$

Evaluate Case II

$LLV = 8\left(1-\frac{5}{25}\right) + 32\left(1-\frac{5+14}{25}\right) = 14.08 \text{ kips}$

So, Max. LLV for H Loading is $\boxed{27.52^k}$

Summary of Maximum LLV

SU2 = 20.96^k

SU3 = 35.79^k

SU4 = 36.92^k

C3 = 22.40^k

C4 = 31.53^k

C5 = 31.53^k

H = 27.52^k

CHAPTER 4

TIMBER DECK AND STRINGER SUPERSTRUCTURE

SECTION 4.1 - OVERVIEW FOR THE STRUCTURAL RATING OF TIMBER DECK AND STRINGER SUPERSTRUCTURES

As indicated by excellent performance of timber highway bridges over many decades, wood has proven itself as a viable construction material for highway bridges. Timber is one of the stronger materials in proportion to its weight, and has an added advantage of being able to absorb impact stresses to the degree that impact loads may be neglected in the rating of wood highway structures.

As demonstrated by the fact that some timber highway bridges over 100 years old are still in service, timber is not significantly affected by deterioration when adequately protected. Lignin and cellulose, the primary constituents of wood, are not subject to significant chemical change with time and only slight deterioration occurs from long exposure to air. However, severe deterioration may occur given less than adequate measures of protection. Some of the leading causes of deterioration and damage to timber members are:

1) Fungi - decay of timber by molds and stains.

2) Vermin - decay by insects that tunnel in and hollow out the insides of timber members for food or shelter. Includes termites, powder post beetles, carpenter ants, and marine borers.

3) Weathering and Warping - decay as the result of repeated dimensional changes in the wood. Usually caused by repeated wetting.

4) Chemical Action - decay caused by accidental chemical spills, chemical action of runoff or chemical action of animal waste.

5) Fire - deterioration due to fire or heat.

6) Abrasion or Mechanical Wear - decay in the form of gradual loss of section at points of wear.

7) Collisions and Overloading - damage will be evidenced by shattering or cracking of timbers.

The analysis of timber bridges presented in this manual is in accordance with the "Standard Specifications for Highway Bridges"[1] and the

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1 Standard Specifications for Highway Bridges, Twelfth Edition, 1977, Adopted by the American Association of State Highway and Transportation Officials, published by the Association General Offices, Washington, D.C. and approved Interim Specifications.

SECTION 4.1 - OVERVIEW FOR THE STRUCTURAL RATING OF TIMBER DECK AND STRINGER SUPERSTRUCTURES (CONT.)

"Timber Construction Manual".[2] The rating procedures are in accordance with the AASHTO Bridge Manual.[3] As a consequence of the above specifications and generally accepted engineering practice, the following conditions are imposed on the analysis:

1) The analysis of timber members is by working stress methods.

2) Normal duration loading is used in the rating equations.

3) Impact is not considered in the analysis of timber bridges.

4) Horizontal shear is evaluated in lieu of vertical shear.

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2 American Institute of Timber Construction. Timber Construction Manual, Second Edition, Englewood, Colorado: John Wiley and Sons, 1974.

3 Manual for Maintenance Inspection of Bridges, 1978.

SECTION 4.2 - PROCEDURE FOR RATING A TIMBER BRIDGE SUPERSTRUCTURE

The following steps should be followed to compute the Inventory and Operating ratings for a Timber Bridge Superstructure. The Timber Bridge Inspection Data Sheets and calculation of Live Load Moments and Shears should already have been computed by the procedures of Chapter 2 and Chapter 3.

STEP 1. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR TIMBER FLOOR. This step is accomplished by completing the forms "Rating of Timber Flooring", Sec. 4.3.3. The ratings will be evaluated based on the moment carrying capacity of the floor. Horizontal shear is not critical for decks and will not be evaluated.[5] The portion of the deck to be evaluated should be that between the two stringers with the greatest spacing. This spacing and other data required to complete the forms "Rating of Timber Flooring" is to be obtained from the "Timber Bridge Inspection Data Sheet", Sec. 12.1 in Appendix C.

STEP 2. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR MOMENT CAPACITY FOR EXTERIOR STRINGERS. This step is accomplished by completing the forms "Moment Capacity Rating of an Exterior Stringer", Sec. 4.3.4 If the exterior stringers are similar in size, and the spacing to the next stringer is approximately the same, only one exterior stringer need be evaluated. However, if the exterior stringers are dissimilar in size or spacing; or if different loading conditions are present (e.g. there is a sidewalk on one side of the structure but not on the other) the forms "Moment Capacity Rating of an Exterior Stringer" will need to be completed for each individual and dissimilar exterior stringer.

The width and height of the stringer used in the equations should be the most critical section within the center one-half of the span (See Figure 4.1).

STEP 3. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR SHEAR CAPACITY FOR EXTERIOR STRINGERS. This step is accomplished by completing the forms "Shear Capacity Rating of an Exterior Stringer", Sec. 4.3.5. In general, horizontal shear does not control the stringer rating unless an adverse condition exists, such as a loss in section near the bearing area. If a loss in section near the bearing area* was noted on the "Timber Bridge Inspection Data Sheet", Sec. 2.2, Step 3 should be completed for the stringers in which the loss in

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[5] For additional information on Horizontal shear, the reader is directed to reference No. 1 in the Bibliography.

* Within one-fourth the span length of the end of the member. (See Figure 4-2.)

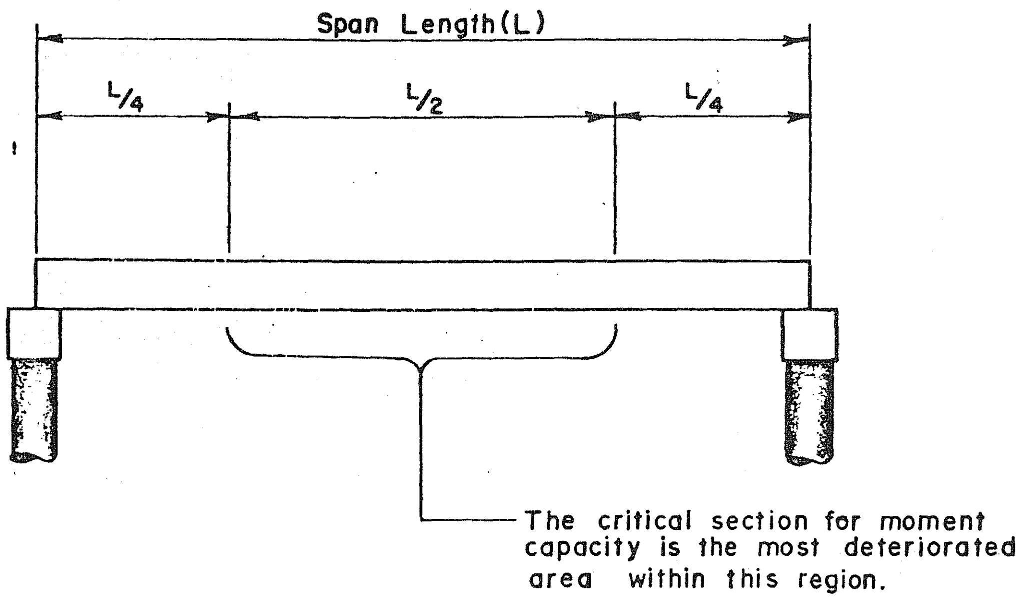
Span Length(L)
L/4
L/2
L/4
The critical section for moment capacity is the most deteriorated area within this region.

Figure 4-1

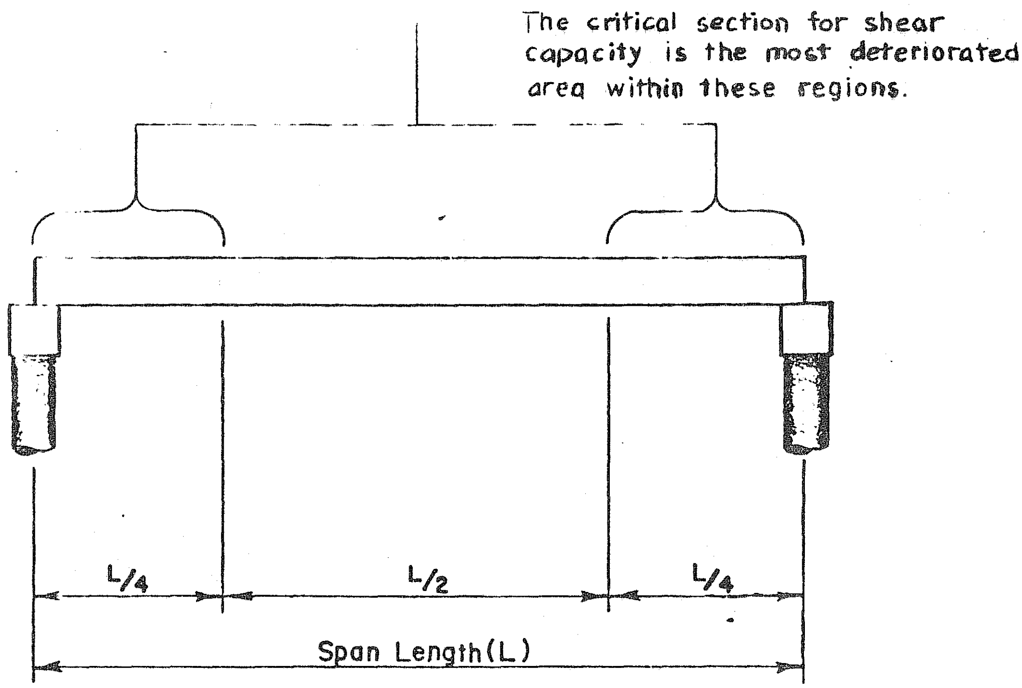
The critical section for shear capacity is the most deteriorated area within these regions.
L/4
L/2
L/4
Span Length(L)

Figure 4-2

section occurs. If there is no loss in section near the bearing area and no significant horizontal cracks are present, the stringer does not need to be rated for shear capacity and section 4.3.5 may be skipped.

STEP 4. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR MOMENT CAPACITY FOR INTERIOR STRINGERS. This step is accomplished by completing the forms "Moment Capacity Rating of an Interior Stringer", Sec. 4.3.6, for each interior stringer to be rated. Interior stringers falling in the following categories should be rated based on their moment carrying capacity.

1) Any stringer showing signs of distress or damage.

2) The stringer with the least cross-sectional area.

3) The stringer with the greatest distance between adjacent stringers.

4) A representative stringer, in the event the stringer sizes and spacings are approximately uniform.

The width and height of the stringer used in the equations should be the most critical section within the center one-half of the span.

STEP 5. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR SHEAR CAPACITY FOR INTERIOR STRINGERS. This step is accomplished by completing the forms "Shear Capacity Rating of an Interior Stringer", Sec. 4.3.7. As noted in Step 3, horizontal shear does not normally control the stringer rating unless an adverse condition exists, such as a loss in section near the bearing area. If a loss in section near the bearing area* was noted on the "Timber Bridge Inspection Data Sheet", Sec. 2.2, Step 5 should be completed for the stringers in which the loss in section occurs. If there is no loss in section near the bearing area and no significant horizontal cracks are present, the stringer does not need to be rated for shear capacity and section 4.3.7 may be skipped.

This completes the structural rating calculations. The Rating Values should be tabulated in accordance with the instructions at the end of each section and in Chapter 8.

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* Within one-fourth the span length of the end of the member. (See Figure 4-2.)

Section 4.3 illustrates the use of this manual to rate a Timber Deck and Stringer Superstructure. All forms needed to rate this type superstructure are included in Section 12.1 of Appendix C.

STEP 1 OBTAIN DATA

Figure 4-3 is an example Bridge Inspection Data Report in which the need for a structural rating has been determined by the engineer. No as-built plans are available. By examination of current legal load requirements it is determined that the legal load cases and AASHTO load cases shown on Plate III of Appendix A are applicable. Allowable stresses are determined by the Engineer to be f_b=2000 psi & f_v=90 psi.

SECTION 4.3 - EXAMPLE COMPUTATIONS

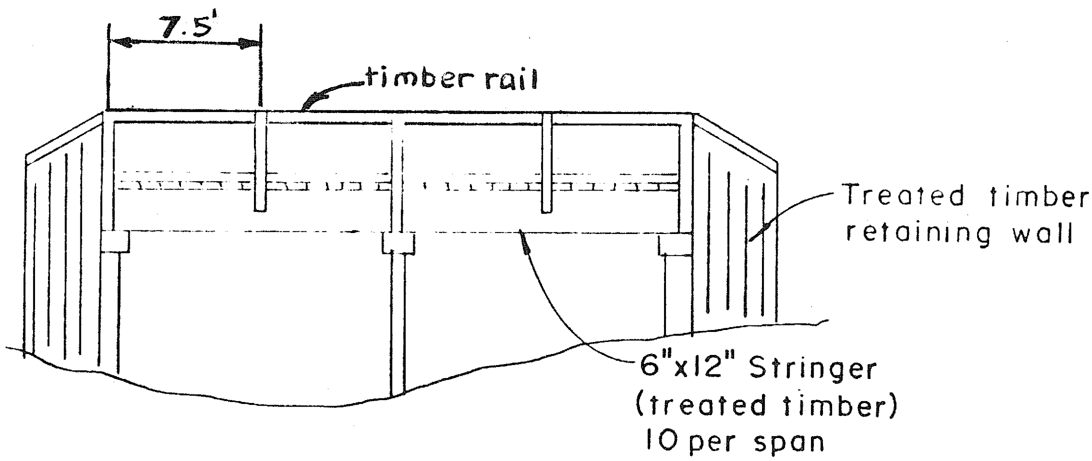
FIGURE 4-3 - EXAMPLE BRIDGE INSPECTION DATA REPORT

Bridge No. XXXXXXX - XXXXXXX County - SR XXXXXXX

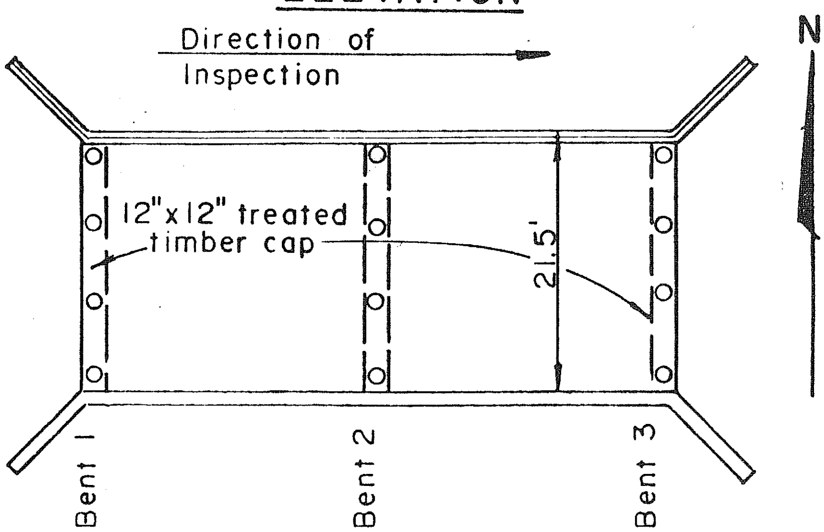
Description: located on SR XXX, one mile east of SR 00. It is a two span, all timber structure carrying local traffic east and west, and was constructed in the early 1960's. Two lanes are provided for traffic.

The following measurements were made during the inspection:

Overall Length = 28 ft.
Spans = 2 at 14'
Piling = 10" dia.
Curb to Curb (inside) = 20.5'
Roadway Approach = 24'
Skew = 0°



ELEVATION



PLAN

FIGURE 4-3

Approaches: This is a tangent roadway approaching both ends of the structure with a vertical curve which creates a hump-back effect. Drainage ditches on all four corners provide adequate handling of the runoff with some small washouts around the wings.

Waterway: A branch of XX creek flows between bents 1 and 2 with about 2 ft. of water. A sandy bottom exists with considerable vegetation growing in the upstream area.

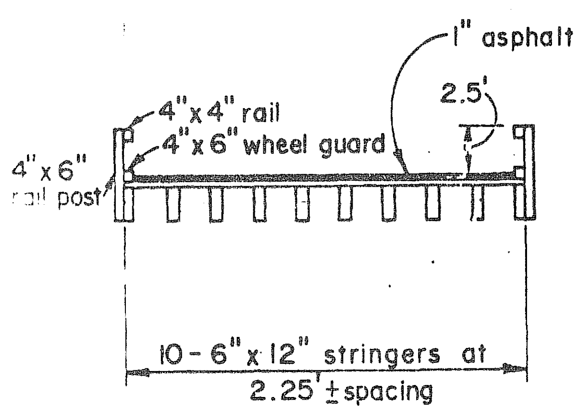
Utilities: None.

Piling: Six at each bent. 10" ø treated timber all driven in the plumb position. Approximately 1/2" to 1" deterioration exists at the mud line on each pile. After penetrating this outer area, a solid interior was found to exist.

Caps: 12" x 12" treated timber caps, all somewhat weathered and cracked.

Stringers: 6" x 12" treated timber stringers with an average spacing of 2.25' and a length of 15'. Stringers are in good condition except as noted below.

Span 2 stringer No. 9 (as numbered from left to right): longitudinal cracking, bleaching, and small amount of fungus growth on north side of stringer. Deterioration consisting of dry rot and insect infestation found at the west end of stringer measuring 1" x 3" wide x 1' - 6" long on the top and 1" x 3" wide x 12" long on the bottom. (See Sketch A on Page 4-15).



SECTION

Span 2 stringer No.10: deterioration (Dry Rot) found on top of stringer measuring 1" x 6" wide x 15' long and 1" x 6" wide x 14" long on bottom of stringer. (See Sketch B on Page 4-15).

Deck: Treated timbers 3-1/2" x 12" wide with 1" asphalt surfacing. Deck is in good condition except in vicinity of stringer deterioration at southeast end of bridge where effective deck thickness is reduced to 3" x 12".

STEP 2 DETERMINE TYPE OF SUPERSTRUCTURE

Compare the description of the superstructure given in the Bridge Data Inspection Report with the configurations detailed in Chapter 2. The timber deck in Section 2.2 is found to be applicable.

STEP 3 COMPLETE INSPECTION DATA SHEET

Obtain a photocopy of the "Timber Deck and Stringer Superstructure Inspection Data Sheets" (Section 12.1) and record the necessary information in the spaces provided. When information requested on the inspection data sheet is not available from the Bridge Inspection Data Report prepared by the field inspector, a field visit will be necessary to obtain the information.

SECTION 4.3.1 - TIMBER DECK AND STRINGER SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE:

1. These sheets should be completed in their entirety before continuing with the rating computations.
2. Sketches of the bridge superstructure under consideration should be made and dimensions, spacings, and direction of inspection should be shown thereon. These sketches should include a section and plan similar to Figures 2-1 and 2-2.

By: John Doe Date: xx-xx-xx

Bridge Name: State Road x over xx Creek

Bridge No.: xxxxx Road No.: x County: xx

Bridge Length = 28 ft. Number of Spans = 2

Span Length (℄ support to ℄) = 2 at 14 ft. (if different list each)

Overall Bridge Width (out-to-out) = 21.5 ft.

Roadway Width (curb to curb) = 20.5 ft.

Sidewalks: None

LEFT SIDE (looking in direction of inspection):

Width = — ft.

Rail Size = — in. x — in.

Rail Post Size & Length = — in. x — in. x — ft.

Average Post Spacing = — ft.

RIGHT SIDE (looking in direction of inspection):

Width = ___—___ ft.

Rail Size = ___—___ in. x ___—___ in.

Rail Post Size & Length = ___—___ in. x ___—___ in. x ___—___ ft.

Average Post Spacing = ___—___ ft.

Barrier Rails: Rail Size = ___4___ in. x ___4___ in.

Rail Post Size & Length = ___4___ in. x ___6___ in. x ___2.5___ ft.

Average Post Spacing = ___6.25___ ft.

Wearing Surface:

asphalt; thickness = ___—___ in.

concrete; thickness = ___—___ in.

Deck:

Type; Strip (tongue & groove or dowled) or Plank

Continuous over three or more stringers

thickness = ___3 1/2"___ in. (average thickness not considering deterioration)

thickness = ___3"___ in. (average thickness taking into account deterioration.)

Width of Strip or Plank = ___12___ in.

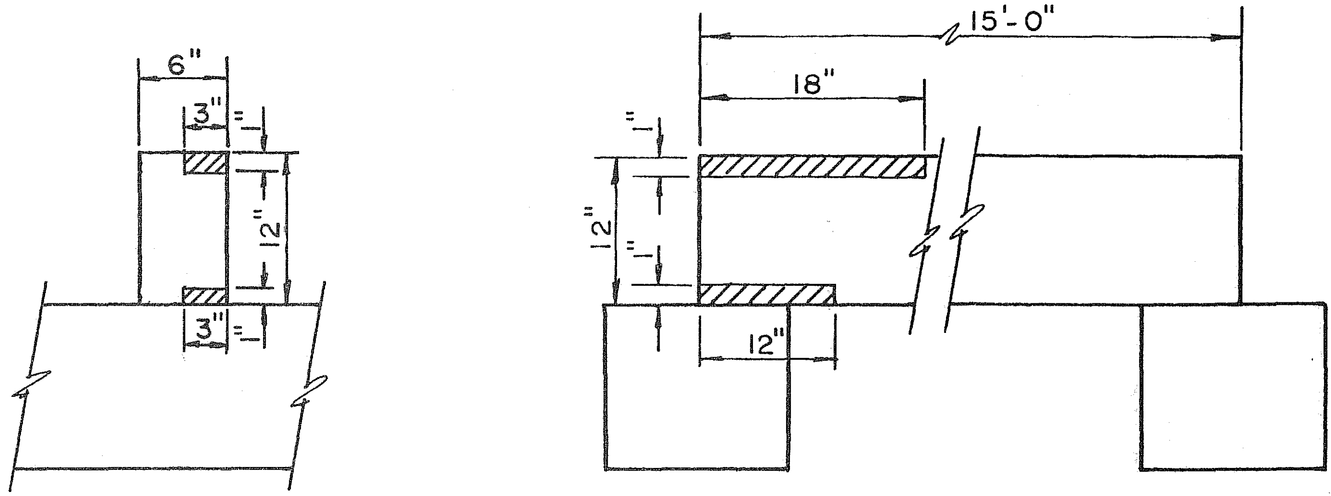
SECTION 4.3.1 - TIMBER DECK AND STRINGER SUPERSTRUCTURE INSPECTION DATA SHEET (CONT.)

STRINGER DATA:

Stringer Number	Stringer Cross-Sectional Dimensions (Height and Width) (inches)	Spacing Between Adjacent Stringers (ft.)
SPAN 1		
1-10	12 x 6	2.25 TYPICAL
SPAN 2		
1-8	12 x 6	2.25 TYP.
9	12 x 6 (See Sketch A)	2.25
10	12 x 6 (See Sketch B)	2.25

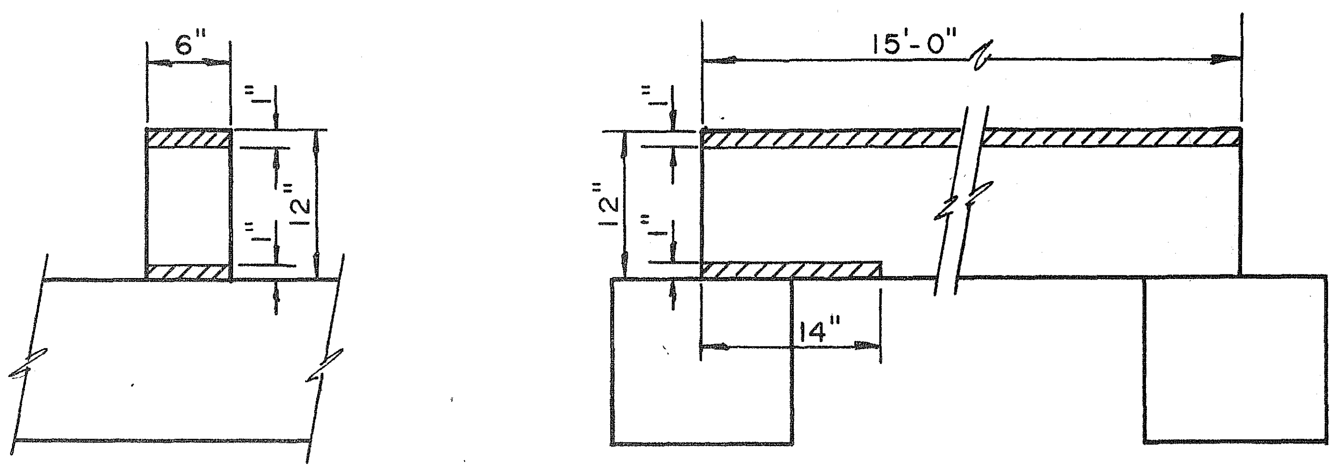
STRINGER DATA NOTES:

1. Deterioration of the stringer should be accounted for by using an average height x width of the sound material. If interior deterioration has occurred (the stringer is a "shell") the average exterior height x width along with the approximate interior height x width of deterioration and dimensions from center of deteriorated area to bottom of stringer should be recorded under "stringer cross-sectional dimensions".



SKETCH A

SPAN 2 - STRINGER No. 9



SKETCH B

SPAN 2 - STRINGER No. 10

2. The location of deteriorated sections as measured from the beginning of stringer (as defined by the direction of inspection) should be noted.

3. It should be specifically noted if a loss of stringer section has occurred near the end of the stringer. If no specific comments were noted in the Bridge Inspectors Report, it should be verified that the stringer has suffered no loss of section due to deterioration or crushing, as this condition will be significant in the horizontal shear rating of the stringer.

Material Strengths:

Allowable stress in bending (f_b) as determined by the Bridge

Engineer* = 2000 psi.

Allowable stress in shear (f_v) as determined by the Bridge

Engineer* = 90 psi.

Allowable bending stress at inventory rating level (f_{bi})

$= f_b =$ $\boxed{2000}$ psi.

Allowable bending stress at operating rating level (f_{bo})

$= 1.33\ f_b =$ $\boxed{2660}$ psi.

Allowable shear stress at inventory rating level (f_{vi})

$= f_v =$ $\boxed{90}$ psi

Allowable shear stress at operating rating level (f_{vo})

$= 1.33\ f_v =$ $\boxed{120}$ psi.

---

*Standard Specifications for Highway Bridges, Twelfth Edition, 1977; Interim Specifications Bridges 1978, 1979 and 1980; Section 1.10.1 and Table 1.10.1A will be used in determining the allowable stresses for timber members when, in the judgment of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the allowable stresses shall be fixed by a Registered Professional Engineer.

STEP 4 COMPUTE LIVE LOAD SHEARS AND MOMENTS

Load cases are same as those shown on Plate III in Appendix A, therefore, use the table on Plate I in Appendix A to obtain maximum live load moment for each loading case.

Live Load shear for each load case is computed by the method given in Section 3.3.

It is evident that by placing the SU2 load case on the span in the opposite direction only the 12 kip load would be on the span thus yielding a smaller Live Load Shear.

DATE	DESIGN	Section 4.3.2 - Example Computations	SHEET OF
	CHECK	JOB / FOR	JOB NO.

SUBJECT

Timber Deck And Stringer Superstructure

Span Length = 14.0'

Design Loading – Unknown

Live Load Moments – From Plate I

SU2 = 77.0 ft-k

SU3 = 111.6 ft-k

SU4 = 118.4 ft-k

C3 = 77.0 ft-k

C4 = 111.6 ft-k

C5 = 111.6 ft-k

H = 112.0 ft-k

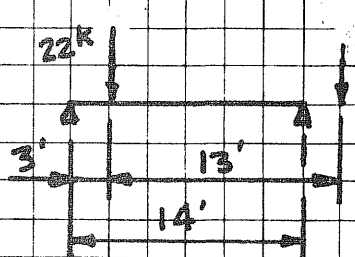
HS = 112.0 ft-k

Live Load Shear From Section 3.3

$X = 3\,(12\text{ in})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right) = 3.0\text{ ft.}$ ⇐ controls

OR $X = \frac{1}{4}(14.0) = 3.5\text{ ft.}$

Examine SU2 Load Case



$LLV = 22\left(1 - \frac{3}{14}\right) = \boxed{17.29^{k}}$

Note that by placing the SU3 load case on the span in the opposite direction, one 22 kip load would fall off the span and one 22 kip load would fall directly at the right support. This would leave only one 22 kip load on the span which clearly yields a smaller value for Live Load Shear than the placement shown on the following page.

Note that by placing the SU4 load case on the span in the opposite direction, only one 18.7 kip load and the 13.9 kip load would fall on the span, which would yield a smaller value for Live Load Shear than the placement shown on the following page.

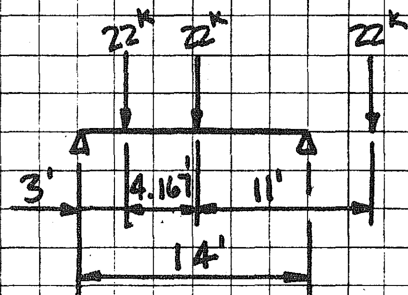
Note that by placing the C3 load case on the span in the opposite direction, the 12 kip load would fall 3 ft. from the left support and the 22 kip load would fall 1 ft. from the right support, which would yield a smaller value for Live Load Shear than the placement shown on the following page.

Note that by placing the C4 load case on the span in the opposite direction, only the 7.271 kip load and one 22 kip load would fall on the span, which would clearly yield a smaller value for the Live Load Shear than the placement shown on the following page.

DATE	DESIGN	Sec. 4.3.2 - Example Computations Cont.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

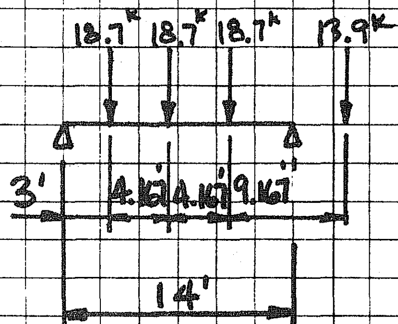
Examine SU3 Load Case



$$LLV = 22\left(1-\frac{3}{14}\right) + 22\left(1-\frac{3+4.167}{14}\right)$$

$$LLV = \boxed{28.02^k}$$

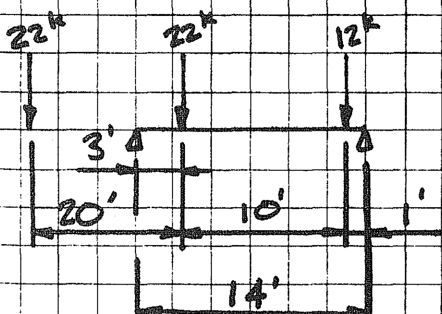
Examine SU4 Load Case



$$LLV = 18.7\left(1-\frac{3}{14}\right) + 18.7\left(1-\frac{3+4.167}{14}\right) + 18.7\left(1-\frac{3+4.167+4.167}{14}\right)$$

$$LLV = \boxed{27.38^k}$$

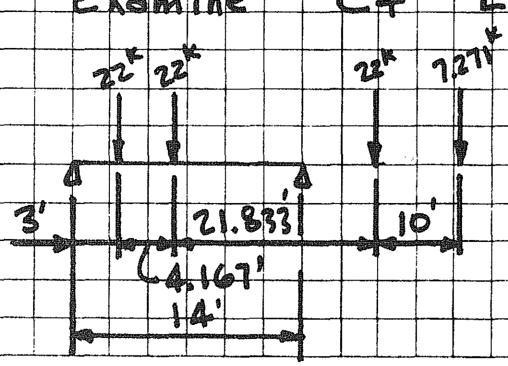
Examine C3 Load Case



$$LLV = 22\left(1-\frac{3}{14}\right) + 12\left(1-\frac{3+10}{14}\right)$$

$$LLV = \boxed{18.14^k}$$

Examine C4 Load Case



$$LLV = 22\left(1-\frac{3}{14}\right) + 22\left(1-\frac{3+4.167}{14}\right)$$

$$LLV = \boxed{28.02^k}$$

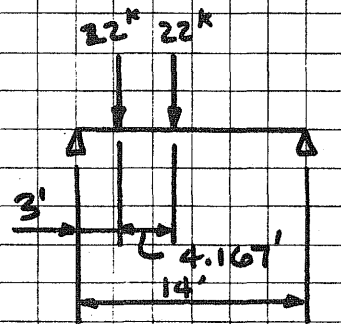
It is not possible to have more than two axles of the C5 load case on the span at one time, therefore, the only possible placement is shown on the following page.

Note it is possible to only have one axle on the span for both H and HS loading cases.

DATE	DESIGN	SEC. 4.3.2-EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

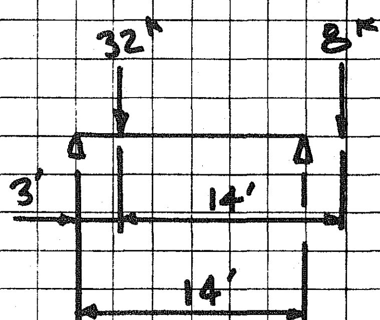
Examine C5 Load Case



$$LLV = 22\left(1-\frac{3}{14}\right) + 22\left(1-\frac{3+4.167}{14}\right)$$

$$LLV = \boxed{28.02^k}$$

Examine H Load Case (and HS Load Case)



$$LLV = 32\left(1-\frac{3}{14}\right) = \boxed{25.14^k}$$

Summary of Maximum Live Load Shears

SU2 = 17.29 Kips

SU3 = 28.02 Kips

SU4 = 27.38 Kips

C3 = 18.14 Kips

C4 = 28.02 Kips

C5 = 28.02 Kips

H = 25.14 Kips

HS = 25.14 Kips

STEP 5 PERFORM RATING CALCULATIONS

The rating calculations are made in accordance with the procedures given in Section 4.2

1. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR TIMBER FLOOR. Obtain a photocopy of the forms headed "Section 12.1 - Rating of Timber Flooring" and compute the Operating and Inventory Ratings by performing the calculations detailed on the forms.

DEAD LOAD consists of the weight of the members comprising the structure. In this case the DEAD LOAD is the weight of the flooring itself plus the weight of any wearing surface.

SECTION 4.3.3 RATING OF TIMBER FLOORING

Compute Dead Load (DL) per foot of flooring:

Asphalt Wearing Surface = (Thickness)(144 lb/ft^3)(1/12)
= (1 in.)(144 lb/ft^3)(1/12) = 12 lb/ft

Concrete Wearing Surface = (Thickness)(150 lb/ft^3)(1/12)
= (— in.)(150 lb/ft^3)(1/12) = — lb/ft

Timber Floor = (Thickness)(50 lb/ft^3)(1/12)
(3.5 in.)(50 lb/ft^3)(1/12) = 14.58 lb/ft

Total DL = $\boxed{26.58}$ lb/ft

Determine Effective Deck Span(s):

s = Stringer Spacing ($\mathcal{C}_L$ to $\mathcal{C}_L$)
s = 2.25 ft.

but not greater than:

(Stringer Spacing - Stringer Width)+(Floor Thickness*)
(2.25 - .5)ft. + (3.5 in. x 1/12) = 2.04 ft.

So, s = $\boxed{2.04}$ ft.

---

*not including wearing surface

The Floor Thickness (T_F) used should be the effective Floor Thickness (ie. taking into account deterioration). In this case we have a 3-1/2" deck with 1/2" deterioration, yielding an effective Floor Thickness of 3".

The APPLIED DEAD LOAD MOMENT is the force resulting from the weight of the components making up the structure (in this case the weight of the timber flooring and the wearing surfaces).

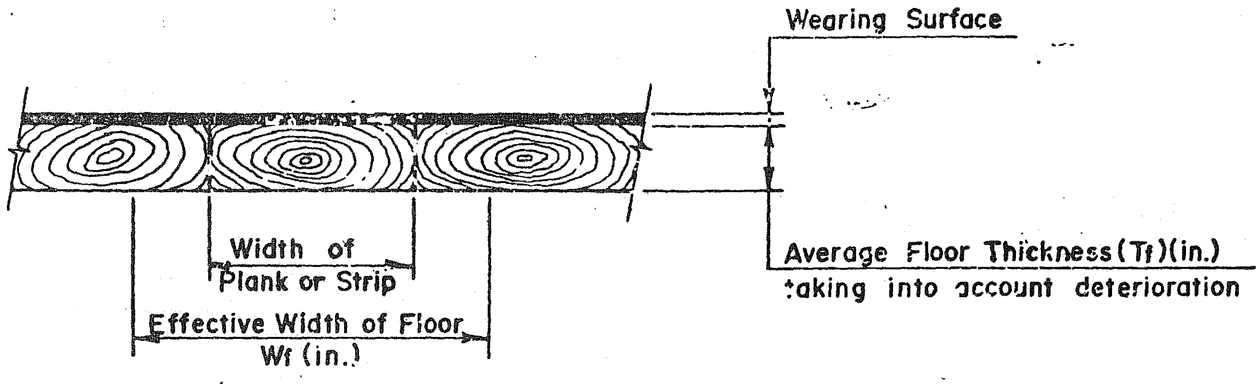


Figure 4-3

Determine Applied Dead Load Moment (M_{DL}):

M_{DL} = (Total DL)$(s)^2(1/8)$
M_{DL} = (26.58lb/ft.)(2.04ft.)^2(1/8) = $\boxed{13.83}$ ft-lbs

Determine distribution of Wheel Loads on Flooring Parallel to Bridge:

W_F = Effective Width of floor to be considered to support wheel load

for Strip Floor W_F = 4 x (Floor Thickness*) but not less than 5-1/2 "

W_F = 4 x (—)in. = — (in.)

for Plank Floor W_F = width of plank = 12 (in.)

So, W_F = $\boxed{12}$ (in.)

If W_F comes out less than 5-1/2, use 5-1/2

Calculate Section Modulus (Z):

$$Z = \frac{W_F \times (T_F)^2}{6}$$

$$Z = \frac{(12 \text{ in})(3 \text{ in})^2}{6} = \boxed{18.0} \text{ in.}^3$$

* Not including wearing surface

The INVENTORY RESISTING MOMENT is the Moment Capacity of the member where stresses do not exceed those specified for Inventory Rating.

The OPERATING RESISTING MOMENT is the Moment Capacity of the member where stresses do not exceed those specified for Operating Rating.

RESISTING MOMENT AVAILABLE FOR LIVE LOAD is the Moment Capacity of the member (Inventory or Operating resisting moment) less the Dead Load Moment.

Calculate Resisting Moments:

Inventory Resisting Moment $(MR_i) = \frac{f_{bi} \times Z}{12} \times$ (continuity factor)*

$$= \frac{(2000 lb/in^2)(18.0 in^3)}{12} (1.25) = \boxed{3750} \text{ ft-lbs.}$$

Operating Resisting Moments $(MR_o) = \frac{f_{bo} \times Z}{12} \times$ (continuity factor)*

$$= \frac{(2660 lb/in^2)(18.0 in^3)}{12} (1.25) = \boxed{4988} \text{ ft-lbs.}$$

*Note: If the flooring is continuous over stringers (i.e., the floor timbers span 3 or more stringers) the continuity factor is 1.25. If the flooring is not continuous over stringers (i.e., the floor timbers span between 2 stringers only) the continuity factor is 1.0.

Determine Resisting Moments Available for Live Load:

Available Live Load Inventory Moment

$$(MLLi) = (MR_i) - (M_{DL})$$
$$(MLLi) = (3750) - (13.83) = \boxed{3736.17} \text{ ft-lbs.}$$

Available Live Load Operating Moment

$$(MLLo) = (MR_o) - (M_{DL})$$
$$(MLLo) = (4988) - (13.83) = \boxed{4974.17} \text{ ft-lbs.}$$

AVAILABLE LIVE LOAD PER WHEEL is the wheel load (or 1/2 the axle load) that will produce a moment equal to the resisting moment available for live load.

P_i & P_o = available Live Load per wheel for Inventory and Operating levels (kips)

Max. axle load = maximum weight set of axles for the loading type under consideration (from Section 3.2)(kips)

GVW = Gross Vehicle Weight (from Section 3.2) (kips)

Determine Available Live Load per Wheel (P):

For Inventory Rating: $P_i = \frac{4 \times (MLLi)}{(s)(1000)}$

$$P_i = \frac{(4)(3736.17)}{(2.04)(1000)} = \boxed{7.33} \text{ Kips}$$

For Operating Rating: $P_o = \frac{4 \times (MLLo)}{(s)(1000)}$

$$P_o = \frac{(4)(4974.17)}{(2.04)(1000)} = \boxed{9.75} \text{ Kips}$$

Calculate Inventory and Operating Ratings:

The formula to be used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{Pi}{1/2(\text{Max. Axle Load})}(GVW)$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{Po}{1/2\ (\text{Max. Axle Load})}(GVW)$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 3.2 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Floor Rating".

C5: Since the C5 load case used to evaluate the Live Load Shear consisted of the two trailer axles on the span, use the max. axle load of 22 kips and the GVW of 80 kips.

CALCULATE INVENTORY AND OPERATING RATINGS FOR DECK

SU2: Inv. $\frac{7.33}{(1/2)(22.0)}(34.0) = 22.7$ Kips

Op. $\frac{9.75}{(1/2)(22.0)}(34.0) = 30.1$ Kips

SU3: Inv. $\frac{7.33}{(1/2)(22.0)}(66.0) = 43.9$ Kips

Op. $\frac{9.75}{(1/2)(22.0)}(66.0) = 58.5$ Kips

SU4: Inv. $\frac{7.33}{(1/2)(18.7)}(70.0) = 54.9$ Kips

Op. $\frac{9.75}{(1/2)(18.7)}(70.0) = 73.0$ Kips

C3: Inv. $\frac{7.33}{(1/2)(22.0)}(56.0) = 37.3$ Kips

Op. $\frac{9.75}{(1/2)(22.0)}(56.0) = 49.6$ Kips

C4: Inv. $\frac{7.33}{(1/2)(22.0)}(73.271) = 48.8$ Kips

Op. $\frac{9.75}{(1/2)(22.0)}(73.271) = 64.9$ Kips

C5: Inv. $\frac{7.33}{(1/2)(22.0)}(80.0) = 53.3$ Kips

Op. $\frac{9.75}{(1/2)(22.0)}(80.0) = 70.9$ Kips

H: Inv. $\frac{7.33}{(1/2)(32.0)}(40.0) = 18.3$ Kips

Op. $\frac{9.75}{(1/2)(32.0)}(40.0) = 24.4$ Kips

DATE	DESIGN	SEC. 4.33-EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

HS : INV. $\dfrac{7.33}{(1/2)(32.0)}(72.0) = 33.0$ Kips

Op. $\dfrac{9.75}{(1/2)(32.0)}(72.0) = 43.9$ Kips

This completes the rating of the timber floor. The Rating Values are summarized in the Summary of Rating Table at the end of this section.

STEP 5 2. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR MOMENT CAPACITY FOR EXTERIORS STRINGERS.

By examining the Timber Deck and Stringer Superstructure Inspection Data Sheet, it is determined that each exterior stringer is of similar size, spacing, and loading condition; but one particular exterior stringer (no. 10 in span 2) is clearly more critical than the other stringers due to deterioration in the middle one-half of the member. This stringer is the obvious candidate for the moment capacity rating.

A photocopy of the forms "Section 12.1 - Moment Capacity Rating of an Exterior Stringer" should be obtained and the Operating and Inventory Ratings determined by performing the computations detailed on the forms.

The intent of this DEAD LOAD computation is to calculate the weight per linear foot of barrier and sum it with the weight per linear foot of the deck and stringer to obtain the total dead load per foot of stringer. The barrier configuration illustrated is a general configuration consisting of a wheel guard, rail, and rail posts. Actual configurations of existing barriers may vary from this.

For barriers constructed of timber that differ in configuration from the barrier indicated, the average barrier area should be calculated and a unit weight of 50 lbs. per cubic foot should be used to compute the weight per linear foot of the barrier. This weight per linear foot should then be applied to the exterior stringer as dead load. For barriers employing steel, the area per linear foot should be computed and a unit weight of 490 lbs/ft.^{3} should be used. If standard rolled steel sections are used, the weights per linear foot may be obtained from the AISC manual.* For aluminum, a unit weight of 175 lbs/ft^3 is to be used and for concrete, a unit weight of 150 lbs/ft.^{3} is to be used.

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* American Institute of Steel Construction, Inc., Manual of Steel Construction, Part I

SECTION 4.3.4 - MOMENT CAPACITY RATING OF AN EXTERIOR STRINGER

Compute Dead Load DL per foot of Stringer:

Rail: (Number of Rails)(Width)(Height)($50\#/ft^3$)(1/144)

= (1)(4 in.)(4 in.)($50\#/ft^3$)(1/144) =

5.56 lb/ft.

Rail Posts: (Width)(Height)(Length)$\left(\frac{1}{\text{Post Spacing}}\right)$($50\#/ft^3$)(1/144)

= (4 in.)(6 in.)(2.5 ft.)(1/6.25 ft.)($50\#/ft.^3$)(1/144) =

3.33 lb/ft.

Wheel Guard: (Width)(Height)($50\#/ft^3$)(1/144)

= (4 in.)(6 in.)($50\#/ft^3$)(1/144) =

8.33 lb/ft.

s = 1/2 (spacing between exterior stringer & first interior stringer)

s = 1.125 ft.

Wearing Surface:

asphalt: (Thickness)(S)($144/ft^3$)(1/12)
(1 in.)(1.125 ft.)($144/ft^3$)(1/12) = 13.50 lb/ft.

concrete: (Thickness)(S)($150\#/ft^3$)(1/12)
(— in.)(— ft.)($150\#/ft^3$)(1/12) = — lb/ft.

Timber Deck: (Thickness)(S)($50\#/ft^3$)(/12)
(3.5 in.)(1.125 ft.)($50\#/ft^3$)(1/12) = 16.41 lb/ft.

Stringer: (Width)(Height)($50\#/ft^3$)(1/144)
(6 in.)(12 in.)($50\#/ft^3$)(1/144) = 25.00 lb/ft.

Total DL per of foot stringer (W_{DL}) = $\boxed{72.13}$ lb/ft.

The APPLIED DEAD LOAD MOMENT is the moment resulting from the computed dead load per foot of stringer.

The SECTION MODULUS is the ratio of the moment of inertia of a member to the distance between the neutral axis and the outer fiber of the member.

CASE I:

No Interior Deterioration

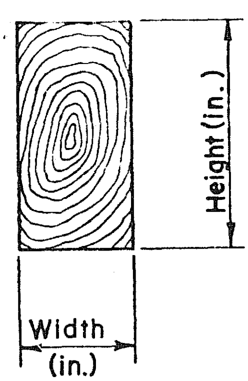

Figure 4-4

CASE II:

Interior Deterioration Present

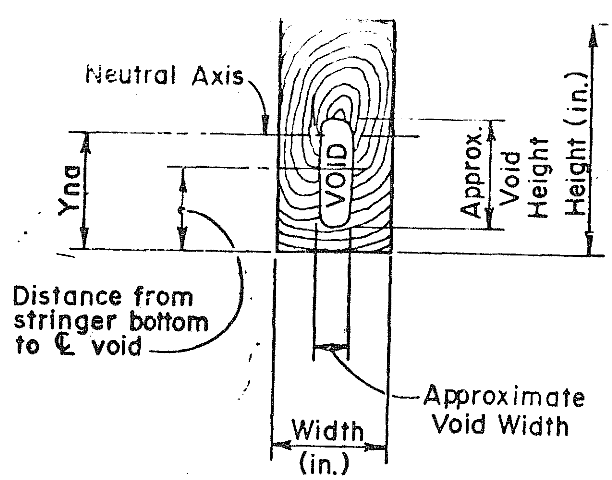

Figure 4-5

Calculate Applied Dead Load Moment (M_{DL})

M_{DL} = (1/8)(W_{DL})(Span Length)^2

= (1/8)(72.13 lb/ft.)(14.0 ft.)^2 = $\boxed{1767.2}$ ft.lb.

Compute Section Modulus (Z) of Exterior Stringer:

Due to the nature of deterioration in timber members, two conditions may exist; deterioration of the surfaces of the member, and internal deterioration. The computation of the section modulus is different for the two conditions, therefore, the deterioration condition should be determined and the section modulus calculated using the appropriate case below. Where no deterioration is present, use Case I.

CASE I: No interior deterioration. Height and width are measurements of the "sound" material, taking into account deterioration. The section modulus for the top of the stringer equals the section modulus for the bottom of the stringer. (i.e. $Z_{top} = Z_{bottom} = Z$).

$$Z = \frac{(\text{Width})(\text{Height})^2}{6}$$

$$Z = \frac{(\ 6 \text{ in.})(\ 11 \text{ in.})^2}{6} = \boxed{121.0} \text{ in.}^3$$

CASE II: Interior deterioration is present. Void size is to be approximated and represented by a void height, void width, and the distance from the center of the void to the bottom of the stringer. Section modulus will be calculated for the "shell" of sound material.

$$Y_{na} = \frac{(1/2)(\text{Width})(\text{Height})^2 - (\text{Void Width})(\text{Void Height})(\text{Dist. bot. stringer to } \mathrm{C\!\!\!L} \text{ Void})}{(\text{Width})(\text{Height}) - (\text{Void Width})(\text{Void Height})}$$

$$Y_{na} = \frac{(1/2)(\ - \text{ in.})(\ - \text{ in.})^2 - (\ - \text{ in.})(\ - \text{ in.})(\ - \text{ in.})}{(\ - \text{ in.})(\ - \text{ in.}) - (\ - \text{ in.})(\ - \text{ in.})}$$

Y_{na} = ______ – ______ in.

CASE II (Continued)

$$I_o \text{ Stringer} = \frac{(\text{Width})(\text{Height})^3}{12}$$

$$= \frac{(- \text{ in.})(- \text{ in.})^3}{12} = \underline{\quad - \quad} \text{in.}^4$$

$$I_o \text{ Void} = \frac{(\text{Void Width})(\text{Void Height})^3}{12}$$

$$= \frac{(- \text{ in.})(- \text{ in.})^3}{12} = \underline{\quad - \quad} \text{in.}^4$$

$$\overline{Y} \text{ Stringer} = Y_{na} - 1/2 \text{ (height)}$$

$$= (- \text{ in.}) - 1/2 (- \text{ in.}) = + \underline{\quad - \quad} \text{ in.*}$$

$$\overline{Y} \text{ Void} = Y_{na} - \text{(dist. bot. stringer to } \mathcal{C}\!\!\!L \text{ Void)}$$

$$= (- \text{ in.}) - (- \text{ in.}) = + \underline{\quad - \quad} \text{ in.*}$$

$$I_{na} = (I_o \text{ Stringer}) - (I_o \text{ Void}) + (\text{Width})(\text{Height})(\overline{Y} \text{ Stringer})^2$$
$$- (\text{Void Width})(\text{Void Height})(\overline{Y} \text{ Void})^2$$

$$= (- \text{ in}^4) - (- \text{ in.}^4) + (- \text{ in.})(- \text{ in.})(- \text{ in.})^2$$
$$- (- \text{ in.})(- \text{ in.})(- \text{ in.})^2 = \underline{\quad - \quad} \text{ in.}^4$$

---

* If negative, the negative sign should be dropped. Both values should be positive.

The INVENTORY RESISTING MOMENT is the moment capacity of the member where stresses do not exceed those specified for Inventory Rating.

The OPERATING RESISTING MOMENT is the moment capacity of the member where stresses do not exceed those specified for Operating Rating.

CASE II (Continued)

$$Z_{bottom} = \frac{I_{na}}{Y_{na}} = \frac{\text{— in.}^4}{\text{— in.}} = \boxed{\text{—}} \text{ in.}^3$$

$$Z_{top} = \frac{I_{na}}{(\text{Height} - Y_{na})} = \frac{\text{— in.}^4}{(\text{— in.} - \text{— in.})} = \boxed{\text{—}} \text{ in.}^3$$

Calculate Resisting Moments:

CASE I: No interior deterioration.

$$\text{Inventory Resisting Moment } (MR_i) = \frac{(f_{bi})(Z)}{12}$$

$$= \frac{(2000\text{psi})(121.0\text{in}^3)}{12} = \boxed{20167} \text{ ft-lbs.}$$

$$\text{Operating Resisting Moment } (MR_o) = \frac{(f_{bo})(Z)}{12}$$

$$= \frac{(2660\text{psi})(121.0\text{in}^3)}{12} = \boxed{26822} \text{ ft-lbs.}$$

CASE II: Interior deterioration is present.

Inventory Resisting Moment (MR_i) =

$$\text{smaller of } \frac{(f_{bi})(Z_{top})}{12} \text{ or } \frac{(f_{bi})(Z_{bottom})}{12}$$

$$= \frac{(\text{— psi})(\text{— in}^3)}{12} = \boxed{\text{—}} \text{ ft-lbs.}$$

Use $\boxed{\text{—}}$ ft-lbs.

$$= \frac{(\text{— psi})(\text{— in}^3)}{12} = \boxed{\text{—}} \text{ ft-lbs.}$$

Operating Resisting Moment (MR_o) =

$$\text{smaller of } \frac{(f_{bo})(Z_{top})}{12} \text{ or } \frac{(f_{bo})(Z_{bottom})}{12}$$

$$= \frac{(\ -\ \text{lb/in}^2)(\ -\ \text{in.}^3)}{12} = \underline{\quad - \quad} \text{ ft-lbs.}$$

use $\boxed{\quad - \quad}$ ft-lbs.

$$= \frac{(\ -\ \text{lb/in}^2)(\ -\ \text{in.}^3)}{12} = \underline{\quad - \quad} \text{ ft-lbs.}$$

Determine Resisting Moments Available for Live Load:

Available Capacity for Live Load Inventory Moment

$(MLL_i) = (MR_i) - (M_{DL})$

$(MLL_i) = (20167) - (1767) = \boxed{18400}$ ft-lbs.

Available Capacity for Live Load Operating Moment

$(MLL_o) = (MR_o) - (M_{DL})$

$(MLL_o) = (26822) - (1767) = \boxed{25\ 055}$ ft-lbs.

RESISTING MOMENT AVAILABLE FOR LIVE LOAD is the Moment Capacity of the member (Inventory or Operating resisting moment) less the Dead Load Moment.

s = One-half of the spacing between exterior stringer and first interior stringer, from Sec. 4.3.4 under computation of DL per foot of stringer.

Calculate Inventory and Operating Moment Ratings:

CASE I: Sidewalk at Exterior Stringer

Available Live Load Capacity per ft. at Inventory Level Rating = (W_i)

$$(W_i) = \frac{8\ (MLL_i)}{(\text{Des. Span Length})^2}$$

$$= \frac{8(\ \text{—}\ \text{ft-lbs})}{(\ \text{—}\ \text{ft.})^2} = \boxed{\ \text{—}\ }\ \text{lb/ft.}$$

Available Live Load Capacity per ft. at Operating Rating = (W_o)

$$(W_o) = \frac{8\ (MLL_o)}{(\text{Des. Span Length})^2}$$

$$= \frac{8(\ \text{—}\ \text{ft-lbs})}{(\ \text{—}\ \text{ft.})^2} = \boxed{\ \text{—}\ }\ \text{lb/ft.}$$

Inventory Rating =

$$\frac{W_i}{s} = \frac{(\ \text{—}\ \text{lb/ft})}{(\ \text{—}\ \text{ft})} = \boxed{\ \text{—}\ }\ \text{psf}$$

Operating Rating =

$$\frac{W_o}{s} = \frac{(\ \text{—}\ \text{lb/ft})}{(\ \text{—}\ \text{ft})} = \boxed{\ \text{—}\ }\ \text{psf}$$

These Inventory and Operating ratings are the same for all loading types and should be entered in each row under "Exterior Stringer (Moment)" on the "Summary of Ratings" table in Chapter 8.

The distance from the centerline of the exterior stringer to the centerline of wheel load is to be computed for the centerline of wheel load placed 2 feet from the face of the wheel guard. Given the condition shown in Figure 4-6, the equation for x_1 is shown on the following page. If the outer face of the exterior stringer and wheelguard do not line up, x_1 should be determined consistent with the above intent.

M_{LLi} & M_{LLo} = Available Live Load Moment capacity for Inventory and Operating levels (ft-lbs)

LLM = Applied Live Load Moment (from Section 3.2 for each loading type)(ft-Kips)

GVW = Gross Vehicle Weight (from Section 3.2 for each loading type)(Kips)

DF = Distribution Factor.

CASE II: No sidewalk at exterior stringer.

Determine distance from centerline of exterior stringer to centerline of wheel load (x_1):

x_1 = 2' + Wheel Guard Width - 1/2 stringer width (only if outer face of stringer and wheel guard line up).

x_1 = 2' + (4 in)(1/12) - (1/2)(6 in.)(1/12)

x_1 = 2.083 ft.

Determine Distribution Factor (DF):

$$DF = \frac{\text{Stringer Spacing} - x_1}{\text{Stringer Spacing}}$$

$$DF = \frac{(2.25 \text{ ft.}) - (2.083 \text{ ft.})}{(2.25 \text{ ft.})} = 0.074$$

Calculate Inventory and Operating Ratings:

The formula to be used to calcualte the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(M_{LLi})(GVW)}{(LLM)(1/2)(DF)(1000)}$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{(M_{LLo})(GVW)}{(LLM)(1/2)(DF)(1000)}$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 2.2 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Ext. Stringer (Moment)".

C5 Load Case: Note on Plate III in Appendix A that for a 14 ft. span two axles are used on the span; therefore, a GVW equaling 80 kips is used.

CALCULATE INVENTORY AND OPERATING RATINGS FOR EXTERIOR STRINGER (MOMENT):

SU2 : INV. $\dfrac{(18400)(34.0)}{(77.0)(1/2)(0.074)(1000)} =$ 219.6 Kips

Op. $\dfrac{(25055)(34.0)}{(77.0)(1/2)(0.074)(1000)} =$ 299.0 Kips

SU3 : INV. $\dfrac{(18400)(66.0)}{(111.6)(1/2)(0.074)(1000)} =$ 294.1 Kips

Op. $\dfrac{(25055)(66.0)}{(111.6)(1/2)(0.074)(1000)} =$ 400.5 Kips

SU4 : INV. $\dfrac{(18400)(70.0)}{(118.4)(1/2)(0.074)(1000)} =$ 294.0 Kips

Op. $\dfrac{(25055)(70.0)}{(118.4)(1/2)(0.074)(1000)} =$ 400.3 Kips

C3 : INV. $\dfrac{(18400)(56.0)}{(77.0)(1/2)(0.074)(1000)} =$ 361.7 Kips

Op. $\dfrac{(25055)(56.0)}{(77.0)(1/2)(0.074)(1000)} =$ 492.5 Kips

C4 : INV. $\dfrac{(18400)(73.271)}{(111.6)(1/2)(0.074)(1000)} =$ 326.5 Kips

Op. $\dfrac{(25055)(73.271)}{(111.6)(1/2)(0.074)(1000)} =$ 444.6 Kips

C5 : INV. $\dfrac{(18400)(80.0)}{(111.6)(1/2)(0.074)(1000)} =$ 356.5 Kips

Op. $\dfrac{(25055)(80.0)}{(111.6)(1/2)(0.074)(1000)} =$ 485.4 Kips

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H : Inv. $\dfrac{(18400)(40.0)}{(112.0)(1/2)(0.074)(1000)} =$ 177.6 Kips

Op. $\dfrac{(25055)(40.0)}{(112.0)(1/2)(0.074)(1000)} =$ 241.8 Kips

HS : Inv. $\dfrac{(18400)(72.0)}{(112.0)(1/2)(0.074)(1000)} =$ 319.7 Kips

Op. $\dfrac{(25055)(72.0)}{(112.0)(1/2)(0.074)(1000)} =$ 435.3 Kips

This completes the Moment Capacity Rating for the exterior stringer. The Rating Values are summarized in the Summary of Ratings Table at the end of this section.

STEP 5 3. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR SHEAR CAPACITY FOR EXTERIOR STRINGERS.

By examining the Timber Deck and Stringer Superstructure Inspection Data Sheet it is determined that each exterior stringer is of similar size, spacing, and loading condition; but one particular exterior (No. 10 in span 2) stringer is clearly more critical than the other stringers due to deterioration in the outer one-quarter of the member. This stringer is then the obvious candididate for the Moment Capacity Rating.

A photocopy of the forms "Section 12.1 - Shear Capacity Rating of an Exterior Stringer" should be obtained and the Operating and Inventory Ratings are to be determined by performing the computations detailed on the forms.

APPLIED DEAD LOAD HORIZONTAL SHEAR is the shear resulting from the computed dead load per foot of stringer.

W_{DL} = Total DL per foot of stringer from Section 4.3.4.

x = Distance from support to nearest axle (point where V_{DL} is to be computed) from Section 3.3

The Stringer Width and Stringer Height used should take deterioration into account by averaging the width and height of sound material. If interior deterioration is present (the Stringer is a "shell") the void area (average void width multiplied by average void height) should be deducted from the average stringer area (stringer width multiplied by stringer height)

The INVENTORY RESISTING HORIZONTAL SHEAR is the horizontal shear capacity of the member where stresses do not exceed those specified for Inventory Rating.

The OPERATING RESISTING HORIZONTAL SHEAR is the horizontal shear capacity of the member where stresses do not exceed those specified for Operating Rating.

SECTION 4.3.5 - SHEAR CAPACITY RATING OF AN EXTERIOR STRINGER

Calculate Applied Dead Load Horizontal Shear (V_{DL})

$V_{DL} = W_{DL}[(\text{Span Length})(1/2) - (x)]$

$V_{DL} = (\ 72.13\ \text{lb/ft.})[(\ 14\ \text{ft.})(1/2) - (\ 3.0\ \text{ft.})]$

$V_{DL} = \boxed{288.52}$ lbs.

Calculate Resisting Horizontal Shear:

Inventory Resisting Horizontal Shear = (VR_i)

$VR_i = (2/3)(\text{Stringer Width})(\text{Stringer Height})(f_{vi})$

$VR_i = (2/3)(\ 6\ \text{in.})(\ 10\ \text{in.})(\ 90\ \text{psi.}) = \boxed{3600}$ lbs.

Operating Resisting Horizontal Shear = (VR_o)

$VR_o = (2/3)(\text{Stringer Width})(\text{Stringer Height})(f_{vo})$

$VR_o = (2/3)(\ 6\ \text{in.})(\ 10\ \text{in.})(\ 120\ \text{psi.}) = \boxed{4800}$ lbs.

RESISTING HORIZONTAL SHEAR AVAILABLE FOR LIVE LOAD is the horizontal shear capacity of the member (Inventory or Operating) less the Applied Dead Load horizontal shear.

V_{LLi} & V_{LLo} = Available Live Load Shear capacity for Inventory and Operating levels (lbs.)

LLV = Applied Live Load Shear (from Section 3.3 for each loading type)(kips)

GVW = Gross Vehicle Weight (from Section 3.3 for each loading type)(kips)

DF = Distribution Factor from Section 4.3.6

Determine Resisting Horizontal Shear Available for Live Load:

Available Live Load Inventory Horizontal Shear = (V_{LLi})

$$(V_{LLi}) = (VR_i) - (V_{DL})$$

$$(V_{LLi}) = (3600\text{lbs.}) - (288.5\text{lbs.}) = \boxed{3311.5}\ \text{lbs.}$$

Available Live Load Operating Horizontal Shear = (V_{LLo})

$$(V_{LLo}) = (VR_o) - (V_{DL})$$

$$(V_{LLo}) = (4800\text{lbs.}) - (288.5\text{lbs.}) = \boxed{4511.5}\ \text{lbs.}$$

Calculate Inventory and Operating Horizontal Shear Ratings:

CASE I: Sidewalk at Exterior Stringer

Available Live Load Capacity per ft. at Inventory Rating (W_i)

$$(W_i) = \frac{2(V_{LLi})}{(\text{Span Length})}$$

$$(W_i) = \frac{2(\text{ — lbs})}{(\text{ — ft.})} = \boxed{\text{—}}\ \text{lb/ft.}$$

Available Live Load capacity per ft. at Operating Rating (W_o)

$$(W_o) = \frac{2(V_{LLo})}{(\text{Span Length})}$$

$$(W_o) = \frac{2(\text{ — lbs})}{(\text{ — ft.})} = \boxed{\text{—}}\ \text{lb/ft.}$$

Inventory Rating

$$\frac{2\ (W_i)}{(\text{Stringer Spacing})} = \frac{(\ \text{—}\ \text{lb/ft})}{(\ \text{—}\ \text{ft.})} = \boxed{\text{—}}\ \text{lb/ft}^2$$

Operating Rating:

$$\frac{2\ (W_o)}{(\text{Stringer Spacing})} = \frac{(\ \text{—}\ \text{lb/ft})}{(\ \text{—}\ \text{ft.})} = \boxed{\text{—}}\ \text{lb/ft}^2$$

These Inventory and Operating Ratings are the same for all loading types and should be entered in each row under "Ext. Stringer Horizontal Shear" on the "Summary of Ratings" table in Chapter 8.

CASE II: No Sidewalk at Exterior Stringer

Calculate Inventory and Operating Ratings:

the formula to be used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(V_{LLi})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{(V_{LLo})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 3.3 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Ext. Stringer (Horiz. Shear)".

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Calculate Inventory And Operating Ratings For Exterior Stringer (Horiz. Shear)

SU2 : Inv. $\dfrac{(3311.5)(34.0)}{(1/4)[(.6)(17.29)+(.074)(17.29)](1000)} = 38.6\text{ Kips}$

Op. $\dfrac{(4511.5)(34.0)}{(1/4)[(.6)(17.29)+(.074)(17.29)](1000)} = 52.7\text{ Kips}$

SU3 : Inv. $\dfrac{(3311.5)(66.0)}{(1/4)[(.6)(28.02)+(.074)(28.02)](1000)} = 46.3\text{ Kips}$

Op. $\dfrac{(4511.5)(66.0)}{(1/4)[(.6)(28.02)+(.074)(28.02)](1000)} = 63.1\text{ Kips}$

SU4 : Inv. $\dfrac{(3311.5)(70.0)}{(1/4)[(.6)(27.38)+(.074)(27.38)](1000)} = 50.2\text{ Kips}$

Op. $\dfrac{(4511.5)(70.0)}{(1/4)[(.6)(27.38)+(.074)(27.38)](1000)} = 68.5\text{ Kips}$

C3 : Inv. $\dfrac{(3311.5)(56.0)}{(1/4)[(.6)(18.14)+(.074)(18.14)](1000)} = 60.7\text{ Kips}$

Op. $\dfrac{(4511.5)(56.0)}{(1/4)[(.6)(18.14)+(.074)(18.14)](1000)} = 82.7\text{ Kips}$

C4 : Inv. $\dfrac{(3311.5)(73.271)}{(1/4)[(.6)(28.02)+(.074)(28.02)](1000)} = 51.4\text{ Kips}$

Op. $\dfrac{(4511.5)(73.271)}{(1/4)[(.6)(28.02)+(.074)(28.02)](1000)} = 70.0\text{ Kips}$

C5 : Inv. $\dfrac{(3311.5)(80)}{(1/4)[(.6)(28.02)+(.074)(28.02)](1000)} = 56.1 \text{ Kips}$

Op. $\dfrac{(4511.5)(80)}{(1/4)[(.6)(28.02)+(.074)(28.02)](1000)} = 76.4 \text{ Kips}$

H : Inv. $\dfrac{(3311.5)(40)}{(1/4)[(.6)(25.14)+(.074)(25.14)](1000)} = 31.3 \text{ Kips}$

Op. $\dfrac{(4511.5)(40)}{(1/4)[(.6)(25.14)+(.074)(25.14)](1000)} = 42.6 \text{ Kips}$

HS : Inv. $\dfrac{(3311.5)(72)}{(1/4)[(.6)(25.14)+(.074)(25.14)](1000)} = 56.3 \text{ Kips}$

Op. $\dfrac{(4511.5)(72)}{(1/4)[(.6)(25.14)+(.074)(25.14)](1000)} = 76.7 \text{ Kips}$

Note that two axles of the C5 loading case were on the span to compute the live load shear, therefore a GVW of 80 kips is used.

This completes the horizontal shear capacity rating of the exterior stringer. The rating values are summarized in the Summary of Ratings Table at the end of this section.

STEP 5

4. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR MOMENT CAPACITY FOR INTERIOR STRINGERS.

By examining the Timber Deck and Stringer Superstructure Inspection Data Sheet it is determined that all interior stringers are of similar size and spacings and no interior stringers have deterioration in the middle one-half of the span. It therefore is necessary to choose a representative stringer to be evaluated for moment capacity. Stringer no. 9 in Span 2 is chosen since it has deterioration in the outer one-quarter of the span and will need to be rated for shear capacity in the next section.

When interior stringers are of varying size, spacing, or have varying degrees of deterioration it is not always apparent which interior stringer is most critical. The most likely candidates for the more critical rating value should be selected. These are the stringers with the least cross-sectional area, the stringer having the greatest distance between adjacent stringers, the most severely deteriorated stringer, or other stringers showing signs of distress. Each of these stringers must be evaluated to determine the controlling rating.

A photocopy of the forms "Section 12.1 - Moment Capacity Rating of an Interior Stringer" should be obtained and the Operating and Inventory Ratings are to be determined by performing the computations detailed on the forms.

The DEAD LOAD is the weight of the stringer itself, plus the weight of the timber floor, plus the weight of the surfacing.

The APPLIED DEAD LOAD MOMENT is the moment resulting from the weight of the components making up the structure (in this case the weight of the stringer, the timber deck, and the wearing surface).

SECTION 4.3.6 - MOMENT CAPACITY RATING OF AN INTERIOR STRINGER

Compute Dead Load (DL) per foot of Stringer:

s = 1/2[(Spacing of Stringer to Right) + (Spacing of Stringer to Left)]

s = 1/2[(2.25 ft.) + (2.25 ft.)] = 2.25 ft.

wearing Surface:

Asphalt: (Thickness)(S)(144 lb/ft^3)(1/12)

= (1 in.)(2.25 ft.)(144 lb/ft^3)(1/12) = 27.00 lb/ft.

Concrete: (Thickness)(S)(150 lb/ft^3)(1/12)

= (— in.)(— ft.)(150 lb/ft^3)(1/12) = — lb/ft.

Timber Deck: (Thickness)(S)(50 lb/ft^3)(1/12)

= (3.5 in.)(2.25 ft.)(50 lb/ft^3)(1/12) = 32.81 lb/ft.

Stringer: (Width)(Height)(50 lb/ft^3)(1/144)

= (6 in.)(12 in.)(50 lb/ft^3)(1/144) = 25.00 lb/ft.

Total DL per ft. stringer (W_{DL}) = $\boxed{84.81}$ lb/ft.

Calculate Applied Dead Load Moment (M_{DL}):

M_{DL} = (1/8)(W_{DL})(Span Length)^2

= (1/8)(84.81 lb/ft)(14 ft.)^2 = $\boxed{2078}$ ft-lbs.

The SECTION MODULUS is the ratio of the moment of inertia of a member to the distance between the neutral axis and the outer fiber of the member.

CASE I
No Interior Deterioration

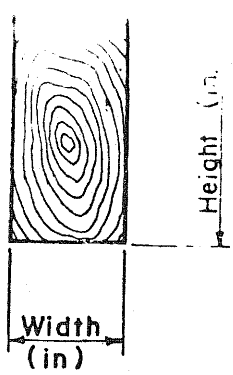

Figure 4-7

CASE II
Interior Deterioration Present

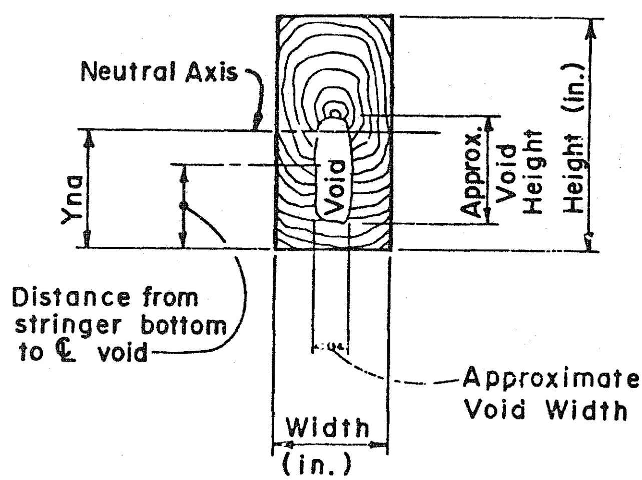

Figure 4-8

Compute Section Modulus (Z) of Interior Stringer:

Due to the nature of deterioration in timber members, two conditions may exist; deterioration of the surfaces of the member, and internal deterioration. The computation of the section modulus is different for the two conditions, therefore the deterioration condition should be determined and the section modulus calculated using the appropriate case below. Where no interior deterioration is present use Case I.

CASE I: No interior deterioration. Height and width are measurements of the "sound" material, taking into account deterioration.

$$Z = \frac{(\text{Width})(\text{Height})^2}{6}$$

$$Z = \frac{(\ 6\ \text{in.})(\ 12\ \text{in.})^2}{6} = \boxed{144.0}\ \text{in.}^3$$

CASE II: Interior deterioration is present. Void size is to be approximated and represented by a void height, void width and the distance from the center of the void to the bottom of the stringer. Section modulus will be calculated for the "shell" of sound material.

$$\frac{(1/2)(\text{Width})(\text{Height})^2 - (\text{Void Width})(\text{Void Height})(\text{dist. bot. stringer to } \mathrm{C\!\!\!L}\ \text{Void})}{(\text{Width})(\text{Height}) - (\text{Void Width})(\text{Void Height})}$$

$$Y_{na} = \frac{(1/2)(\ \text{—}\ \text{in.})(\ \text{—}\ \text{in.})^2 - (\ \text{—}\ \text{in.})(\ \text{—}\ \text{in.})(\ \text{—}\ \text{in.})}{(\ \text{—}\ \text{in.})(\ \text{—}\ \text{in.}) - (\ \text{—}\ \text{in.})(\ \text{—}\ \text{in.})}$$

Y_{na} = ____—____ in.

$$I_o \text{ Stringer} = \frac{(\text{Width})(\text{Height})^2}{12}$$

$$= \frac{(\text{—} \text{ in.})(\text{—} \text{ in.})^3}{12} = \underline{\quad\text{—}\quad} \text{ in.}^4$$

$$I_o \text{ Void} = \frac{(\text{Void Width})(\text{Void Height})^3}{12}$$

$$= \frac{(\text{—} \text{ in.})(\text{—} \text{ in.})^3}{12} = \underline{\quad\text{—}\quad} \text{ in.}^4$$

$$\bar{Y} \text{ Stringer} = Y_{na} - 1/2(\text{height})$$

$$= (\text{—} \text{ in.}) - 1/2(\text{—} \text{ in.}) = + \underline{\quad\text{—}\quad} \text{ in.}^*$$

$$\bar{Y} \text{ Void} = Y_{na} - (\text{dist. bot. stringer to } \mathrm{C_L} \text{ Void})$$

$$= (\text{—} \text{ in.}) - (\text{—} \text{ in.}) = + \underline{\quad\text{—}\quad} \text{ in.}^*$$

$$I_{na} = (I_o \text{ Stringer}) - (I_o \text{ Void}) + (\text{Width})(\text{Height})(\bar{Y} \text{ Stringer})^2 - (\text{Void Width})(\text{Void Height})(\bar{Y} \text{ Void})^2$$

$$I_{na} = (\text{—} \text{ in}^4) - (\text{—} \text{ in.}^4) + (\text{—} \text{ in.})(\text{—} \text{ in.})(\text{—} \text{ in.})^2 - (\text{—} \text{ in.})(\text{—} \text{ in.})(\text{—} \text{ in.})^2 = \underline{\quad\text{—}\quad} \text{ in}^4$$

$$Z_{bottom} = \frac{I_{na}}{Y_{na}} = \frac{\text{—} \text{ in.}^4}{\text{—} \text{ in.}} = \boxed{\quad\text{—}\quad} \text{ in.}^3$$

$$Z_{top} = \frac{I_{na}}{(\text{Height} - Y_{na})} = \frac{\text{—} \text{ in.}^4}{(\text{—} \text{ in.} - \text{—} \text{ in.})} = \boxed{\quad\text{—}\quad} \text{ in.}^3$$

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*If negative, the negative sign should be dropped. Both values should be positive.

The INVENTORY RESISTING MOMENT is the moment capacity of the member where stresses do not exceed those specified for Inventory Rating.

The OPERATING RESISTING MOMENT is the moment capacity of the member where stresses do not exceed those specified for Operating Rating.

Calculate Resisting Moments:

CASE I: No interior deterioration.

Inventory Resisting Moment $(MR_i) = \dfrac{(f_{bi})(Z)}{12}$

$$= \frac{(2000\ \text{lb/in.}^2)(144.0\ \text{in.}^3)}{12} = \boxed{24\,000}\ \text{ft-lbs.}$$

Operating Resisting Moment $(MR_o) = \dfrac{(f_{bo})(Z)}{12}$

$$= \frac{(2660\ \text{lb/in.}^2)(144.0\ \text{in.}^3)}{12} = \boxed{31\,920}\ \text{ft-lbs.}$$

CASE II: Interior deterioration is present.

Inventory Resisting Moment (MR_i) =

smaller of $\dfrac{(f_{bi})(Z_{top})}{12}$ or $\dfrac{(f_{bi})(Z_{bottom})}{12}$

$$= \frac{(\text{—}\ \text{lb/in.}^2)(\text{—}\ \text{in.}^3)}{12} = \underline{\quad\text{—}\quad}\ \text{ft-lb.}$$

Use $\boxed{\text{—}}$ ft.lbs.

$$= \frac{(\text{—}\ \text{lb/in.}^2)(\text{—}\ \text{in.}^3)}{12} = \underline{\quad\text{—}\quad}\ \text{ft-lbs.}$$

Operating Resisting Moment (MR_o) =

smaller of $\dfrac{(f_{bo})(Z_{top})}{12}$ or $\dfrac{(f_{bo})(Z_{bottom})}{12}$

$$= \frac{(\text{—}\ \text{lb/in.}^2)(\text{—}\ \text{in.}^3)}{12} = \underline{\quad\text{—}\quad}\ \text{ft-lb.}$$

Use $\boxed{\text{—}}$ ft.lbs.

$$= \frac{(\text{—}\ \text{lb/in.}^2)(\text{—}\ \text{in.}^3)}{12} = \underline{\quad\text{—}\quad}\ \text{ft-lb.}$$

RESISTING MOMENT AVAILABLE FOR LIVE LOAD is the moment capacity of the member (Inventory or Operating resisting moment) less the Dead Load Moment.

Example computation of number of traffic lanes:

For a 17 ft. roadway; number of design traffic lanes = 17/12 = 1.42, therefore use one design traffic lane.

For a 34 ft. roadway; number of design traffic lanes = 34/12 = 2.83, therefore use two design traffic lanes.

M_{LLi} & M_{LLo} = Available Live Load Moment capacity for Inventory and Operating levels.

LLM = Applied Live Load Moment (from Section 3.2 for each loading type)

GVW = Gross Vehicle Weight (from Section 3.2 for each loading type)

DF = Distribution Factor.

Since the roadway width (20.5 ft.) is between 18 and 24 ft., use two traffic lanes.

Distribution Factor obtained from Plate II in Appendix A for a timber plank floor and two traffic lanes is:

$$\left(\frac{S}{3.75}\right)$$

Determining Resisting Moments Available for Live Load:

Available Capacity for Live Load Inventory Moment

$$(M_{LLi}) = (MR_i) - (M_{DL})$$

$$(M_{LLi}) = (24\,000) - (2\,078) = \boxed{21\,922} \text{ ft-lbs.}$$

Available Capacity for Live Load Operating Moment

$$(M_{LLo}) = (MR_o) - (M_{DL})$$

$$(M_{LLo}) = (31\,920) - (2\,078) = \boxed{29\,842} \text{ ft-lbs.}$$

Determine Distribution Factor (DF) =

Number of design traffic lanes = $\frac{\text{(Roadway width)}}{12}$

rounded off to the next lower number of lanes except roadway widths from 18 to 24 ft. shall have two design traffic lanes.

$\frac{(20.5 \text{ ft.})}{12 \text{ ft.}} = $ 1.7 use $\boxed{2}$ design lanes

Based on type of floor and number of design lanes, determine Wheel Load Distribution Factor from Plate II in Appendix A:

$$DF = \frac{(2.25 \text{ ft.})}{(3.75) \text{ ft.}} = \boxed{0.60}$$

Calculate Inventory and Operating Ratings

The formula to be used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(M_{LLi})(GVW)}{(LLM)(1/2)(DF)(1000)}$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{(M_{LLo})(GVW)}{(LLM)(1/2)(DF)(1000)}$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 3.2 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Int. Stringer (Moment)".

DATE	DESIGN	SEC. 4.3.6 EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

CALCULATE INVENTORY AND OPERATING RATINGS FOR INTERIOR STRINGER (MOMENT)

SU2 :

$$\text{Inv. } \frac{(21922)(34.0)}{(77.0)(1/2)(0.60)(1000)} = 32.3 \text{ Kips}$$

$$\text{Op. } \frac{(29842)(34.0)}{(77.0)(1/2)(0.60)(1000)} = 43.9 \text{ Kips}$$

SU3 :

$$\text{Inv. } \frac{(21922)(66.0)}{(111.6)(1/2)(0.60)(1000)} = 43.2 \text{ Kips}$$

$$\text{Op. } \frac{(29842)(66.0)}{(111.6)(1/2)(0.60)(1000)} = 58.8 \text{ Kips}$$

SU4 :

$$\text{Inv. } \frac{(21922)(70.0)}{(118.4)(1/2)(0.60)(1000)} = 43.2 \text{ Kips}$$

$$\text{Op. } \frac{(29842)(70.0)}{(118.4)(1/2)(0.60)(1000)} = 58.8 \text{ Kips}$$

C3 :

$$\text{Inv. } \frac{(21922)(56.0)}{(77.0)(1/2)(0.60)(1000)} = 53.1 \text{ Kips}$$

$$\text{Op. } \frac{(29.842)(56.0)}{(77.0)(1/2)(0.60)(1000)} = 72.3 \text{ Kips}$$

C4 :

$$\text{Inv. } \frac{(21922)(73.271)}{(111.6)(1/2)(0.60)(1000)} = 48.0 \text{ Kips}$$

$$\text{Op. } \frac{(29842)(73.271)}{(111.6)(1/2)(0.60)(1000)} = 65.3 \text{ Kips}$$

C5 :

$$\text{Inv. } \frac{(21922)(80.0)}{(111.6)(1/2)(0.60)(1000)} = 52.4 \text{ Kips}$$

$$\text{Op. } \frac{(29842)(80.0)}{(111.6)(1/2)(0.60)(1000)} = 71.3 \text{ Kips}$$

DATE	DESIGN	SEC. 4.3.6 EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

H : Inv. $\dfrac{(21922)(40.0)}{(112.0)(1/2)(0.60)(1000)} =$ 26.1 Kips

Op. $\dfrac{(29842)(40.0)}{(112.0)(1/2)(0.60)(1000)} =$ 35.5 Kips

HS : Inv. $\dfrac{(21922)(72.0)}{(112.0)(1/2)(0.60)(1000)} =$ 47.0 Kips

Op. $\dfrac{(29842)(72.0)}{(112.0)(1/2)(0.60)(1000)} =$ 63.9 Kips

STEP 5

5. DETERMINE THE OPERATING AND INVENTORY RATINGS FOR HORIZONTAL SHEAR CAPACITY FOR INTERIOR STRINGERS.

By examining the Timber Deck and Stringer Superstructure Inspection Data Sheet, it is determined that all interior stringers are of similar size and spacings; but one particular interior stringer (No. 9 in span 2) is clearly more critical than the other stringers due to deterioration in the outer one-quarter of the length of the stringer. This stringer is the obvious candidate for the Moment Capacity Rating.

A photocopy of the forms "Section 12.1 - Shear Capacity Rating of an Interior Stringer" should be obtained and the Operating and Inventory Ratings are to be determined by performing the computations detailed on the forms.

APPLIED DEAD LOAD HORIZONTAL SHEAR is the shear resulting from the computed dead load per foot of stringer.

W_{DL} = Total DL per foot of stringer from Section 4.3.6.

X = Distance from support to nearest axle (point where V_{DL} is to be computed) from Section 3.3.

The INVENTORY RESISTING HORIZONTAL SHEAR is the horizontal shear capacity of the member where stresses do not exceed those specified for Inventory Rating.

The OPERATING RESISTING HORIZONTAL SHEAR is the horizontal shear capacity of the member where stresses do not exceed those specified for Operating Rating.

Since the stringer being noted for horizontal shear is the same stringer that was rated for moment in Section 4.3.6, W^{DL} may be taken from Section 4.3.6. If the stringer being rated for horizontal shear was not rated for moment, the portion of Section 4.3.6 headed "Dead Load (DL) per foot of stringer" would need to be completed to obtain W^{DL}.

The stringer width multiplied by stringer height shown below must consider deterioration. For stringer No. 9 in span 2, the area of sound material is calculated as follows:

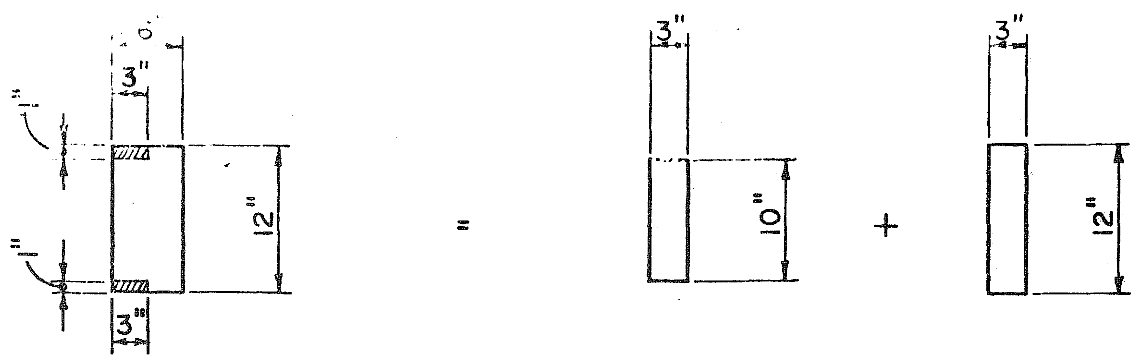

Figure 4-9

$$= (10")(3") + (12")(3")$$

$$= 66 \text{ in.}^2$$

SECTION 4.3.7 - SHEAR CAPACITY RATING OF AN INTERIOR STRINGER

Calculate Applied Dead Load Horizontal Shear (V_{DL})

$V_{DL} = W_{DL}$[(Span Length)(1/2) - (X)]

V_{DL} = (84.81 lb/ft.)[(14 ft.)(1/2) - (3.0 ft.)]

$V_{DL} = \boxed{339.2}$ lbs.

Calculate Resisting Horizontal Shear:

Inventory Resisting Horizontal Shear (VR_i)

VR_i = (2/3)(Stringer Width)(Stringer Height)(f_{vi})

$VR_i = (2/3)\overbrace{(\quad \text{in.})(\quad \text{in.})}^{66\,in^2}(\ 90\ \text{psi.}) = \boxed{3960}$ lbs.

Operating Resisting Horizontal Shear:

VR_o = (2/3)(Stringer Width)(Stringer Height)(f_{vo})

$VR_o = (2/3)\overbrace{(\quad \text{in.})(\quad \text{in.})}^{66\,in^2}(\ 120\ \text{psi.}) = \boxed{5280}$ lbs.

RESISTING HORIZONTAL SHEAR AVAILABLE FOR LIVE LOAD is the horizontal shear capacity of the member (Inventory or Operating) less the applied dead load horizontal shear.

V_{LLi} & V_{LLo} = Available Live Load Shear Capacity for Inventory and Operating levels.

LLV = Applied Live Load Shear (from Section 3.3 for each loading type)

GVW = Gross Vehicle Weight (from Section 3.3 for each loading type)

DF = Distribution Factor from Section 4.3.6 for each loading type)

SECTION 4.3.7 - SHEAR CAPACITY RATING OF AN INTERIOR STRINGER (CONT.)

Determine Resisting Horizontal Shear Available for Live Load

Available Live Load Inventory Horizontal Shear

$$(V_{LLi}) = (VR_i) - (V_{DL})$$

$$(V_{LLi}) = (3960 \text{ lbs.}) - (339.2 \text{ lbs.}) = \boxed{3620.8} \text{ lbs.}$$

Available Live Load Operating Horizontal Shear

$$(V_{LLo}) = (VR_o) - (V_{DL})$$

$$(V_{LLo}) = (5280 \text{ lbs.}) - (339.2 \text{ lbs.}) = \boxed{4940.8} \text{ lbs.}$$

Calculate Inventory and Operating Horizontal Shear Ratings:

The formula used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(V_{LLi})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

and the formula used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{(V_{LLo})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 3.3 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Int. Stringer (Horiz. Shear)".

CALCULATE INVENTORY AND OPERATING RATINGS FOR INT. STRINGER (HORIZ. SHEAR)

SU2 : INV. $\dfrac{(3621)(34.0)}{(1/4)[(.6)(17.29)+(0.60)(17.29)](1000)} =$ 23.7 Kips

Op. $\dfrac{(4941)(34.0)}{(1/4)[(.6)(17.29)+(0.60)(17.29)](1000)} =$ 32.4 Kips

SU3 : INV. $\dfrac{(3621)(66.0)}{(1/4)[(.6)(28.02)+(0.60)(28.02)](1000)} =$ 28.4 Kips

Op. $\dfrac{(4941)(66.0)}{(1/4)[(.6)(28.02)+(0.60)(28.02)](1000)} =$ 38.8 Kips

SU4 : INV. $\dfrac{(3621)(70.0)}{(1/4)[(.6)(27.38)+(0.60)(27.38)](1000)} =$ 30.9 Kips

Op. $\dfrac{(4941)(70.0)}{(1/4)[(.6)(27.38)+(0.60)(27.38)](1000)} =$ 42.1 Kips

C3 : INV. $\dfrac{(3621)(56.0)}{(1/4)[(.6)(18.14)+(0.60)(18.14)](1000)} =$ 37.3 Kips

Op. $\dfrac{(4941)(56.0)}{(1/4)[(.6)(18.14)+(0.60)(18.14)](1000)} =$ 50.8 Kips

C4 : INV. $\dfrac{(3621)(73.271)}{(1/4)[(.6)(28.02)+(0.60)(28.02)](1000)} =$ 31.6 Kips

Op. $\dfrac{(4941)(73.271)}{(1/4)[(.6)(28.02)+(0.60)(28.02)](1000)} =$ 43.1 Kips

C5 : INV. $\dfrac{(3621)(80.0)}{(1/4)[(.6)(28.02)+(0.60)(28.02)](1000)} =$ 34.5 Kips

Op. $\dfrac{(4941)(80.0)}{(1/4)[(.6)(28.02)+(0.60)(28.02)](1000)} =$ 47.0 Kips

DATE	DESIGN	SEC. 4.3.7-EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

H : Inv. $\dfrac{(3621)(40.0)}{(1/4)[(.6)(25.14)+(.6)(25.14)](1000)} = 19.2$ Kips

Op. $\dfrac{(4941)(40.0)}{(1/4)[(.6)(25.14)+(.60)(25.14)](1000)} = 26.2$ Kips

HS : Inv. $\dfrac{(3621)(72.0)}{(1/4)[(.6)(25.14)+(0.60)(25.14)](1000)} = 34.6$ Kips

Op. $\dfrac{(4941)(72.0)}{(1/4)[(.6)(25.14)+(0.60)(25.14)](1000)} = 47.2$ Kips

After computing and tabulating the ratings for each member, the controlling rating is determined by choosing the lowest Inventory and Operating value for each type of loading and entering them under the column "Controlling Rating". This completes the Load Rating for the example problem

Loading Classification	TYPE OF LOADING	RATING LEVEL	FLOOR RATING	EXT. STRINGER (MOMENT)	EXT. STRINGER (HORIZ. SHEAR)	INT. STRINGER (MOMENT)	INT. STRINGER (HORIZ. SHEAR)						CONTROLLING RATING
Florida Legal Loading	SU2	Inventory	22.7	219.6	38.6	32.3	23.7						22.7
		Operating	30.1	299.0	52.7	43.9	32.4						30.1
	SU3	Inventory	43.9	294.1	46.3	43.2	28.4						28.4
		Operating	58.5	400.5	63.1	58.8	38.8						38.8
	SU4	Inventory	54.9	294.0	50.2	43.2	30.9						30.9
		Operating	73.0	400.3	68.5	58.8	42.1						42.1
	C3	Inventory	37.3	361.7	60.7	53.1	37.3						37.3
		Operating	49.6	492.5	82.7	72.3	50.8						49.6
	C4	Inventory	48.8	326.5	51.4	48.0	31.6						31.6
		Operating	64.9	444.6	70.0	65.3	43.1						43.1
	C5	Inventory	53.3	356.5	56.1	52.4	34.5						34.5
		Operating	70.9	485.4	76.4	71.3	47.0						47.0
Design Loading	H	Inventory	18.3	177.6	31.3	26.1	19.2						18.3
		Operating	24.4	241.8	42.6	35.5	26.2						24.4
	HS	Inventory	33.0	319.7	56.3	47.0	34.6						33.0
		Operating	43.9	435.3	76.7	63.9	47.2						43.9

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

CHAPTER 5

REINFORCED CONCRETE SLAB SUPERSTRUCTURE

SECTION 5.1 - OVERVIEW FOR THE STRUCTURAL RATING OF REINFORCED CONCRETE SLAB SUPERSTRUCTURES

A Concrete Slab Superstructure, as the term is used in this manual, is a bridge having a superstructure composed of a conventionally reinforced concrete slab constructed in place as a single unit, spanning between the supporting abutments or other substructure parts. Slab superstructures comprised of precast segments are not evaluated under the scope of this manual.

Reinforced concrete has proven to be a durable and economical structural material for bridge construction. This is acknowledged by the fact that the majority of the bridges in the state of Florida are of concrete construction. Although concrete is very durable when properly proportioned, mixed, and placed, there are several factors that cause deterioration in concrete members. Among those pertinent to reinforced concrete slab superstructures are:

1. Salt Action - Spray from salt or brackish waterways contributes to weathering through recrystallization. Salt also increases water retention or may chemically attack the concrete if certain compounds are present.

2. Differential Thermal Strains - Large temperature variations which set up severe differential strains between the surface and the interior of a concrete mass are an occasional cause of concrete deterioration. Aggregates with lower coefficients of thermal expansion than the cement paste may also set up high tensile stresses.

3. Unsound Aggregates - Structurally weak and/or readily cleavable aggregates are vulnerable to the weathering effects of moisture and cold.

4. Sulfate Compounds in Soil and Water - Sodium, magnesium, and calcium sulfates have a deleterious effect upon compounds in the cement paste and cause rapid deterioration of the concrete.

5. Leaching - Water seeping through cracks and voids in the hardened concrete leaches or dissolves the calcium hydroxide and other material compounds. The resultant of such action is either efflorescence or incrustation at the surface of the cracks.

6. Wear or Abrasion - Traffic abrasion and impact cause wearing of bridge decks.

7. Shrinkage and Flexure Forces - Both of these kinds of forces set up tensile stresses that result in cracks.

8. Corroded Reinforcing Steel - The increase in volume due to corrosion exerts expansive pressures on concrete.

Deterioration caused by the factors described above will appear as three distinct types: Scaling, Spalling, or Cracking. Scaling is the gradual and continuing loss of surface mortar and aggregate over a widespread area. Scaling is the most critical form of deterioration since the effective depth of the slab is reduced over a relatively large area. Spalling is a roughly circular or oval depression in the concrete surface. Spalling results from the separation and removal of a portion of the surface concrete revealing a fracture roughly parallel or slightly inclined, to the surface. Usually, a portion of the depression rim is perpendicular to the surface and often reinforcing steel is exposed. When rating a structure based on the effective depth of a spalled area, it must be realized that since the spalled area is well defined and limited to a confined region (and repair is relatively straightforward) a low rating based solely on a spalled area may not reflect the true condition of the structure. For this reason, spalling is not as critical as scaling. Cracking is a linear fracture in the concrete. The structural rating of the superstructure as analyzed in this manual is not affected by cracking. It should be noted, however, that cracks extending through the entire member may have serious consequences on the shear rating, map cracking may result in significant reduction of the moment capacity rating, and other types of cracks may have varied results on the structural ratings. This manual does not attempt to compute a numerical rating value for members having cracks since the consequences of cracks are not well suited to generalizations and many factors affect the severity of the cracks affect on the rating. Therefore, cracking must be investigated on an individual basis by a qualified engineer. Any cracking noted on the Bridge Inspectors Report should be clearly noted on the Summary of Ratings sheet.

The analysis and rating of Reinforced Concrete Slab Superstructures presented herein is in accordance with "Standard Specifications for Highway Bridges"[1] and "Manual for Maintenance Inspection of Bridges."[2] As a result of these specifications and general engineering design practices, the analysis and rating methods described in this section are subject to the following:

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[1]Standard Specifications for Highway Bridges, Twelfth Edition, 1977.

[2]Manual for Maintenance Inspection of Bridges, 1978.

SECTION 5.1 - OVERVIEW FOR THE STRUCTURAL RATING OF REINFORCED CONCRETE SLAB SUPERSTRUCTURES

1. The analysis is valid for simple spans only.
2. Analysis is accomplished by Ultimate Strength methods[3].
3. Impact loading is included.
4. Reinforcement in the compression zone is not considered in the flexural analysis.
5. The modulus of elasticity of the reinforcing steel is taken as 29,000,000 psi.
6. The concrete strain at failure is taken as .003.

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[3]For a discussion of the design assumptions imposed by employing ultimate strength methods see Standard Specifications for Highway Bridges, p. 101 and 102. For additional description of the ultimate strength method (also called strength method or load factor method) see Design of Concrete Structures, Chapter 3.

SECTION 5.2 - PROCEDURE FOR RATING A REINFORCED CONCRETE SLAB SUPERSTRUCTURE

The following steps should be followed to compute the Inventory and Operating Ratings for a reinforced concrete slab superstructure. The Reinforced Concrete Slab Superstructure Inspection Data Sheets and calculation of Live Load Moments and Shears should already have been computed by the procedures of Chapter 2 and Chapter 3.

STEP 1 COMPUTE THE DEAD LOAD PER FOOT WIDTH OF SLAB. This step is accomplished by completing the forms "Dead Load Computation", Section 5.3.3.

STEP 2 DETERMINE OPERATING AND INVENTORY RATINGS OF SLAB FOR MOMENT CAPACITY. This step is accomplished by completing the forms "Moment Capacity Rating", Section 5.3.4. Only the middle one-half of the slab should be considered for determining the critical section for moment capacity (see Figure 5.1). Spalling or scaling of the bottom part of the slab does not affect the moment capacity of the slab as long as corrosion has not reduced the cross-sectional area of the reinforcing bars. For instances where the corrosion of the reinforcing bars has reduced the effective area of the reinforcing steel, the area of steel (A_s) used should be the lesser of the actual uncorroded area of steel or .75 A_{sb} (balanced area of steel). Spalling or scaling in the top portion of the slab does affect the moment capacity of the member. This should be taken into account by using d measured from the stable concrete to the center of the bottom longitudinal reinforcing. Corrosion of the top longitudinal reinforcing does not affect this analysis for moment capacity rating.

STEP 3 DETERMINE OPERATING AND INVENTORY RATINGS OF SLAB FOR SHEAR CAPACITY. This step is accomplished by completing the forms "Shear Capacity Rating", Section 5.3.5. Only the outer one-quarter of the slab should be considered for determining the critical section for shear capacity (see Figure 5.2). Spalling or scaling of the slab below the bottom reinforcing steel does not affect the shear capacity of the slab. Spalling or scaling in the top surface of the slab does affect the shear capacity of the slab and should be taken into account by using d measured from the stable concrete to the center of the bottom longitudinal reinforcing. Corrosion of the reinforcing steel does not affect the shear capacity rating.

This completes the structural rating calculations. The rating values should be tabulated in accordance with the instructions at the end of each Section and those in Chapter 8.

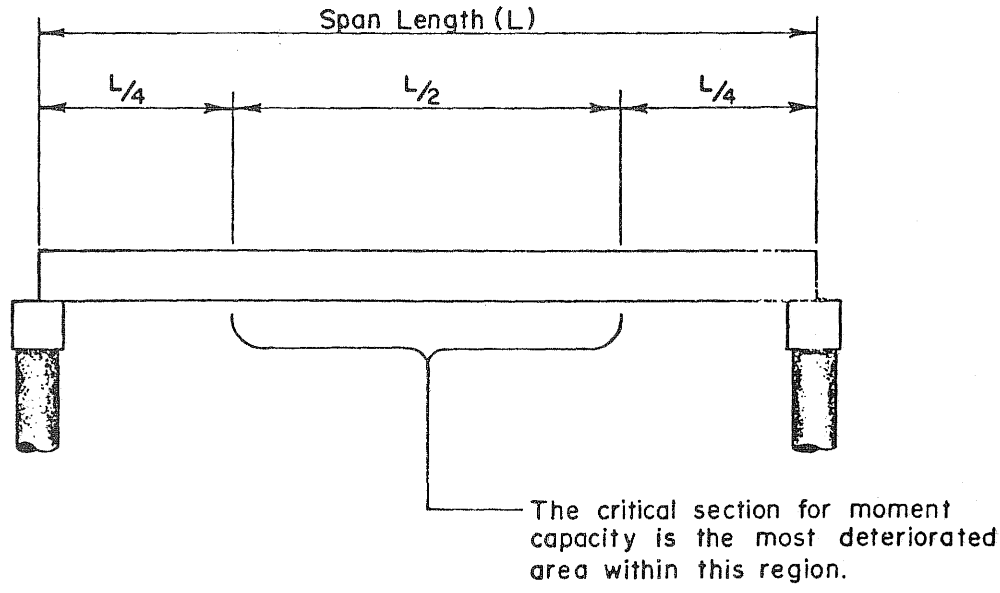


Figure 5-1

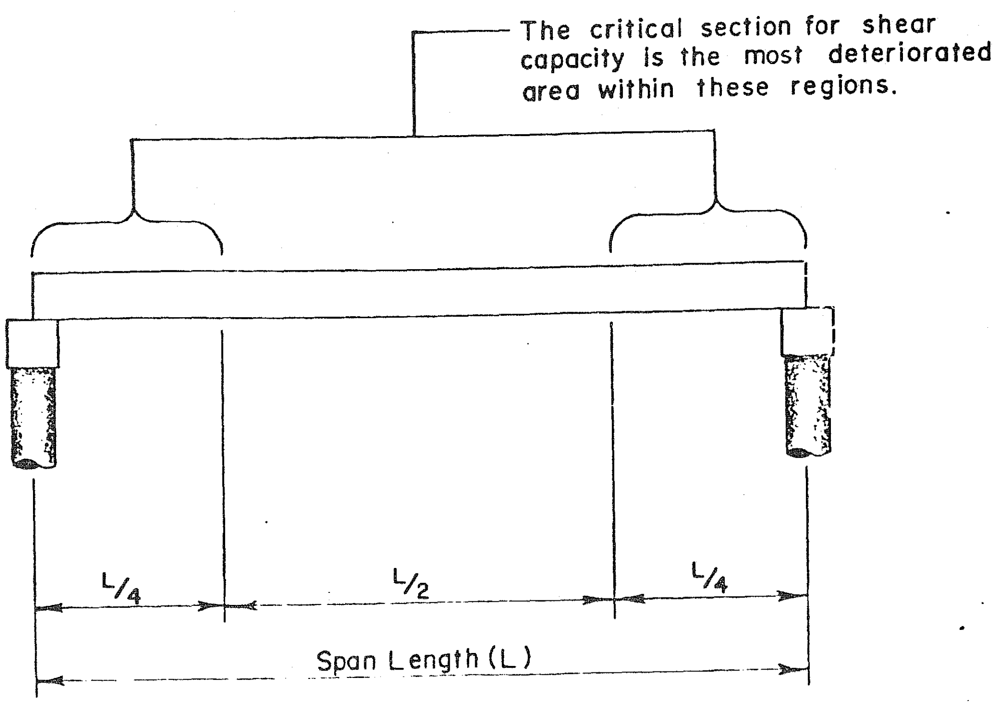


Figure 5-2

Section 5.3 illustrates the use of this manual to rate a reinforced concrete slab superstructure. All forms needed to rate this type superstructure are included in Section 12.2 of Appendix C.

STEP 1 OBTAIN DATA

Figure 5-3 is an example bridge inspection report in which the need for a structural rating has been determined by the engineer. By examination of current legal load requirements it is determined that the Legal Load cases and AASHTO Load cases shown on Plate III of Appendix A are applicable. From a copy of the as-built construction plans the following is noted:

design loading = H20
bottom longitudinal reinforcing = #8 @ 7"
bottom transverse reinforcing = #4 @ 1'-4"
top longitudinal reinforcing = #4 @ 1'-4"
top transverse reinforcing = #4 @ 1'-4"
slab bar clearance = 2" top & bottom
reinforcing steel = grade 40
f'c = 3,000 psi

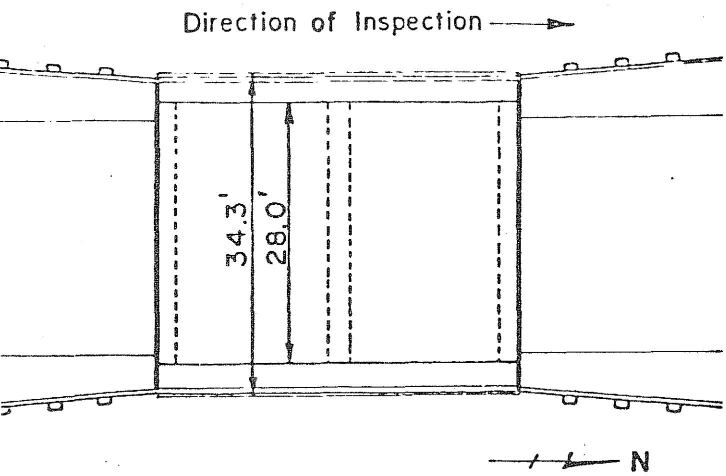
FIGURE 5-3 - EXAMPLE BRIDGE INSPECTION DATA REPORT

Bridge No. XXXXXX XX County SR XXX

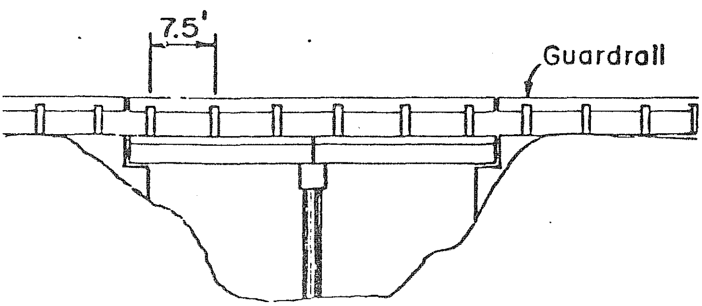
Description: Located on SR XXX at M.P. 00. It is a two span, all concrete structure carrying two lanes of traffic north and south, and was constructed in 1960.

The following measurements were made during the inspection:

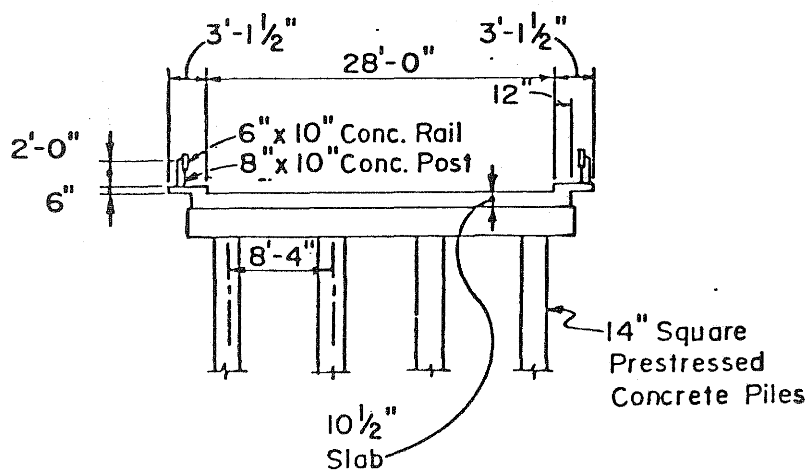
Overall length	=	28 ft.
Spans	=	2 @ 14'
Piling	=	14" sq. prest. conc.
Curb to Curb	=	28.0'
Deck Width	=	34.3'
Appr. Rdwy. Width	=	40.0'
Skew	=	0°



PLAN



ELEVATION



TYPICAL SECTION

REV. 1181

FIGURE 5-3

Approaches: This is a tangent roadway approaching both ends of the structure with a +0.5% grade. Drainage ditches provide adequate handling of the runoff.

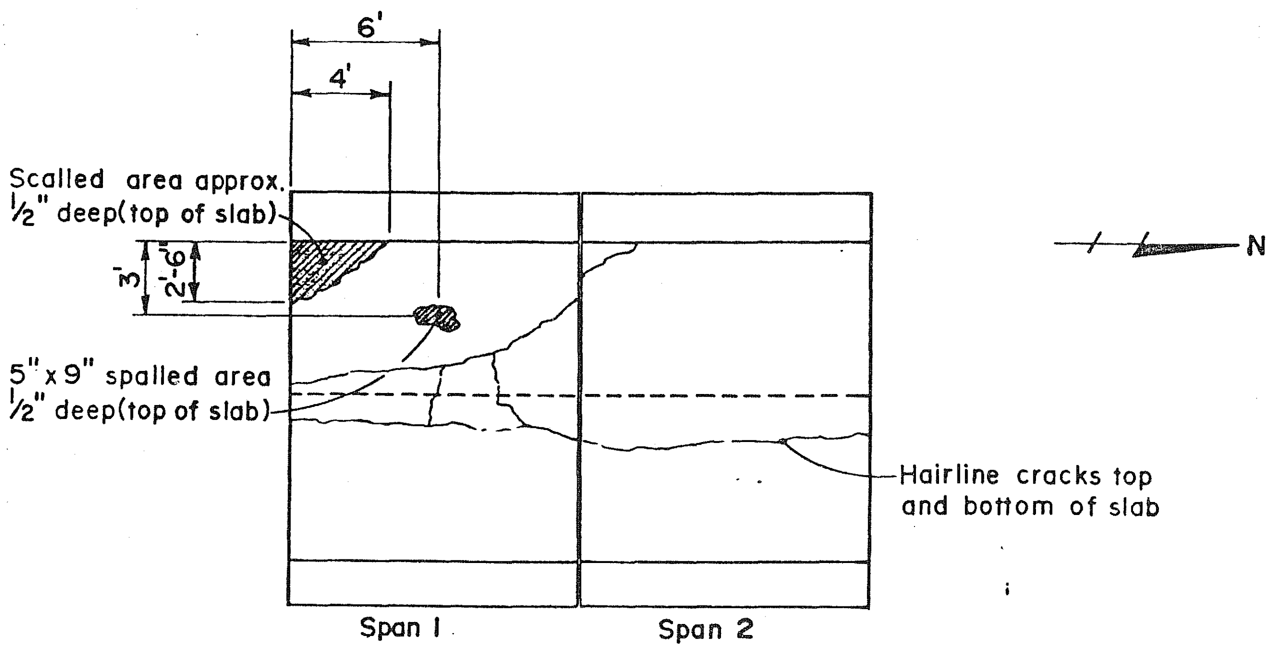
Waterway: XX Creek flows south with about 3 ft. of water. A sandy bottom exists with relatively little vegetation.

Utilities: None.

Piling: Four 14" square prestressed concrete piles at intermediate bent in plumb position. Piles are in good condition.

Cap: 2' x 2' reinforced concrete cap in good condition.

Slab: Reinforced concrete slab with deterioration as shown below:



STEP 2 DETERMINE TYPE OF SUPERSTRUCTURE

Compare the description of the superstructure given in the Bridge Inspection Data Report with the configurations detailed in Chapter 2. The reinforced concrete slab superstructure shown in Section 2.3 is found to be applicable.

STEP 3 COMPLETE INSPECTION DATA SHEET

Obtain a photocopy of the "Reinforced Concrete Slab Superstructure Inspection Data Sheets" (Section 12.2) and record the necessary information in the spaces provided. When information requested on the inspection data sheet is not available from the Bridge Inspection Data Report or "as-built" plans, a field visit will be necessary to obtain the information.

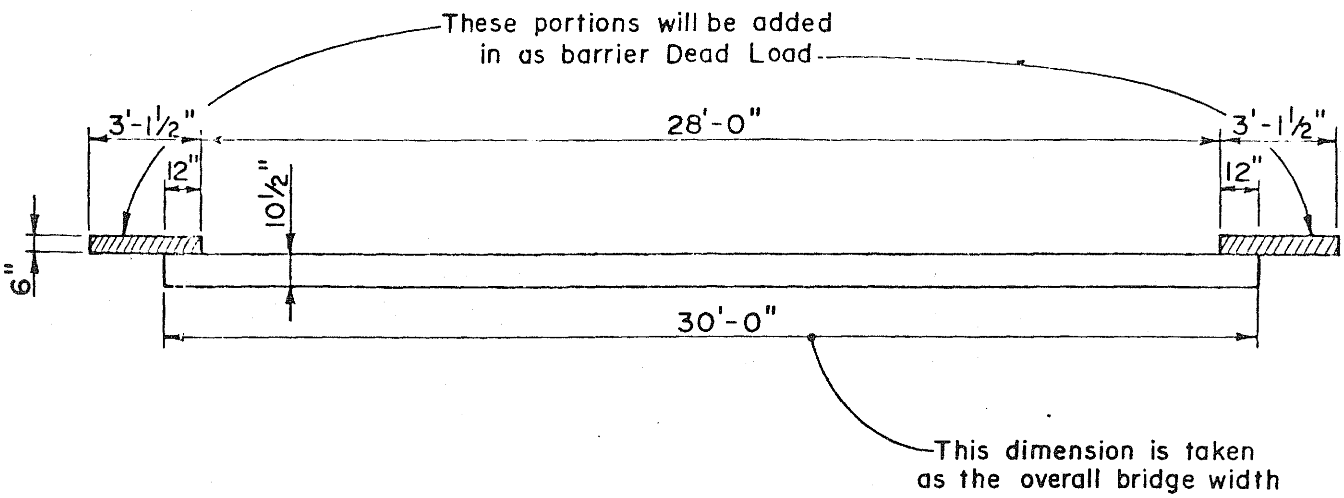


Figure 5-4

d = (slab thickness)-(clearance at bottom)-(1/2)(bot. longitudinal reinf. dia.)

Note botomm longitudinal bars are #8 & from Plate IV in Appendix A, the diameter = 1.00"
d = (10-1/2")-(2")-(1/2)(1.00")
d = 8.0"

SECTION 5.3.1 - REINFORCED CONCRETE SLAB SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE: These sheets should be completed in their entirety before continuing with the rating computations.

By: JOHN DOE Date: XX-XX-XX ft.

Bridge Name: State Road x Over x Creek

Bridge No.: XXXXX Road No.: x County: x

Bridge Length: 30 ft. Number of Spans: 2

Span Length (C_L Support to C_L Support) = 2 @ 14'
(if different, list each)

Overall Bridge Width = 34.3 ft.

Roadway Width (curb to curb) = 28.0 ft.

Sidewalk: Width = None ft.
Rail Size = ______ in. x ______ in.
Rail Post Size & Length = ______ in. x ______ in. x ______ ft.
Average Post Spacing = ______ ft.

Barrier Rails: Rail Size = 6 in. x 10 in.
Rail Post Size & Length = 8 in. x 10 in. x 2.0 ft.
Average Post Spacing = 7.5 ft.

Wearing Surface:

Thickness = — in.

Concrete Slab:

Thickness = 10.5 in.
Longitudinal Rein. (Bottom) Size #8 Avg. Spacing = 7 in.
Longitudinal Rein. (Top) Size #4 Avg. Spacing = 16 in.
Transverse Rein. (Bottom) Size #4 Avg. Spacing = 16 in
Transverse Rein. (Top) Size #4 Avg. Spacing = 16 in.
d = 8.0 in. (distance from top of slab to centerline of bottom longitudinal reinforcing).

SECTION 5.3.1 - REINFORCED CONCRETE SLAB SUPERSTRUCTURE INSPECTION DATA SHEET (CONT.)

Material Strengths:

f'c (compressive strength of concrete) = 3000 psi.
(from contract plans*)

fy (yield strength of reinforcing steel) = 40 000 psi.
(from contract plans**)

These unit stresses shall be used when, in the judgement of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the maximum yield stresses and compressive strength of concrete shall be fixed by the Registered Professional Engineer, based on the field investigation, and shall be substituted for the previously given stresses. These stresses shall in no case be greater than the previously given stresses.

---

* When the compressive strength of the concrete is unknown, f'c may be taken as 3,000 psi., subject to the above paragraph.

**When the Grade is given, or the steel is unknown, use the following yield stresses for reinforcing steel:

Reinforcing Steel	Yield Point (fy) psi
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Intermediate Grade and unknown after 1954 (GRADE 40)	40,000
Hard grade (GRADE 50)	50,000
(GRADE 60)	60,000

STEP 4 COMPUTE LIVE LOAD SHEARS AND MOMENTS

Load cases are same as those on Plate III in Appendix A; therefore, we use the table on Plate I in Appendix A to obtain maximum Live Load Moment for each loading case.

Since the design loading is specified as H-20 (H loading), the HS load case will not need to be investigated.

Live Load Shear for each load case is computed by the method given in Section 3.3.

It is evident that by placing the SU2 load case on the span in the opposite direction, only the 12 kip load would be on the span, thus yielding a smaller Live Load Shear.

REINFORCED CONCRETE SLAB SUPERSTRUCTURE

SPAN LENGTH = 14.0'

DESIGN LOADING = H-20

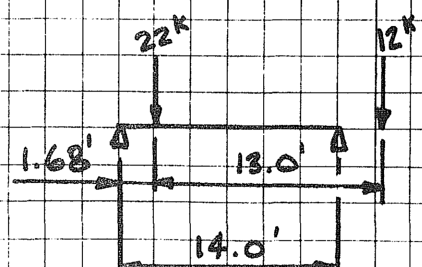
LIVE LOAD MOMENTS - FROM PLATE I

SU2	=	77.0	ft-k
SU3	=	111.6	ft-k
SU4	=	118.4	ft-k
C3	=	77.0	ft-k
C4	=	111.6	ft-k
C5	=	111.6	ft-k
H	=	112.0	ft-k

LIVE LOAD SHEAR

$X = \left(\frac{1}{2}\right)(2.0) + 8.00\text{in.}\left(\frac{1\text{ ft}}{12\text{ in}}\right) = 1.68\text{ ft.}$

Examine SU2 Load Case



$LLV = 22\left(1 - \frac{1.68}{14}\right) = \boxed{19.36^k}$

Note that by placing the SU3 load case on the span in the opposite direction, one 22 kip load would fall off the span and one 22 kip load would fall nearer to the right support than for the placement shown on the following page, thus yielding a smaller Live Load Shear.

Note that by placing the SU4 load case on the span in the opposite direction, only the 13.9 kip load and one 18.7 kip load would fall on the span, thus yielding a smaller value for Live Load Shear than the placement shown on the following page.

Note that by placing the C3 load case on the span in the opposite direction, the 12 kip load would fall 1.68 ft. from the left support and the 22 kip load would fall near the right support, thus yielding a smaller value for Live Load Shear than the placement shown on the following page.

Note that by placing the C4 load case on the span in the opposite direction, only the 7.271 kip load and one 22 kip load would fall on the span. This would clearly yield a smaller value for Live Load Shear than the placement shown on the following page.

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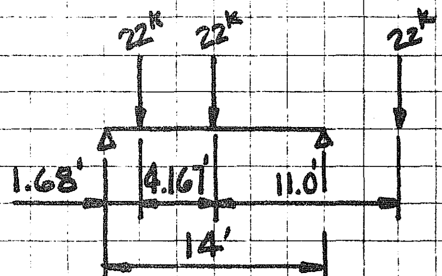
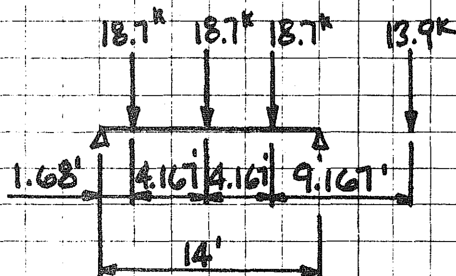
EXAMINE SU3 LOAD CASE



$$LLV = 22\left(1-\frac{1.68}{14}\right) + 22\left(1-\frac{1.68+4.167}{14}\right)$$

$$\boxed{LLV = 32.17^{k}}$$

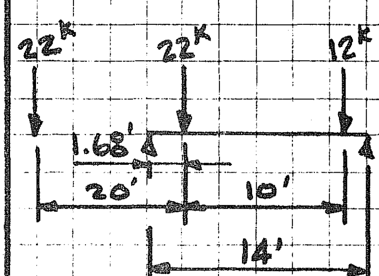
EXAMINE SU4 LOAD CASE



$$LLV = 18.7\left(1-\frac{1.68}{14}\right) + 18.7\left(1-\frac{1.68+4.167}{14}\right) + 18.7\left(1-\frac{1.68+4.167+4.167}{14}\right)$$

$$\boxed{LLV = 32.67^{k}}$$

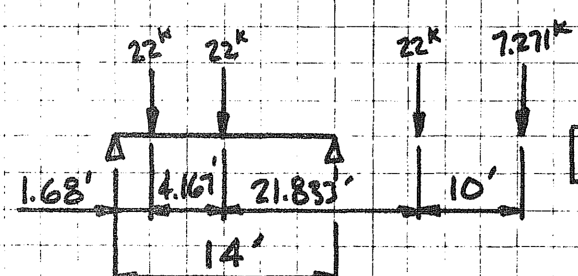
EXAMINE C3 LOAD CASE



$$LLV = 22\left(1-\frac{1.68}{14}\right) + 12\left(1-\frac{1.68+10}{14}\right)$$

$$\boxed{LLV = 21.35^{k}}$$

EXAMINE C4 LOAD CASE



$$LLV = 22\left(1-\frac{1.68}{14}\right) + 22\left(1-\frac{1.68+4.167}{14}\right)$$

$$\boxed{LLV = 32.17^{k}}$$

Note that it is not possible to have more than two axles of the C5 load case on the span at one time, therefore, the only possible placement is shown on the following page.

It is possible to have only one axle for the H load case on the span at a time, therefore, the axles placement shown on the following page is most critical.

Note that design loading was specified as H-20 in the Bridge Inspection Data Report, therefore, the HS loading case need not be considered.

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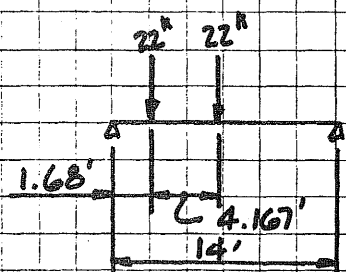
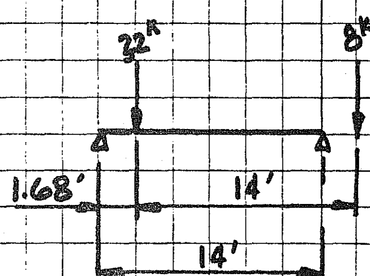
EXAMINE C5 LOAD CASE



$$LLV = 22\left(1-\frac{1.68}{14}\right) + 22\left(1-\frac{1.68+4.167}{14}\right)$$

$$\boxed{LLV = 32.17^k}$$

EXAMINE H LOAD CASE



$$LLV = 32\left(1-\frac{1.68}{14}\right) = \boxed{28.16^k}$$

SUMMARY OF MAXIMUM LIVE LOAD SHEARS

SU2 = 19.36^k

SU3 = 32.17^k

SU4 = 32.67^k

C3 = 21.35^k

C4 = 32.17^k

C5 = 32.17^k

H = 28.16^k

STEP 5 PERFORM RATING CALCULATIONS

The rating calculations are made in accordance with the procedures given in Section 5.2.

1. COMPUTE THE DEAD LOAD PER FOOT WIDTH OF SLAB. Obtain a photocopy of the forms headed "Section 12.2 - Dead Load Computation" and compute the total dead load per one foot strip by performing the calculations detailed thereon.

   For the curb, use the 6" x 3'-1-1/2" raised portion of the slab. (3'-1 1/2" = 37.5")

The intent of the barrier weight per foot computations is to calculate the combined weight per linear foot of barriers and then divide it by the overall bridge width in order to arrive at a weight per linear foot of a one foot wide strip of slab. The barrier configuration illustrated is a general concrete curb, rail and rail post type barrier, but actual configurations of existing barriers may vary somewhat.

For barriers constructed of concrete that differ in configuration from the barrier indicated, the average barrier area should be calculated and a unit weight of 150 lbs/ft^3 used to compute a per foot weight of each barrier. Then the total per foot weight of all barriers should be divided by the overall bridge width to obtain the per foot weight for a one foot strip. For barriers made of steel, the area of steel per linear foot should be computed and a unit weight of 490 lbs/ft^3 used. If standard rolled steel sections are used, the weights per linear foot may be obtained from the AISC Manual.* For aluminum, a unit weight of 175 lb/ft^3 is to be used and for timber a unit weight of 50 lb/ft^3 is to be used.

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*American Institute of Steel Construction, Inc., Manual of Steel Construction, Part I.

SECTION 5.3.3 - DEAD LOAD COMPUTATION

Dead Load (DL) per linear foot for a one foot strip of slab:

Barrier:

(Rail Width)(Rail Height)
(6 in.)(10 in.)(1/144) = 0.42 ft.^{2} -(1)

(Curb Width)(Curb Height)
(37.5 in.)(6 in.)(1/144) = 1.56 ft.^{2} -(2)

(Rail Post Length)(Height)(Width)(1/Avg. Post Spacing)
(2 ft.)(8 in.)(10 in.)(1/7.5 ft.)(1/144)
= 0.15 ft.^{2} -(3)

Barrier Weight per foot =

[(1)+(2)+(3)](150 lb/ft.^{3}) (Number of Barriers)(1/Overall Bridge Width)
= [(0.42 ft.^{2})+(1.56 ft.^{2})+(0.15 ft.^{2})](150)(2)
(1/30.0 ft.) = $\boxed{21.30}$ lb/ft. per one foot strip

Asphalt Wearing Surface:

(Roadway Width)(144 lb/ft.^{3})(Thickness)(1/Overall Bridge Width)
= (— ft.)(144)(— in.)(1/12)(1/ — ft.)

= $\boxed{—}$ lb/ft. per one foot strip

Concrete Slab:

(Thickness)(150 lb/ft.^{3})

= (10.5 in.)(1/12)(150) = $\boxed{131.25}$ lb/ft. per one foot strip

Total DL per one foot strip (W_{DL}) = $\boxed{152.55}$ lb/ft.

The APPLIED DEAD LOAD MOMENT is the moment resulting from the weight of the components making up the structure (in this case the weight of the barrier, the wearing surface, and the concrete slab).

c = distance from the neutral axis of a member in flexure to the extreme compressive fiber.

β = a factor describing the stress curve as a function of c. β shall be taken as 0.85 for concrete strengths (f'c) up to 4000 psi and shall be reduced continuously at a rate of 0.05 for each 1,000 psi in excess of 4,000 psi.

STEP 5

2. DETERMINE OPERATING AND INVENTORY RATINGS OF SLAB FOR MOMENT CAPACITY.

A photocopy of the forms "Section 12.2 - Moment Capacity Rating" should be obtained and the Operating and Inventory Ratings determined by performing the computations detailed thereon.

From Plate IV in Appendix A the area of a #8 bar = 0.79 in^2.

Since the spalled area in span 1 falls in the center one-half of the span, we want to check the slab in the spalled area, so use:

d = actual d - depth of spalled area
d = 8.00" - .50"
d = 7.50"

Since we have 3,000 psi concrete use β = 0.85.

SECTION 5.3.4 - MOMENT CAPACITY RATING

Calculate Applied Dead Load Moment (M_{DL}):

M_{DL} = (1/8)(W_{DL})(Span Length)^2(1/1000)

= (1/8)(152.55 lb/ft)(14 ft.)^2(1/1000) = $\boxed{3.74}$ ft-kips

Determine Area of Reinforcing Steel (As) to be used in computing Moment Capacity of Slab:

actual As = (Ab)*(1/Avg. spacing for Bottom Longitudinal Reinf.)(12)

= (0.79 in.^{2})(1/7 in.)(12) = $\boxed{1.35}$ in.^{2}/ft.

Calculate Asb:

$$A_{sb} = \frac{.85(\beta)(f'c)(d)(12\text{ in.})}{(fy)}\left[\frac{87{,}000}{87{,}000 + fy}\right]$$

$$.75\ A_{sb} = \frac{(.75)(.85)(.85\ )(3000\text{ psi})(\ 7.5\text{ in.})(12)}{(40{,}000\text{ psi})}\left[\frac{87{,}000}{87{,}000 + (40{,}000\text{ psi})}\right]$$

$.75\ A_{sb}$ = $\boxed{2.51}$ in.^{2}/ft.

If, as most instances should, the actual area of steel (A_s) is less than the balanced steel area (A_{sb}) use the lesser of the actual A_s or .75 A_{sb}.

As = $\boxed{1.35}$ in.^{2}/ft.

If the actual area of steel (A_s) is greater than the balanced steel area (A_{sb}), compression controls the analysis and a professional engineer should be consulted.

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*Area of reinforcing bar, see Plate IV in Appendix A for area of bar, given the bar designation from the Inspection Data Sheet.

Determine Ultimate Moment Capacity (Mu) of Member:

$$Mu = \frac{(As)(fy)(d)}{12} - \frac{(As)^2(fy)^2}{2(.85)(f'c)(144)}$$

$$Mu = \frac{(1.35\ \text{in.}^2)(40{,}000\ \text{psi.})(7.50\ \text{in.})}{12} - \left[\frac{(1.35\ \text{in.}^2)^2(40{,}000\ \text{psi})^2}{(1.7)(3000\ \text{psi.})(144)}\right]$$

Mu = (29,779 ft-lbs)(1/1000)

Mu = $\boxed{29.78}$ ft-kips.

Determine Impact Factor:

$$I = 1 + \left[\frac{50}{(\text{Span Length} + 125)}\right] \text{, but not greater than 1.30}$$

$$I = 1 + \left[\frac{50}{(\ 14\ \text{ft.} + 125)}\right] = \underline{1.36}$$

use I = $\boxed{1.30}$

Determine Distribution Factor:

Span Length = center to center of Supports = 14 ft. but not greater than:

($\mathbb{C}_L$ to $\mathbb{C}_L$ of Supports) - (Pier Cap Width) + (Slab Thickness)

= (14 ft.) - (24 in.)(1/12) + (10.5 in.)(1/12)

= 12.875 ft.

use span length = $\boxed{12.875}$ ft.

DF = 4 + 0.06 (Span Length), but not greater than 7.0

DF = 4 + 0.06 (12.875 ft.) = 4.77

use DF = $\boxed{4.77}$

LLM = Applied Live Load Moment from Section 3.2 for each loading type (ft.-kips).

GVW = Gross Vehicle Weight from Section 3.2 for each loading type (kips).

Calculate Inventory and Operating Moment Ratings:

The following formulae are to be used to compute Inventory and Operating Moment Ratings for each loading case. Live Load Moment (LLM) used in the formulae is from Section 3.2 of this Manual. Notice that the numerator of the Rating Factor equations do not change for the different loading cases, therefore it will need to be calculated only once with the denominator changing for each different loading case. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the GVW to obtain the rating. This rating should then be entered under the appropriate type of loading in the "Summary of Ratings" table in Chapter 8, under the heading "Moment Rating".

Inventory Rating:

$$RF = \frac{\left[\frac{(0.9)(Mu)}{(1.3)} - M_{DL}\right](3/5)(DF)}{(1/2)(LLM)(I)}$$

Operating Rating:

$$RF = \frac{\left[\frac{(0.9)(Mu)}{(1.3)} - M_{DL}\right](DF)}{(1/2)(LLM)(I)}$$

Since the numerator of each rating equation is the same, first evaluate that part of the equation.

Now evaluate the Rating Factor for Inventory and Operating Levels and multiply by the GVW to obtain the rating.

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CALCULATE INVENTORY AND OPERATING RATINGS FOR MOMENT CAPACITY:

CALCULATE NUMERATOR FOR INVENTORY RATING EQUATION:

$$\left[\frac{(0.9)(29.78)}{1.3} - 3.74\right]\left(\frac{3}{5}\right)(4.77) = 48.30$$

CALCULATE NUMERATOR FOR OPERATING RATING EQUATION:

$$\left[\frac{(0.9)(29.78)}{1.3} - 3.74\right] 4.77 = 80.50$$

SU2: INV. $\dfrac{48.30}{(1/2)(77.0)(1.3)} = 0.965$

$34.0\,(0.965) = 32.8$ KIPS

OP. $\dfrac{80.50}{(1/2)(77.0)(1.3)} = 1.61$

$34.0\,(1.61) = 54.7$ KIPS

SU3: INV. $\dfrac{48.3}{(1/2)(11.6)(1.3)} = 0.666$

$66.0\,(0.666) = 44.0$ KIPS

OP. $\dfrac{80.5}{(1/2)(11.6)(1.3)} = 1.11$

$66.0\,(1.11) = 73.2$ KIPS

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SU4: INV. $\dfrac{48.30}{(1/2)(118.4)(1.3)} = 0.628$

$70.0\,(0.628) = 43.9$ KIPS

OP. $\dfrac{80.5}{(1/2)(118.4)(1.3)} = 1.05$

$70.0\,(1.05) = 73.2$ KIPS

C3: INV. $\dfrac{48.3}{(1/2)(77.0)(1.3)} = 0.965$

$56.0\,(0.965) = 54.0$ KIPS

OP. $\dfrac{80.5}{(1/2)(77.0)(1.3)} = 1.61$

$56.0\,(1.61) = 90.1$ KIPS

C4: INV. $\dfrac{48.3}{(1/2)(111.6)(1.3)} = 0.666$

$73.271\,(0.666) = 48.8$ KIPS

OP $\dfrac{80.5}{(1/2)(111.6)(1.3)} = 1.11$

$73.271\,(1.11) = 81.3$ KIPS

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C5: INV. $\frac{48.3}{(1/2)(111.6)(1.3)} = 0.666$

$80.0\,(0.666) = 53.3$ KIPS

OP. $\frac{80.5}{(1/2)(111.6)(1.3)} = 1.11$

$80.0\,(1.11) = 88.8$ KIPS

H: INV $\frac{48.3}{(1/2)(112.0)(1.3)} = 0.663$

$40.0\,(0.663) = 26.5$ KIPS

OP. $\frac{80.5}{(1/2)(112.0)(1.3)} = 1.106$

$40.0\,(1.106) = 44.2$ KIPS

This completes the slab rating for Moment Capacity. The Rating Values are summarized in the Summary of Ratings Table at the end of this section.

STEP 5

3. DETERMINE OPERATING AND INVENTORY RATINGS OF SLAB FOR SHEAR CAPACITY.

A photocopy of the forms "Section 12.2 - Shear Capacity Rating" should be obtained and the Operating and Inventory Ratings determined by performing the computations detailed therein.

As suggested in Section 5.3.5, method one will be used to determine the Shear Ratings. When the Rating Values are compared in the Summary of Ratings Table, those cases in which the Shear Rating controls will be reevaluated using method two.

Applied Dead Load Shear is the shear resulting from the computed Dead Load per foot of stringer.

W_{DL} = Total DL per foot of stringer from Section 5.3.4

x = Distance from support to nearest axle load (the point where V_{DL} is to be computed) from Section 3.3.

There are two acceptable methods of computing V_c. Method one is the simpler method, but yields a somewhat conservative value for permitted shear stress. Method two is more involved and yields a higher permitted shear stress than does method one. Obviously we want to limit the amount of computation so method one should be used and the ratings computed and compared with the ratings from the Moment Capacity ratings. If the Moment Capacity ratings control, the problem is finished. But, if the Shear Capacity ratings control, method two must be used to compute V_c and the ratings must be recalculated. Method two need only be used for those loading cases where the Shear Capacity ratings control by using method one.

Obviously, we could always use method two, and then there would be no need to recalculate V_c or the Rating Values. However, in a great many instances the Moment Capacity rating is more critical than the Shear Capacity rating, even by using the more conservative value of V_c from method one. Since method two requires a great deal more computation than does method one, a substantial amount of time may be saved by using the suggested procedure.

SECTION 5.3.5 - SHEAR CAPACITY RATING

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = (W_{DL})[(\text{Span Length})(1/2) - (x)](1/1000)$$

$$V_{DL} = (152.55 \frac{\text{lb}}{\text{ft}})[(14 \text{ ft}) \frac{1}{2} - (1.68 \text{ ft})] \frac{1}{1000} = \boxed{0.812} \text{ kips}$$

Determine Impact Factor

$$I = 1 + \frac{50}{(\text{Span Length}) - (x) + 125}, \text{ but not greater than } 1.30$$

$$I = 1 + \frac{50}{(14 \text{ ft.}) - (1.68 \text{ ft.}) + 125} = \underline{1.36}\text{; use } I = \boxed{1.30}$$

Determine the Maximum permitted Shear Stress for the concrete slab (v_c):

Method One:

$$v_c = 2\sqrt{f'c}$$

$$v_c = 2\sqrt{3000 \text{ psi}} = \underline{109.54} \text{ psi}$$

Method Two:

$$v_c = 1.9\sqrt{f'c} + 2{,}500 \left(\frac{As}{12d}\right)\left(\frac{(V_{DL} + LLV)d}{M'u}\right) \leftarrow \text{equation [A]}$$

but not greater than $3.5\sqrt{f'c}$, and the quantity $\frac{(V_{DL} + LLV)d}{M'u}$ shall not exceed 1.0

First, compute M'u (the factored M_{DL} + LLM occurring simultaneously with V_{DL} + LLV at the section being considered).

contribution of M'_{DL} to M'u:

$$M'_{DL} = \frac{(W_{DL})(X)^*}{2}(\text{Span Length} - x)$$

$$M'_{DL} = \frac{(\ \_\_\ \text{lb/ft.})(\ \_\_\ \text{ft.})}{2}(\ \_\_\ \text{ft.} - \_\_\ \text{ft.})$$

M'_{DL} = __________ ft-lbs.

Contributions of LLM' to M'u shall be determined by evaluating the moment resulting from the axle placements used to compute LLV for each loading case. Shown below is a general equation to calculate the live load moment for these axle placements. If the load case being examined has less than five axles on the span, simply delete the parts of the equation pertaining to the unused axles. Example: if three axles are being used, delete

the terms $\frac{P_4 X}{L}(L-x-a-b-c)$ and $\frac{P_5 X}{L}(L-x-a-b-c-d)$.

This equation is to be evaluated for each loading case that was used in evaluating shear. The variables x, a, b, c, & d are the same as those used in the shear calculations.

$$LLM' = \frac{P_1 X}{L}(L-X)+\frac{P_2 X}{L}(L-X-a)+\frac{P_3 X}{L}(L-X-a-b)+\frac{P_4 X}{L}(L-X-a-b-c)+\frac{P_4 X}{L}(L-X-a-b-c-d)$$

Now, for each loading case compute the value of the expression:

$$\frac{(V_{DL} + LLV)d}{(M'_{DL} + LLM')}$$

The largest value is the one to be used in equation [A]; unless that value exceeds 1.0, in which case 1.0 should be used in equation [A] in place of that computed above.

---

*X is from the live load shear computations completed as a part of Section 3.3

Since the scalled area in span one falls in the outer one-quarter of the span we want to check the slab in the scalled area, so use:

d = actual d - depth of scalled area
d = 8.00" - .50"
d = 7.500"

The following formulae are to be used to compute Inventory and Operating Shear Ratings for each loading case. Live Load Shear (LLV) used in the formulae is from Section 3.3 of this manual and the Distribution Factor (DF) is from the Moment Rating Calculations, Sec. 5.3.4. Notice that the numerator of the rating factor equations does not change for the different loading cases, therefore it will need to be calculated only once and then the denominator changed for each different loading case. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the Gross Vehicle Weight (GVW) to obtain the rating. This rating should then be entered under the appro- priate type of loading in the "Summary of Ratings" table in Chapter 8, under the heading of "Shear Rating".

Evaluating equation [A]

$$v_c = 1.9\sqrt{\text{— lb/in}} + 2500\left(\frac{(\text{— in})}{12(\text{— in})}\right)(\text{—})$$

$$v_c = \text{—}\ \text{lb/in}$$

but not greater than $3.5\sqrt{f'c}$

$$= \sqrt{\text{— lb/in}}$$

$$= \text{—}\ \text{lb/in}$$

therefore, vc = $\boxed{\text{—}}$ lb/in. by method two.

Calculate Ultimate Shear Capacity (Vu) of member:

$$Vu = (v_c)(d)(12)$$

$$Vu = (109.54\ \text{psi})(7.5\ \text{in.})(12\ \text{in.})(1/1000)$$

$$Vu = \boxed{9.86}\ \text{kips}$$

Calculate Inventory and Operating Shear Ratings:

Inventory Rating:

$$RF = \frac{\left[\frac{(0.85)(Vu)}{1.3} - V_{DL}\right]\left(\frac{3}{5}\right)(DF)}{(1/2)(LLV)(I)}$$

Operating Rating:

$$RF = \frac{\left[\frac{(0.85)(Vu)}{1.3} - V_{DL}\right](DF)}{(1/2)(LLV)(I)}$$

CALCULATE INVENTORY AND OPERATING RATINGS FOR SHEAR CAPACITY :

CALCULATE NUMERATOR FOR INVENTORY RATING EQUATION :

$$\left[\frac{(0.85)(9.86)}{1.3} - 0.812\right]\left(\frac{3}{5}\right)(4.77) = 16.127$$

CALCULATE NUMERATOR FOR OPERATING RATING EQUATION :

$$\left[\frac{(0.85)(9.86)}{1.3} - 0.812\right](4.77) = 26.879$$

SU2 : INV. $\frac{16.127}{(1/2)(19.36)(1.30)} = 1.282$

$34.0\ (1.282) = 43.6$ Kips

Op. $\frac{26.879}{(1/2)(19.36)(1.30)} = 2.136$

$34.0\ (2.136) = 72.6$ Kips

SU3 : INV. $\frac{16.127}{(1/2)(32.17)(1.30)} = 0.771$

$66.0\ (0.771) = 50.9$ Kips

Op. $\frac{26.879}{(1/2)(32.17)(1.30)} = 1.285$

$66.0\ (1.285) = 84.8$ Kips

SU4 : INV $\dfrac{16.127}{(1/2)(32.67)(1.3)} = 0.760$

$70.0\,(0.760) = 53.2$ Kips

Op. $\dfrac{26.879}{(1/2)(32.67)(1.3)} = 1.265$

$70.0\,(1.265) = 88.6$ Kips

C3 : INV $\dfrac{16.127}{(1/2)(21.35)(1.3)} = 1.162$

$56.0\,(1.162) = 65.1$ Kips

Op. $\dfrac{26.879}{(1/2)(21.35)(1.3)} = 1.937$

$56.0\,(1.937) = 108.4$ Kips

C4 : INV. $\dfrac{16.127}{(1/2)(32.17)(1.3)} = 0.772$

$73.271\,(0.772) = 56.5$ Kips

Op. $\dfrac{26.879}{(1/2)(32.17)(1.3)} = 1.285$

$73.271\,(1.285) = 94.2$ Kips

C5 : - INV $\dfrac{16.127}{(1/2)(32.17)(1.3)} = 0.772$

$80.0\,(0.772) = 61.8$ Kips

Op. $\dfrac{26.879}{(1/2)(32.17)(1.3)} = 1.285$

$80.0\,(1.285) = 102.8$ Kips

SEC. 5.3.5 – EXAMPLE COMPUTATIONS CONT.

SUBJECT

H : INV $\dfrac{16.127}{(1/2)(28.16)(1.3)} = 0.881$

$40.0\,(0.881) = 35.2$ Kips

Op. $\dfrac{26.879}{(1/2)(28.16)(1.3)} = 1.468$

$40.0\,(1.468) = 58.7$ Kips

As the ratings for shear are tabulated, note that in each case the Moment Rating controls; therefore, the computation of Vc by method one was not overly conservative. Had the Shear Rating for any load case been lower than the Moment Rating, Vc would have to be reevaluated by method two and the Shear Ratings re-computed. Method two would have to be employed only for those cases where the Shear Rating controlled using method one for computing Vc.

After computing the Moment and Shear Ratings, determing the cases where Shear Rating Controls and re-evaluating the Shear Rating as necessary, the controlling rating is determined by choosing the lowest Inventory and Operating Levels for each type of loading and entering them under the column "Controlling Rating". This marks the completion of the Load Rating for the example problem.

SEC. 5.3.6. - EXAMPLE COMPUTATIONS CONT.

Loading Classification	TYPE OF LOADING	RATING LEVEL	MOMENT RATING	SHEAR RATING									CONTROLLING RATING
Florida Legal Loading	SU2	Inventory	32.8	43.6									32.8
		Operating	54.7	72.6									54.7
	SU3	Inventory	44.0	50.9									44.0
		Operating	73.2	84.8									73.2
	SU4	Inventory	43.9	53.2									43.9
		Operating	73.2	88.6									73.2
	C3	Inventory	54.0	65.1									54.0
		Operating	90.1	108.4									90.1
	C4	Inventory	48.8	56.5									48.8
		Operating	81.3	94.2									81.3
	C5	Inventory	53.3	61.8									53.3
		Operating	88.8	102.8									88.8
Design Loading	H	Inventory	26.5	35.2									26.5
		Operating	44.2	58.7									44.2
	HS	Inventory											
		Operating											

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

CHAPTER 6

REINFORCED CONCRETE T-BEAM SUPERSTRUCTURE

SECTION 6.1 - OVERVIEW FOR THE STRUCTURAL RATING OF T-BEAM SUPERSTRUCTURES

The T-Beam Superstructure derives its name from a similarity of shape to the letter "T", the head or top most element of the letter consisting of a portion of the deck slab which is constructed integrally with the reinforced concrete beam stem. The analysis presented in this chapter applies only to conventionally reinforced T-Beams. Prestressed or post-tensioned construction of T-Beams is not covered in the scope of this manual.

Reinforced concrete has proven to be a durable and economical structural material for bridge construction. This is acknowledged by the fact that the majority of bridges in the State of Florida are of concrete construction. Although concrete is very durable when properly proportioned, mixed, and placed; there are several factors that cause deterioration in concrete members. Among those pertinent to reinforced concrete slab superstructures are:

1. Salt Action - Spray from salt or brackish waterways contributes to weathering through recrystallization. Salt also increases water retention or may chemically attack the concrete if certain compounds are present.

2. Differential Thermal Strains - Large temperature variations which set up severe differential strains between the surface and the interior of a concrete mass are an occasional cause of concrete deterioration. Aggregates with lower coefficients of thermal expansion than the cement paste may also set up high tensile stresses.

3. Unsound Aggregates - Structurally weak and/or readily cleavvable aggregatres are vulnerable to the weathering effects of moisture and cold.

4. Sulfate Compounds in Soil and Water - Sodium, magnesium, and calcium sulfates have a deleterious affect upon compounds in the cement paste and cause rapid deterioration of the concrete.

5. Leaching - Water seeping through cracks and voids in the hardened concrete leaches or dissolves the calcium hydroxide and other material compounds. The results of such action is either efflorescence or incrustation at the surface of the cracks.

6. Wear or Abrasion - Traffic abrasion and impact cause wearing of bridge decks.

7. Shrinkage and Flexure Forces - Both of these kinds of forces set up tensile stresses that result in cracks.

8. Corroded Reinforcing Steel - The increase in volume due to corrosion exerts expansive pressures on concrete.

Deterioration caused by the factors described above will appear as three distinct types: Scaling, Spalling, or Cracking. Scaling is the gradual and continuing loss of surface mortar and aggregate over a widespread area. Scaling is the most critical form of deterioration since the effective depth of the member is reduced over a relatively large area. Spalling is a roughly circular or oval depression in the concrete surface. Spalling results from the separation and removal of a portion of the surface concrete revealing a fracture roughly parallel, or slightly inclined, to the surface. Usually, a portion of the depression rim is perpendicular to the surface and often reinforcing steel is exposed. When rating a structure based on the effective depth of a spalled area, it must be realized that since the spalled area is well defined and limited to a confined region (and repair is relatively straightforward) a low rating based solely on a spalled area may not reflect the true condition of the structure. For this reason, spalling is not as critical as scaling. Cracking is a linear fracture in the concrete. The structural rating of the superstructure as analyzed in this manual is not affected by cracking. It should be noted, however, that cracks extending through the entire member may have serious consequences on the shear rating. Map cracking may necessitate a significant reduction of the moment capacity rating, and other types of cracks may have varied results on the structural ratings. This manual does not attempt to compute a numerical rating value for members having cracks since the consequences of cracks are not well suited to generalizations and many factors affect the severity of the cracks' effect on the rating. Therefore, cracking must be investigated on an individual basis by a Registered Professional Engineer. Any cracking noted on the Bridge Inspectors report should be clearly noted on the Summary of Ratings sheet.

The analysis and rating of Reinforced Concrete Slab Superstructures presented herein is in accordance with "Standard Specifications for Highway Bridges"[1] and "Manual for Maintenance Inspection of Bridges."[2] As a result of these specification and general engineering design practices, the analysis and rating methods described in this section are subject to the following:

---

[1]Standard Speciifcations for Highway Bridges, Twelfth Edition, 1977.
Intermin Specifications Bridges 1978, 1979, 1980.

[2]Manual for Maintenance Inspection of Bridges, 1978.

1. The analysis is valid for simple spans only.
2. Analysis is accomplished by Ultimate Strength Methods.[3]
3. Impact loading is included.
4. Reinforcement in the compression zone is not considered in the flexural analysis.
5. The analysis is based on the assumption that adequate shear is developed between the beam portion and the slab portion so that monolithic action exists.
6. The modulus of elasticity of the reinforcing steel is taken as 29,000,000 psi.
7. The concrete strain at failure is taken as .003.

---

[3]For a discussion of the design assumptions imposed by employing ultimate strength methods, see Standard Specifications for Highway Bridges, p. 101 and 102. For additional description of the ultimate strength method (also called strength method or load factor method) see Design of Concrete Structures, Chapter 3.

The critical section for moment capacity of the slab is the area defined below with the smallest slab thickness (due to reduction by scaling or spalling)

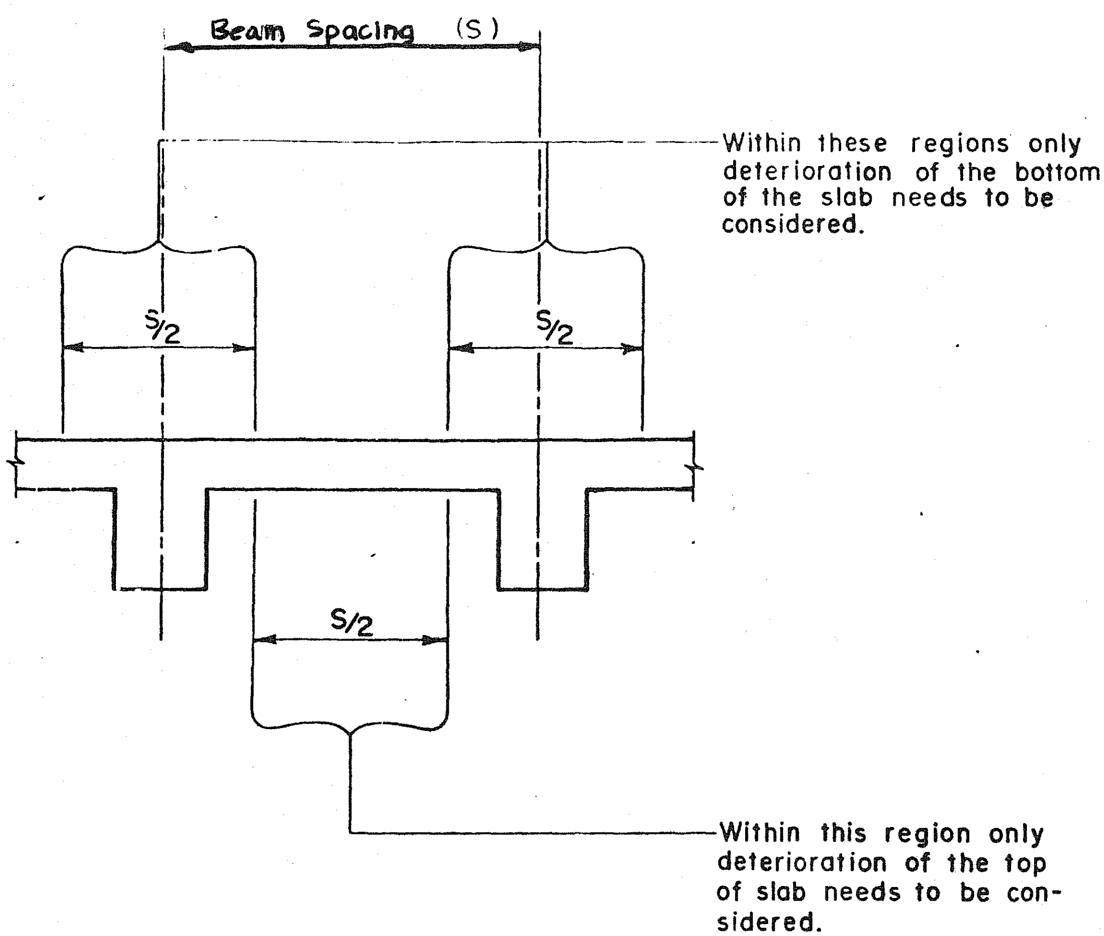


FIGURE 6-1

SECTION 6.2 - PROCEDURE FOR RATING A REINFORCED CONCRETE T-BEAM SUPERSTRUCTURE

The following steps should be followed to compute the Inventory and Operating Rating for a Reinforced Concrete T-Beam Superstructure. The Reinforced Concrete T-beam Inspection Data Sheets and calculation of Live Load Moments and Shears should already have been computed by the procedures of Chapter 2 and 3.

STEP 1 COMPUTE DEAD LOAD PER LINEAR FOOT OF SLAB AND DEAD LOAD PER LINEAR FOOT OF CHANNEL. This step is accomplished by completing the computations on the forms "Section 6.3.3 - Dead Load Computation".

STEP 2 DETERMINE THE OPERATING AND INVENTORY RATINGS OF SLAB FOR MOMENT CAPACITY. This step is accomplished by completing the computations on the forms "Section 6.3.4 - Moment Capacity Rating for Slab" - Spalling or scaling of the slab affects the Moment Capacity of the slab in the areas shown in Figure 6-1. For instances where corrosion has reduced the effective area of the reinforcing steel, the area of steel (A_s) used should be the lesser of the actual areas of steel (taking corrosion into account) or .75 A_{sb} (balanced area of steel.)

STEP 3 DETERMINE THE OPERATING AND INVENTORY RATINGS OF T-BEAM FOR MOMENT CAPACITY. This step is accomplished by completing the computation on the forms" Section 6.3.5 - Moment Capacity Rating for T-Beam." Only the middle one-half of the slab should be considered in determining the critical section for Moment Capacity (see Figure 6-2). Spalling or scaling of the bottom part of the T-beam does not affect the Moment Capacity of the slab as long as corrosion has not reduced the cross-sectional area of the reinforcing bars. For instances where the corrosion of the reinforcing bars has reduced the effective area of the reinforcing steel, the area of steel (A_s) used should be the lesser of the actual uncorroded area of steel or .75 A_{sb} (balanced area of steel). Spalling or scaling of the top portion of the slab does affect the Moment Capacity of the member. This should be taken into account by using d measured from the stable concrete to the center of the bottom longitudinal reinforcing. Corrosion of the slab reinforcing does not affect the analysis for Moment Capacity rating of the T-beam.

STEP 4 DETERMINE THE OPERATING AND INVENTORY RATINGS OF T-BEAM FOR SHEAR CAPACITY. This step is accomplished by completing the computations on the forms "Section 6.3.6 - Shear Capacity Rating for T-Beam." Only the outer one-quarter of the slab should be considered for determining the critical section for Shear Capacity (see Figure 6-3). Spalling or

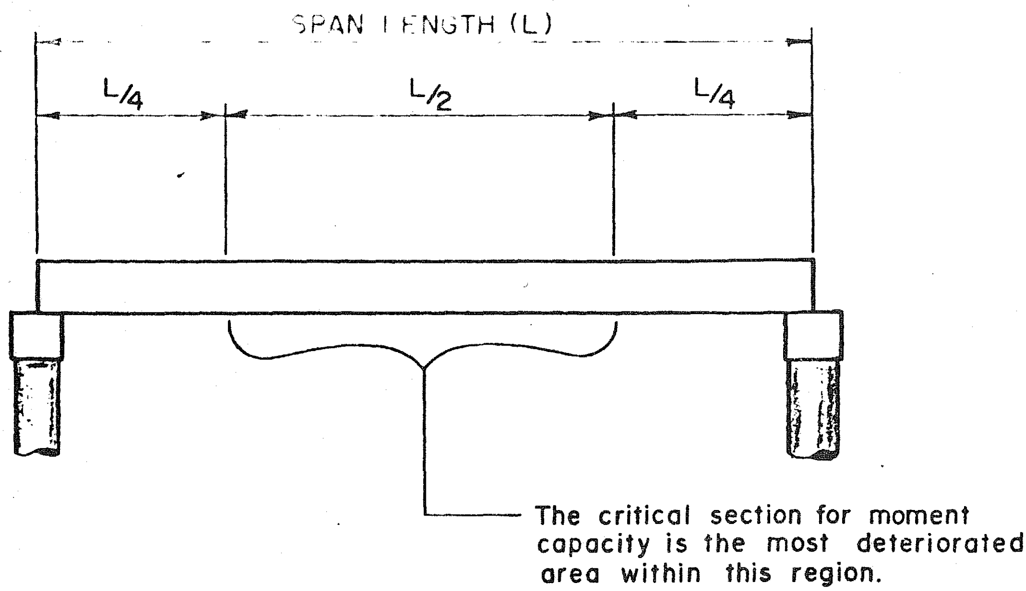


FIGURE 6-2

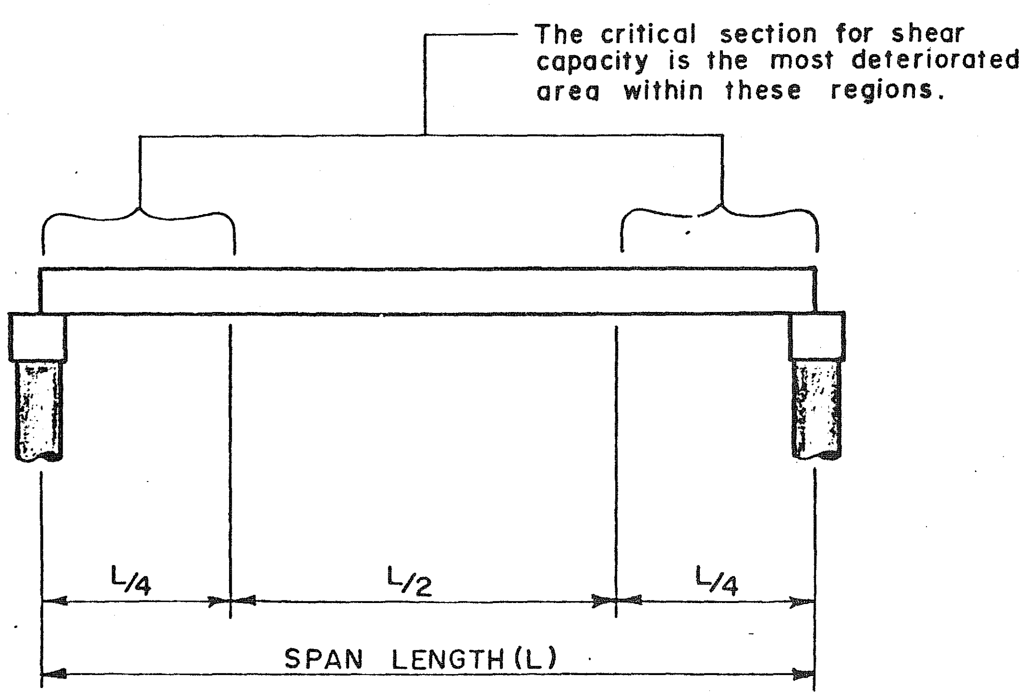


FIGURE 6-3

scaling of the T-beam below the bottom reinforcing steel does not effect the shear capacity of the slab. Spalling or scaling of the top surface of the slab does effect the shear capacity of the slab and should be taken into account by using d measured from the stable concrete to the center of the bottom longitudinal reinforcing. Corrosion of the reinforcing steel does not affect the Shear Capacity Rating.

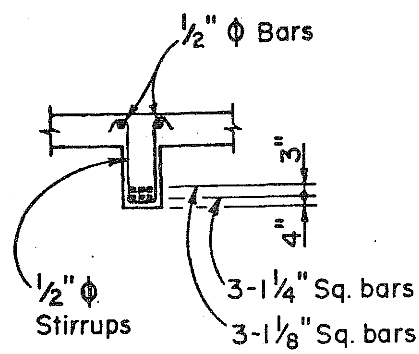
This completes the structural rating calculations. The Rating Values should be tabulated in accordance with the instructions at the end of each Section and those in Chapter 8.

Section 6.3 illustrates the use of this manual to rate a T-Beam superstructure. All forms needed to rate this type superstructure are included in Section 12.3 of Appendix C.

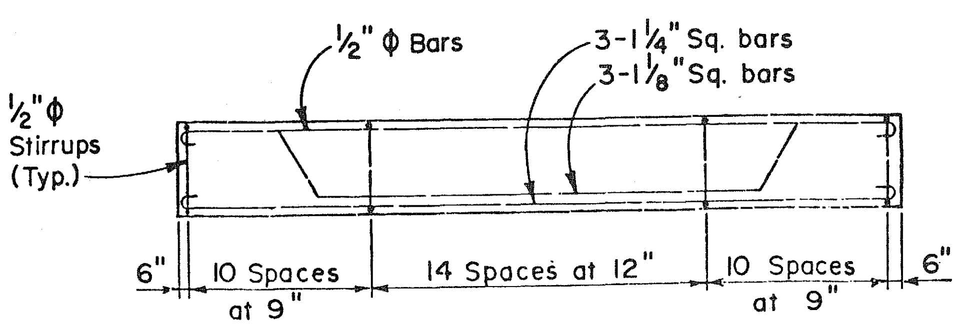
STEP 1 OBTAIN DATA

Figure 6-4 is an example Bridge Inspection report in which the need for a structural rating has been determined by the engineer. By examination of current legal load requirements it is determined that the legal load cases and AASHTO load cases shown on Plate III in Appendix A are applicable. From a copy of the as-built construction plans the following is noted:

Design loading = unknown
Grade of reinforcing steel = medium
Strength of concrete (f'c) = not given
Reinforcement for T-Beam:



SECTION



ELEVATION

Data from as-built plans (Cont.):

Reinforcement for Slab

Longitudinal = 1/2" bars at 18" (alternating top & bottom)
Transverse (top) = 1/2" bars at 9"
(bottom) = 1/2" bars at 6"

Clearance for reinforcement (top and bottom) = 1-1/2"

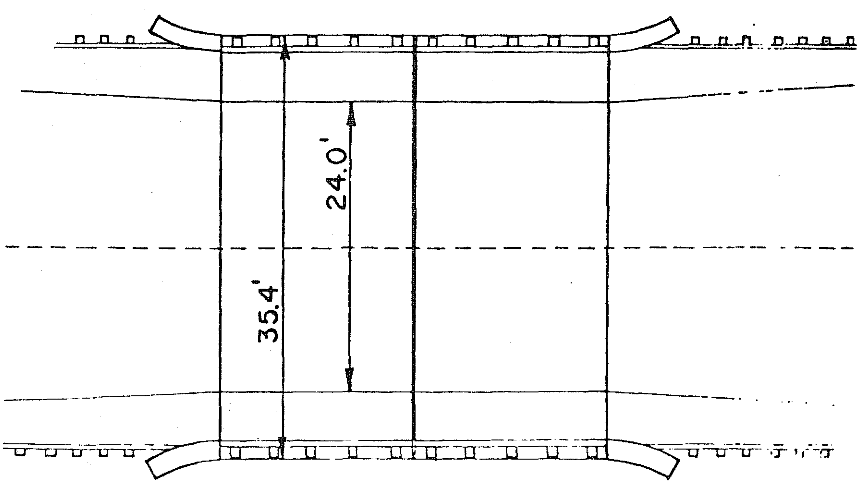
SECTION 6.3 - EXAMPLE PROBLEM
FIGURE 6-4, EXAMPLE BRIDGE INSPECTION DATA REPORT

Bridge No. XXXXXX XX County S.R. XXX

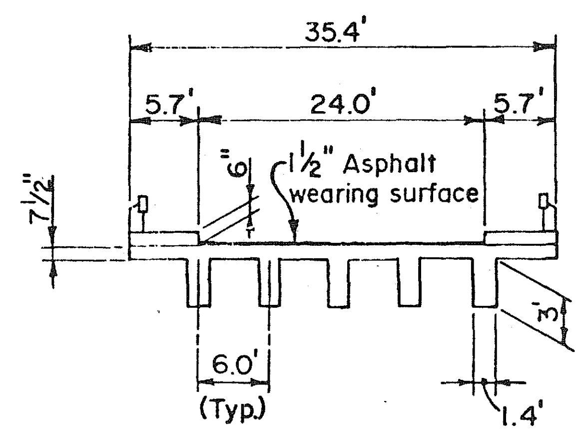
Description: Located on S.R. XXX at M.P. 00. It is a two span, all concrete structure carrying two lanes of traffic north and south, and was constructed in 1927.

The following measurements were made during the inspection:

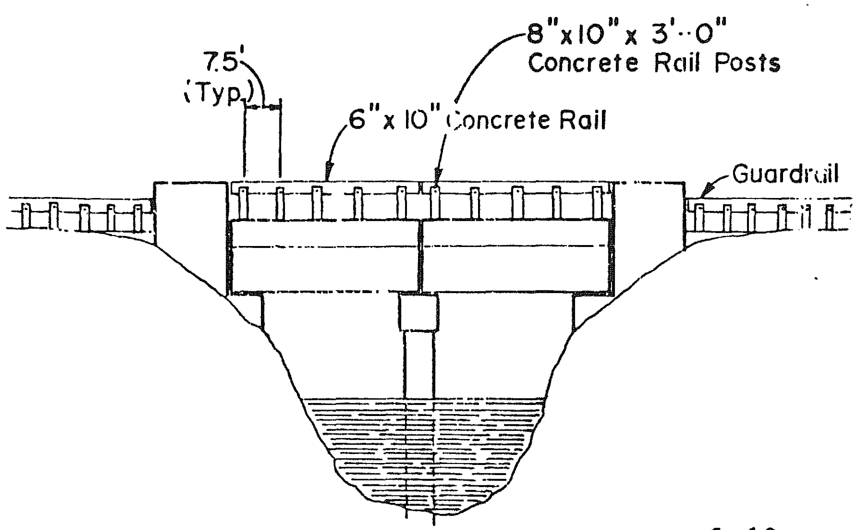
Overall length = 60.0 ft.
Spans = 2 @ 30.0 ft.
Foundation = Conc. pier on spread footing
Curb to Curb = 24.0 ft.
Deck width = 35.4 ft.
Approach roadway width = 30.0 ft.
Skew = 0°



PLAN



TYPICAL SECTION



ELEVATION

FIGURE 6-4

Approaches: There is a tangent roadway approaching both ends of the structure with a 0.0% grade across the bridge.

Waterway: XX canal flows east with about 4 ft. of water. A heavily vegetated bottom exists.

Columns: 3'-0" x 3'-0" concrete columns in good condition.

Caps: 3'-6" wide x 5'-6" deep concrete cap in good condition.

T-Beam and Slab are in good condition with no spalled or scaled areas.

STEP 2 DETERMINE TYPE OF SUPERSTRUCTURE

Compare the description of the superstructure given in the Inspection Data Report with the configurations detailed in Chapter 2. The Reinforced Concrete T-Beam superstructure shown in Section 2.4 is found to be applicable.

STEP 3 COMPLETE INSPECTION DATA SHEET

Obtain a photocopy of the forms headed "Section 12.3 - Reinforced Concrete T-Beam Superstructure Inspection Data Sheet" and record the necessary information in the spaces provided. When information requested on the inspection data sheet is not available from the Bridge Inspection Data Report or as-built plans, a field investigation will be necessary to obtain the data.

d = slab thickness - clearance - 1/2 bar diameter
d = 7-1/2" - 1-1/2" - 1/2 (1/2")
d = 5-3/4"

SECTION 6.3.1 REINFORCED CONCRETE T-BEAM SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE: These sheets should be completed in their entirety before continuing with the rating computations.

By: John Doe Date: xx-xx-xx

Bridge Name: State Road xx Over xx Canal

Bridge No.: xxxxx Road No.: xx County: xx

Bridge Length = 60.0 ft. Number of Spans = 2

Span length ($\mathbb{C}_L$ to $\mathbb{C}_L$ of support = 2 at 30.0 ft.
(if different, list for each)

Overall Bridge Width = 35.4 ft.

Bridge Width (curb to curb) = 24.0 ft.

Sidewalk: Width = 5.7 ft.
Rail Size = — in. x — in.
Rail Post Size & Length = — in. x — in. x — ft.
Average Post Spacing = — ft.

Barrier Rails: Rail Size = 6 in. x 10 in.
Rail Post Size & Length = 8 in. x 10 in. x 3.0 ft.
Average Post Spacing = 7.5 ft.

Wheel Guard: width = 68.4 in.
height = 6 in.

Asphalt Wearing Surface:

Thickness = 1 1/2 in.

Concrete Slab:

Thickness = 7 1/2 in.
Longitudinal Rein. (Bottom) Size 1/2" ϕ Avg. Spacing = 36 in.
Longitudinal Rein. (Top) Size 1/2" ϕ Avg. Spacing = 36 in.
Transverse Rein. (Bottom) Size 1/2" ϕ Avg. Spacing = 6 in.
Transverse Rein. (Top) Size 1/2" ϕ Avg. Spacing = 9 in.
d = 5 3/4 in. (distance from top of slab to centerline of longitudinal reinforcing).

Beam Height - 3.0 ft (12 in./ft.) + 7-1/2 in. = 43.5 in.

$$d = \text{Beam Height} - \left[\left(\begin{array}{c}\text{total area of}\\ \text{1st row steel}\end{array}\right)\left(\begin{array}{c}\text{distance from bot.}\\ \text{to 1st row}\end{array}\right)\right.$$

$$\left. + \left(\begin{array}{c}\text{total area of}\\ \text{2nd row of steel}\end{array}\right)\left(\begin{array}{c}\text{distance from bottom}\\ \text{to 2nd row}\end{array}\right)\right]\left(\frac{1}{\text{total steel area}}\right)$$

$$d = 43.5 \text{ in.} - [(3)(1.56 \text{ in}^2)(4 \text{ in.})+(3)(1.27 \text{ in}^2)(7 \text{ in})]\left[\frac{1}{(3)(1.56 \text{ in}^2)+(3)(1.27 \text{ in}^2)}\right]$$

note area 1-1/4" Φ bar = 1.56 in.^{2}
area 1-1/8" Φ bar = 1.27 in.^{2} from Plate IV, Appendix A.

$$d = 43.5 \text{ in.} - [45.39 \text{ in.}^3]\left[\frac{1}{8.49 \text{ in.}^2}\right] = 38.15 \text{ in.}$$

The compressive strength of the concrete (f'c) is taken as 3,000 psi per Registered Professional Engineers' recommendation.

Since yield strength of the steel is unknown and prior to 1954, fy of 33,000 is used.

Concrete Beam:

Height = 43.5 in.
Width = 16 in.
Spacing Between Beams 6.0 ft.
Tension Reinforcement (reinforcement in bottom of beam)

size 1 1/8" Φ and 1 1/4" Φ
number of bars = 3 - 1 1/4" Φ and 3 - 1 1/8" Φ
d (distance from centroid of reinforcement to top of concrete slab) = 38.15 in.

Shear reinforcement in outer quarters of T-beam.

stirrup size = 1/2" Φ
stirrup spacing = 9 in.

Material Strengths:

f'c (compressive strength of concrete) = 3000 psi.
(from contract plans*)

fy (yield strength of reinforcing steel) = 33000 psi.
(from contract plans**)

These unit stresses shall only be used when, in the judgement of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the maximum yield stresses and compressive strength of concrete shall be fixed by the Registered Professional Engineer, based on the field investigation, and shall be substituted for the previously given stresses. These stresses shall in no case be greater than the previously given stresses.

---

* When the compressive strength of the concrete is unknown, f'c may be taken as 3,000 psi, subject to the above paragraph.

** When the Grade is given or the steel is unknown, use the following yield stresses for reinforcing steel:

Reinforcing Steel	Yield Point (fy) psi
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Intermediate Grade and Unknown after 1954 (GRADE 40)	40,000
Hard grade (GRADE 50)	50,000
(GRADE 60)	60,000

STEP 4 COMPUTE LIVE LOAD SHEARS AND MOMENTS

Load cases are same as those on Plate III in Appendix A; therefore, we use the table on Plate I in Appendix A to obtain maximum Live Load Moment for each loading case.

Live Load Shear for each load case is computed by the method given in Section 3.3.

By placing the SU2 load case on the span in the opposite direction, the 12 kip load would be at the point where the shear is being computed rather than the 22^k load, thus yielding a smaller Live Load Shear.

DATE	DESIGN	Sec. 6.3.2 - Example Computations		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Reinforced Concrete T-Beam Superstructure

Span Length = 30.0 Ft.

Live Load Moments From Plate I

SU2 = 183.0 ft.-Kips

SU3 = 331.0 ft.-Kips

SU4 = 358.5 ft.-Kips

C3 = 198.5 ft.-Kips

C4 = 285.8 ft.-Kips

C5 = 285.8 ft.-Kips

H = 246.6 ft.-Kips

HS = 282.1 ft.-Kips

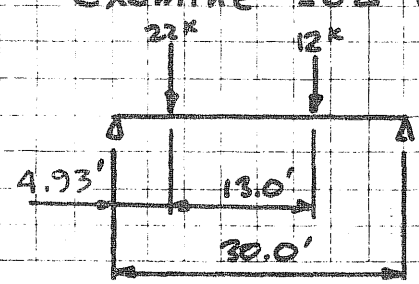
Live Load Shear

X = ½ (Pier Cap Width) + (d)

$X = \frac{1}{2}(3.5 \text{ ft.}) + (38.15 \text{ in.})\left(\frac{1 \text{ ft.}}{12 \text{ in.}}\right)$

X = 4.93 ft.

Examine SU2 Load Case



$LLV = 22\left(1 - \frac{4.93}{30}\right) + 12\left(1 - \frac{4.93 + 13.0}{30}\right)$

$LLV = \boxed{23.21^K}$

REV. 04/82

Note that by placing the SU3 load case on the span in the opposite direction, the middle load would be farther from the point where shear is being computed, thus yielding a smaller Live Load Shear.

Note that by placing the SU4 load case on the span in the opposite direction, the 13.9 kip load would fall at the point where shear is being checked rather than the 18.7 kip load. Thus the 18.7 kip load would be farther from the point where shear is being computed and would contribute to a smaller Live Load Shear.

Note that by placing the C3 load case on the span in the opposite direction two conditions may exist. 1) The 12 kip load will replace the 22^k load at the point where shear is being checked or 2) Both 22 kip loads are placed on the span. Either condition may be shown to yield a smaller Live Load Shear than the placement shown on the following page.

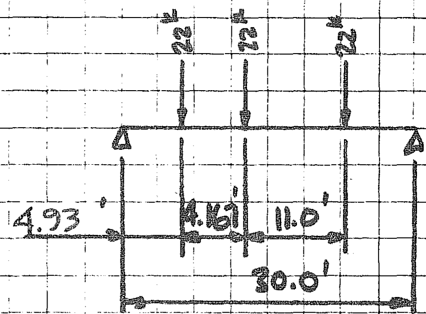
Note that by placing the C4 load case on the span in the opposite direction, the 7.271 kip load would fall at the point where shear is being checked rather than a 22 kip load, and the remaining 22 kip load would be farther from the point where shear is being computed, both of which would contribute to a smaller Live Load Shear.

DATE | DESIGN | CHECK | JOB | FOR | SHEET | OF | JOB NO.

SEC. 6.3.2- EXAMPLE COMPUTATIONS CONT.

SUBJECT

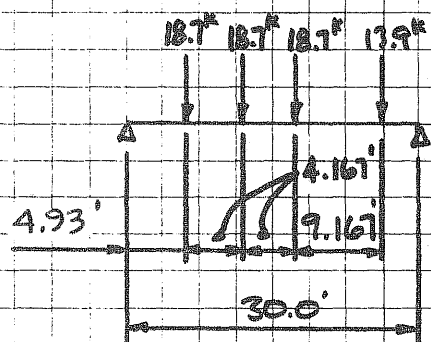
Examine SU3 Load Case



$$LLV = 22\left(1-\frac{4.93}{30}\right) + 22\left(1-\frac{4.93+4.167}{30}\right) + 22\left(1-\frac{4.93+4.167+11.0}{30}\right)$$

$$LLV = \boxed{40.96^{k}}$$

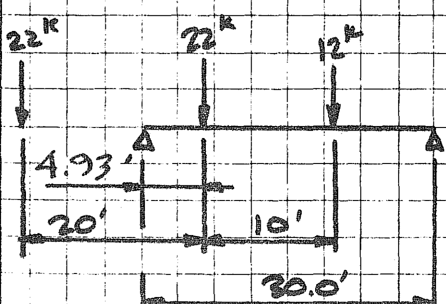
Examine SU4 Load Case



$$LLV = 18.7\left(1-\frac{4.93}{30}\right) + 18.7\left(1-\frac{4.93+4.167}{30}\right) + 18.7\left(1-\frac{4.93+4.167+4.167}{30}\right) + 13.9\left(1-\frac{4.93+4.167+4.167+9.167}{30}\right)$$

$$LLV = \boxed{42.60^{k}}$$

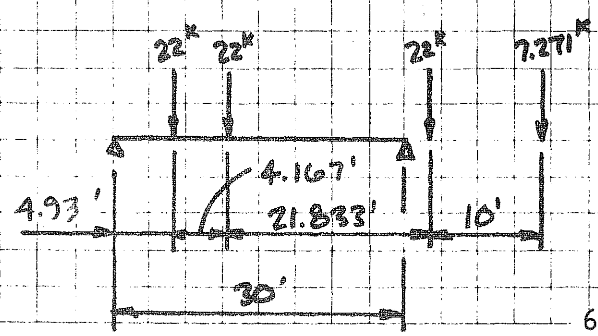
Examine C3 Load Case



$$LLV = 22\left(1-\frac{4.93}{30}\right) + 12\left(1-\frac{4.93+10}{30}\right)$$

$$LLV = \boxed{24.41^{k}}$$

Examine C4 Load Case



$$LLV = 22\left(1-\frac{4.93}{30}\right) + 22\left(1-\frac{4.93+4.167}{30}\right)$$

$$LLV = \boxed{33.71^{k}}$$

For the C5 load case both a two and three axle placement are possible. Since it is not obvious which would yield a larger Live Load Shear, both must be examined and the greater computed Live Load Shear used.

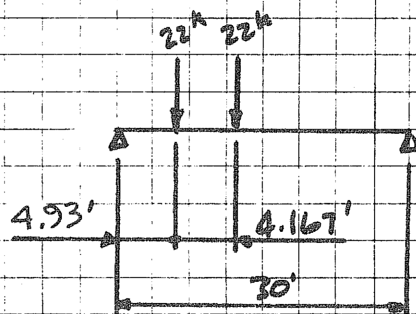
Note that by placing the H load case on the span in the opposite direction, the 8 kip load would fall at the point where shear is being computed and would yield a smaller Live Load Shear.

Note that by placing the HS load case on the span in the opposite direction, the 8 kip load would fall at the point where shear is being computed and would yield a smaller Live Load Shear.

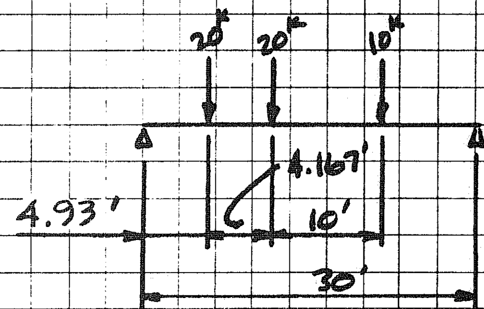
DATE	DESIGN	SEC. 6.3.2- EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Examine C5 Load Case



OR

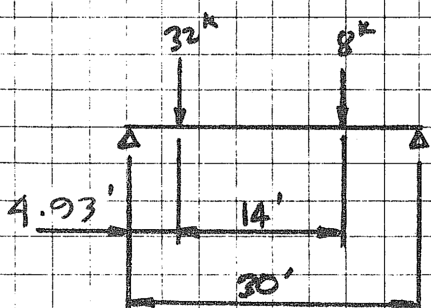


$$LLV = 22\left(1-\frac{4.93}{30}\right)+22\left(1-\frac{4.93+4.167}{30}\right)$$
$$= 33.71^k$$

$$LLV = 20\left(1-\frac{4.93}{30}\right)+20\left(1-\frac{4.93+4.167}{30}\right)$$
$$+10\left(1-\frac{4.93+4.167+10}{30}\right)$$
$$= 34.28^k$$

use $LLV = \boxed{34.28^k}$

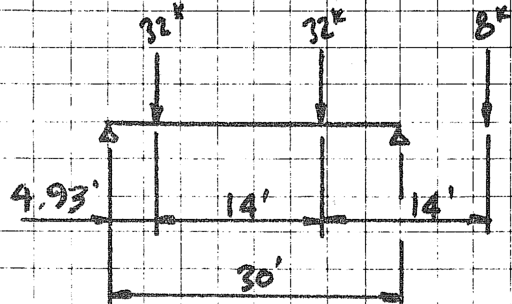
Examine H Load Case



$$LLV = 32\left(1-\frac{4.93}{30}\right)+8\left(1-\frac{4.93+14}{30}\right)$$

$$LLV = \boxed{29.69^k}$$

Examine HS Load Case



$$LLV = 32\left(1-\frac{4.93}{30}\right)+32\left(1-\frac{4.93+14}{30}\right)$$

$$LLV = \boxed{38.55^k}$$

REV. 04/82

SUMMARY OF MAXIMUM LIVE LOAD SHEARS

SU2	=	23.21	Kips
SU3	=	40.96	Kips
SU4	=	42.60	Kips
C3	=	24.41	Kips
C4	=	33.71	Kips
C5	=	34.28	Kips
H	=	29.69	Kips
HS	=	38.55	Kips

STEP 5 PERFORM RATING CALCULATIONS

The rating calculations are made in accordance with the procedures given in Section 6.2.

1. COMPUTE DEAD LOAD PER LINEAR FOOT OF SLAB AND DEAD LOAD PER LINEAR FOOT OF T-BEAM. Obtain a photocopy of the forms headed "Section 12.3 - Dead Load Computation" and complete the computations detailed thereon.

Use 6" x 5.7 ft. raised portion as curb.
Note 5.7 ft. = 5.7 ft(12 in/ft) = 68.4 in.

Total Dead Load per one foot strip of slab is equal to the sum of the dead loads for the asphalt wearing surface and the slab.

SECTION 6.3.3 - DEAD LOAD COMPUTATION

Dead Load (DL) per linear foot for slab:

Asphalt Wearing Surface:

(Thickness)(144 lb/ft^3)(1/12)

= (1.5 in.)(144 lb/ft^3)(1/12) = 18.00 lb/ft per 1 ft. strip

Slab:

(Thickness)(150 lb/ft^3)(1/12)

= (7.5 in.)(150 lb/ft^3)(1/12) = 93.75 lb/ft per 1 ft. strip

Total DL per foot per 1 ft. strip (WS_{DL}) for Slab

= $\boxed{111.75}$ lb/ft per 1 foot strip

Dead Load per linear foot for T-beam:

Barrier:

(Rail Width)(Rail Height)

(6 in.)(10 in.) (1/144) = 0.42 ft.^{2} -(1)

(Curb Width)(Curb Height)

(68.4 in.)(6 in.)(1/144) = 2.85 ft.^{2} -(2)

(Rail Post Length)(Height)(Width)(1/Avg. Post Spacing)

(3.0 ft.)(8 in.)(10 in.)(1/7.5 ft.)(1/144)

= 0.22 ft.^{2} -(3)

The intent of the barrier computations is to calculate the combined weight per linear foot of barriers and then divide it by the number of T-Beam sections in order to arrive at a weight per linear foot of a T-beam section. The barrier configuration illustrated is a general concrete curb, rail and railpost type barrier, but actual configurations of existing barriers may vary somewhat.

For barriers constructed of concrete that differ in configuration from the barrier indicated, the average barrier area should be calculated and a unit weight of 150 lb/ft^3 should be used to compute a per foot weight of each barrier. The total per foot weight of all barriers should then be divided by the number of T-beam sections to obtain the per foot dead load. For barriers made of steel, the area of steel per linear foot should be computed and a unit weight of 490 lbs/ft^3 used. If standard rolled sections are used, the weights per linear foot may be obtained from the AISC manual.* For aluminum, a unit weight of 175 lb/ft^3 is to be used and for timber a unit weight of 50 lb/ft^3 is to be used.

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*American Institute of Steel Construction, Inc., Manual of Steel Construction, Part I.

Barrier Weight per Foot:

= [(1) + (2) + (3)](150 lb/ft^3)(Number of Barriers)(1/Number of T-Beam Sections)

= [(0.42ft^2) + (2.85ft^2) + (0.22ft^2)](150)(2)(1/ 5 ft.)

= $\boxed{209.40}$ lb per foot.

Determine Nominal Flange Width (s):

s = (Beam Spacing)(12)

= (6.0 ft)(12) = 72 in.

Asphalt Wearing Surface:

(S)(Thickness)(144 lb/ft^3)(1/144)

= (72 in)(1.5 in) = $\boxed{108.00}$ lb/ft.

Concrete Slab:

(S)(Thickness)(150 lb/ft^3)(1/144)

= (72 in)(7.5 in)(150)(1/144) = $\boxed{562.50}$ lb/ft

Beam:

Beam Width = 1.333 ft. (12 in./ft.) = 16 in.
Beam Height = 3.0 ft. (12 in./ft.) = 36 in.

(Beam Width)(Beam Height)(150 lb/ft^3)(1/144)

= (16.0 in)(36.0 in)(150)(1/144) = $\boxed{600.00}$ lb/ft.

Total Dead Load per foot of T-Beam Section is equal to the sum of the Dead Loads from the barrier, asphalt wearing surface, concrete slab, and beam.

Total DL per foot =

Barrier Weight Per Foot	=	209.40
+ Asphalt Wearing Surface	=	108.00
+ Concrete Slab	=	562.50
+ Beam	=	600.00
		1,479.90

Total DL per foot of T-Beam Section (W_{DL}) = $\boxed{1479.90}$ lb./ft.

The APPLIED DEAD LOAD MOMENT is the moment resulting from the weight of the components making up the structure (in this case the weight of the wearing surface and slab).

Ab = area of reinforcing bar. See Plate IV in Appendix A for area of bar, given the bar designation from the Inspection Data Sheet.

Max. spacing for transverse reinf. = the greater of the top or bottom transverse reinforcing spacing.

SECTION 6.3.4 - MOMENT CAPACITY RATING FOR SLAB

Calculate Applied Dead Load Moment (M_{DL}):

Effective Slab Span (S) = (Beam Spacing) - (Beam Width)

s = (6 ft) - (16 in)(1/12) = 4.667 ft.

$$M_{DL} = (1/10)(WS)_{DL}(S)^2(1/1000)$$

$$= (1/10)(1/1000)(\ 111.75\ \text{lb/ft})(\ 4.667\ \text{ft})^2 = \boxed{0.243}\ \text{ft-kips}$$

Determine Area of Reinforcing Steel (A_s) to be used in computing Moment Capacity of Slab

If, as most instances should, the actual area of steel (A_s) is less than the balanced steel area (A_{sb}), the lesser of the actual A_s or .75 A_{sb} should be used.

If the actual area of steel (A_s) is greater than the balanced steel area (A_{sb}), compression controls the analysis and a professional engineer should be consulted.

Actual A_s = (Ab)(1/Max. Spacing for Transverse Reinf.)(12)

= (0.20 in^2)(1/ 9 in)(12) = 0.27 in^2/ft.

c = distance from the neutral axis of a member in flexure to the extreme compressive fiber.

β = a factor describing the stress curve as a function of c. β shall be taken as 0.85 for concrete strengths (f'c) up to 4,000 psi and shall be reduced continuously at a rate of 0.05 for each 1,000 psi in excess of 4,000 psi.

d = distance from the top slab to the centerline of the bottom reinforcing steel. The top of slab should be taken as the top of sound material taking into account scaling or spalling.

β is taken as 0.85 since f'c is less than 4,000 psi.

SECTION 6.3.4 - MOMENT CAPACITY RATING FOR SLAB (CONT.)

Calculate A_{sb}:

$$A_{sb} = \frac{.85(\beta)(f'c)(d)(12)}{(fy)} \quad \frac{87000}{87000 + fy}$$

$$.75\ A_{sb} = \frac{(.75)(.85)(.85)(3000\ \text{psi})(5.75\ \text{in})(12)}{(33000\ \text{psi})} \quad \frac{87000}{87000 + (33000\ \text{psi})}$$

$$.75\ A_{sb} = \boxed{2.46}\ \text{in}^2/\text{ft.}$$

now, taking the lesser of the A_s or $.75\ A_{sb}$, $A_s = \boxed{0.27}$ in^2/ft.

Determine Ultimate Moment Capacity (Mu) of Slab:

$$Mu = \frac{(A_s)(fy)(d)}{12} - \frac{(A_s)^2(fy)^2}{2(.85)(f'c)(144)}$$

$$Mu = \frac{(0.27\ \text{in}^2)(33{,}000\ \text{psi})(5.75\ \text{in})}{12} - \frac{(0.27\ \text{in}^2)^2(33{,}000\ \text{psi})^2}{(1.7)(3000\ \text{psi})(144)}$$

$$Mu = \underline{4161}\ \text{ft-lbs} \div 1000 = \boxed{4.16}\ \text{ft-kips.}$$

Notice that the numerator in the Rating Factor equations does not change for the different loading cases, therefore it will need to be calculated only once. The denominator changes and must be recalculated for each different loading case.

Calculate Inventory and Operating Moment Ratings:

The following formulae are to be used to compute the Inventory and Operating Moment Ratings for each loading case. The variable "P" used in the rating equations is Live Load of one rear wheel (ie., 1/2 the axle load) of each rating truck. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the Gross Vehicle Weight (GVW) to obtain the rating. This rating should then be entered under the appropriate type of loading in the "Summary of Ratings" table in Chapter 8, under the heading "Moment Rating for Slab".

Inventory Rating:

$$RF = \frac{\left[\frac{(0.9)(Mu)}{1.3} - M_{DL}\right]\left(\frac{3}{5}\right)}{\left(\frac{(s)+2}{32}\right)(P)(1.30)(0.8)}$$

Operating Rating:

$$RF = \frac{\left[\frac{(0.9)(Mu)}{1.3} - M_{DL}\right]}{\left(\frac{(s)+2}{32}\right)(P)(1.30)(0.8)}$$

Note that for each loading case only the value of "P" is subject to change. Furthermore, since P for the SU3 load case is the same as P for the SU2 load case, the Rating Factors are identical.

CALCULATE RATINGS FOR SLAB

CALCULATE NUMERATOR OF INVENTORY RATING FACTOR EQUATION:

$$\left[\frac{(0.9)(4.16)}{1.3} - 0.243\right]\left(\frac{3}{5}\right) = 1.582$$

CALCULATE NUMERATOR OF OPERATING RATING FACTOR EQUATION:

$$\left[\frac{(0.9)(4.16)}{1.3} - 0.243\right] = 2.637$$

SU2 : INV. $\dfrac{1.582}{\left(\frac{4.667+2}{32}\right)(11.0)(1.30)(0.8)} = 0.664$

$34.0\ (0.664) = 22.6$ Kips

Op. $\dfrac{2.637}{\left(\frac{4.667+2}{32}\right)(11.0)(1.30)(0.8)} = 1.11$

$34.0\ (1.11) = 37.6$ Kips

SU3 : INV. $= 0.664$

$66.0\ (0.664) = 43.8$ Kips

Op. $= 1.11$

$66.0\ (1.11) = 73.3$ Kips

Note the wheel load for C3, C4 and C5 is equal to the wheel load for SU2, thus, the Rating Factors are the same.

DATE	DESIGN	Sec. 6.3.4-Example Computations Cont.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

SU4 : Inv. $\dfrac{1.582}{\left(\frac{4.667+2}{32}\right)(9.35)(1.30)(.8)} = 0.781$

70.0 (0.781) = 54.6 Kips

Op. $\dfrac{2.637}{\left(\frac{4.667+2}{32}\right)(9.35)(1.30)(.8)} = 1.30$

70.0 (1.30) = 91.0 Kips

C3 : Inv. = 0.664

56.0 (0.664) = 37.2 Kips

Op. = 1.11

56.0 (1.11) = 62.2 Kips

C4 : Inv. 73.271 (0.664) = 48.6 Kips

Op. 73.271 (1.11) = 81.3 Kips

C5 : Inv. 80.0 (0.664) = 53.1 Kips

Op. 80.0 (1.11) = 88.8 Kips

H : Inv. $\dfrac{1.582}{\left(\frac{4.667+2}{32}\right)(16.0)(1.30)(.8)} = 0.456$

40 (0.456) = 18.3 Kips

Op. $\dfrac{2.637}{\left(\frac{4.667+2}{32}\right)(16.0)(1.30)(.8)} = 0.761$

40 (0.761) = 30.4 Kips

Note that the wheel load for H and HS load cases are the same, so use the H load case Rating Factors for the HS load case.

DATE	DESIGN	Sec. 6.3.4-Example Computations Cont.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

HS : Inv. = 0.456

Rating = 0.456 (72.0) = 32.8 Kips

Op. = 0.761

Rating = 0.761 (72.0) = 54.8 Kips

STEP 5 3. DETERMINE OPERATING AND INVENTORY RATINGS OF T-BEAM FOR MOMENT CAPACITY.

A photocopy of the forms "Section 12.3 - Moment Capacity Rating for T-Beam" should be obtained and the ratings determined by performing the computations detailed thereon. All T-Beams are of the same configuration and none have differing amounts of deterioration; therefore a representative T-Beam is chosen to be evaluated.

A_b is the reinforcing bar diameter. Since we have two different sizes (1-1/8" and 1-1/4") use the weighted average of the two areas.

$$\frac{(\text{Number of Bars in Row 1})\ (A_b) + (\text{Number of Bars in Row 2})\ (A_b)}{(\text{Total Number of Bars})}$$

$$\frac{(\ 3\ )\ (\ 1.56\ ) + (\ 3\ )\ (\ 1.27\ )}{(\ 6\ )} = 1.415 \text{ in}^2$$

Note that bar areas are taken from Plate IV in Appendix A.

The APPLIED DEAD LOAD MOMENT is the moment resulting from the weight of the components making up the structure (in this case the weight of the wearing surface, barriers, and T-beam itself).

If, as most instances should, the actual area of steel (A_s) is less than the balanced steel area (A_{sb}), the lesser of the actual A_s or .75 A_{sb} should be used.

If the actual area of steel (A_s) is greater than the balanced steel area (A_{sb}), compression controls the analysis and a professional engineer should be consulted.

A_b = Area of reinforcement bar. See Plate IV in Appendix A for area of bar, given the bar designation from the Inspection Data Sheet.

A_{sf} = Area of reinforcement to develop compressive strength in overhanging flanges of T-Section.

W_b = Width of T-Beam from Reinforced Concrete T-Beam Superstructure Inspection Data Sheet.

SECTION 6.3.5 - MOMENT CAPACITY RATING FOR T-BEAM

Calculate Applied Dead Load Moment(M_{DL}):

$$M_{DL} = (1/8)(W_{DL})(\text{Span Length})^2(1/1000)$$

$$M_{DL} = (1/8)(\ 147990\ \text{lb/ft})(\ 30\ \text{ft})^2(1/1000) = \boxed{166}\ \text{ft-kips}$$

Determine Effective Flange Width (s):

$$s = \text{Beam Spacing} = (\ 6.0\ \text{ft})(12\ \text{in./ft}) = \boxed{72}\ \text{in.}$$

But not greater than:

$$\frac{\text{Span Length}}{4} = \frac{(\ 30\ \text{ft})}{4}\ \frac{12\ \text{in}}{\text{ft.}} = \underline{\ 90\ }\ \text{in.}$$

And not greater than:

$$Wb + 12(\text{Slab Thickness}) = (\ 16\ \text{in}) + 12(\ 7.5\ \text{in}) = \underline{\ 106\ }\ \text{in}$$

$$\text{Use } s = \boxed{72}\ \text{in.}$$

Determine Area of Reinforcing Steel (A_s) to be used in computing Moment Capacity of the member:

Actual $A_s = (A_b)(\text{number of bars})$

(for reinforcement in bottom of beam)

$$= (\ 1.415\ \text{in}^2)(\ 6\ ) = \boxed{8.49}\ \text{in}^2$$

Locate Neutral Axis:

$$c = \frac{(A_s)\ (f_y)}{.85\ (f'_c)\ (\beta)\ (s)}$$

$$c = \frac{(\ 8.49\ \text{in}^2)\ (\ 33{,}000\ \text{psi.})}{.85\ (\ 3000\ \text{psi.})\ (\ .85\ )\ (\ 72\ \text{in.})} = \underline{\ 1.795\ }\ \text{in.}$$

If c is greater than the slab thickness:

NOTE: Since c ≤ to slab thickness, A_{sf} was calculated on the previous page and the formulas on this page do not apply.

$$A_{sf} = \frac{.85\ (f'c)(s-W_b)(\text{slab thickness})}{f_y}$$

$$A_{sf} = \frac{.85\ (\quad \text{psi})(\quad \text{in} - \quad \text{in})(\quad \text{in})}{(\quad \text{psi})}$$

A_{sf} = ______________ in^2

Calculate A_{sb}:

$$A_{sb} = \left[\frac{.85(\beta)(f'c)(d)(W_b)}{fy}\ \frac{87000}{87000 + fy} + A_{sf}\right] \frac{W_b}{s}$$

$$.75\ A_{sb} = .75\ \left[\frac{(.85)(\quad)(\quad \text{psi})(\quad \text{in.})(\quad \text{in.})}{(\quad \text{psi})}\right.$$

$$\left.\frac{87000}{87000 + (\quad \text{psi})} + (\quad \text{in.}^2)\right] \frac{(\quad \text{in})}{(\quad \text{in})}$$

$.75A_{sb}$ = $\boxed{\quad\quad}$ in^2

If the actual area of steel (A_s) is less than the balanced steel area (A_{sb}), the lesser of the actual A_s or .75 A_{sb} should be used.

If the actual area of steel (A_s) is greater than the balanced steel area (A_{sb}), a professional engineer should be consulted.

now, taking the lesser of A_s or .75 A_{sb}, A_s = $\boxed{\quad\quad}$ in^2

Determine Ultimate Moment Capacity (Mu) of T-Beam:

The formulas on this page do not apply since $c \le$ slab thickness. Mu calculated on page 6-48.

$$c = \frac{(A_s - A_{sf})(fy)}{.85(f'c)(\beta)(W_b)}$$

$$= \frac{(\quad in^2 - \quad in^2)(\quad psi)}{.85(\quad psi)(\quad)(\quad in)} = \_\_\_\_\_\_\_\_\_ \text{ in.}$$

$a = (\beta)(c)$

$a = (\quad)(\quad in) = \_\_\_\_\_\_\_\_\_$ in.

$$M_u = [(A_s - A_{sf})(fy)(d - \frac{a}{2}) + (A_{sf})(f_y)(d - \frac{\text{slab thickness}}{2})]$$

$$M_u = [(\quad in^2 - \quad in^2)(\quad psi)(\quad in - \frac{in}{2}) + (\quad in^2)$$

$$(\quad psi)(\quad in - \frac{in}{2})]\ (\frac{1}{12})$$

M_u = __________ ft-lbs ÷ 1000 = ___________ ft-kips

GVW = Gross Vehicle Weight from Section 3.2 for each loading type (kips)

LLM = Applied Live Load Moment from Section 3.2 for each loading type (ft-kips)

The following formulae are used to compute the Inventory and Operating Moment Capacity Ratings for each load case. Live Load Moment (LLM) used in the formulae is from Section 3.2 of this manual. Notice that the numerator of the Rating Factor equations do not change for the different loading cases, therefore it will need to be calculated only once. The denominator changes and must be recalculated for each different loading case. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the GVW to obtain the Rating Value. This Rating Value should then be entered under the appropriate type of Loading in the "Summary of Ratings" table in Chapter 8 under the heading "Moment Rating for T-Beam".

Inventory Rating:

$$RF = \frac{\left[\frac{(0.9)(Mu)}{(1.3)} - M_{DL}\right]\left(\frac{3}{5}\right)}{(1/2)(DF)(LLM)(I)}$$

Operating Rating:

$$RF = \frac{\left[\frac{(0.9)(Mu)}{1.3} - M_{DL}\right]}{(1/2)(DF)(LLM)(I)}$$

Referring to Plate III in Appendix A it is found that for a 30.0 ft. span a two axle placement yields maximum moment. Accordingly, a GVW of 80.0 kips is used.

SU4 : Inv. $\dfrac{263.03}{(1/2)(1.0)(358.5)(1.3)} = 1.13$

$70.0\ (1.13) \qquad = 79.1$ Kips

Op. $\dfrac{438.38}{(1/2)(1.0)(358.5)(1.3)} = 1.88$

$70.0\ (1.88) \qquad = 131.6$ Kips

C3 : Inv. $\dfrac{263.03}{(1/2)(1.0)(198.5)(1.3)} = 2.04$

$56.0\ (2.04) \qquad = 114.2$ Kips

Op. $\dfrac{438.38}{(1/2)(1.0)(198.5)(1.3)} = 3.40$

$56.0\ (3.40) \qquad = 190.4$ Kips

C4 : Inv. $\dfrac{263.03}{(1/2)(1.0)(285.8)(1.3)} = 1.42$

$73.271\ (1.42) \qquad = 104.0$ Kips

Op. $\dfrac{438.38}{(1/2)(1.0)(285.8)(1.3)} = 2.36$

$73.271\ (2.36) \qquad = 172.9$ Kips

C5 : Inv. $\dfrac{263.03}{(1/2)(1.0)(285.8)(1.3)} = 1.42$

$80.0\ (1.42) \qquad = 113.6$ Kips

Op. $\dfrac{438.38}{(1/2)(1.0)(285.8)(1.3)} = 2.36$

$80.0\ (2.36) \qquad = 188.8$ Kips

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SUBJECT

H : Inv. $\frac{263.03}{(1/2)(1.0)(246.6)(1.3)} = 1.64$

40.0 (1.64) = 65.6 Kips

Op. $\frac{438.38}{(1/2)(1.0)(246.6)(1.3)} = 2.73$

40.0 (2.73) = 109.2 Kips

HS : Inv. $\frac{263.03}{(1/2)(1.0)(282.1)(1.3)} = 1.43$

72.0 (1.43) = 103.0 Kips

Op. $\frac{438.38}{(1/2)(1.0)(282.1)(1.3)} = 2.39$

72.0 (2.39) = 172.1 Kips

W_{DL} = Total dead load per foot for T-Beam Section from Section 6.3.3

x = Distance from the support to the point where shear is being checked, from Section 3.3.

There are two acceptable methods of computing v_c. Method one is the simpler method, but yields a somewhat conservative value for permitted shear stress. Method two is more involved and yields a higher permitted shear stress than does method one. Obviously we want to limit the amount of computation, so method one should be used and the ratings computed and compared with the ratings from the other components of the structure. If the shear capacity rating is <u>not</u> the controlling rating, the shear capacity rating is finished. However, if the shear capacity rating <u>is</u> the controlling rating, method two must be used to compute v_c and the ratings must be recalculated. Method two need be used <u>only</u> for those loading cases where the Shear Capacity Ratings control as a result of using method one.

Obviously we could always use method two; then there would be no need to recalculate v_c or the Rating Values. However, in a great many instances the Moment Capacity Rating for the T-Beam is more critical than the Shear Capacity Rating, even by using the more conservative value of v_c from method one. Since method two requires a great deal more computation than does method one, a substantial amount of time may be saved by using the suggested procedure.

STEP 5 4. DETERMINE OPERATING AND INVENTORY RATINGS OF T-BEAM FOR SHEAR CAPACITY.

A photocopy of the forms "Section 12.3 - Shear Capacity Rating for T-Beam" should be obtained and the ratings determined by performing the computation detailed thereon. All T-Beams are of the same configuration and none have differing amounts of deterioration; therefore, a representative T-Beam is chosen to be evaluated.

As previously suggested, method one will be used to determine the Shear Ratings. When the rating values are compared in the Summary of Ratings Table, those cases in which the Shear Rating controls will be reevaluated using method two.

SECTION 6.3.6 - SHEAR CAPACITY RATING FOR T-BEAM

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = (W_{DL})[(\text{Span Length})(1/2) - (x)](1/1000)$$

$$V_{DL} = (1479.9\,\text{lb/ft})[(30.0\,\text{ft.})(1/2) - (5.375\,\text{ft.})](1/1000)$$

$$V_{DL} = \boxed{14.24}\ \text{kips}$$

Calculate Impact Factor (I):

$$I = 1 + \frac{50}{(\text{Span Length}) - (x) + (125)}\text{, but not greater than 1.30}$$

$$I = 1 + \frac{50}{(30\,\text{ft.}) - (5.375\,\text{ft.}) + (125)}$$

I = 1.33 Use I = $\boxed{1.3}$

Determine the maximum permitted Shear Stress for concrete (v_c):

Method One:

$$v_c = 2\sqrt{f'c}$$

$$v_c = 2\sqrt{3000\ \text{psi}} = \underline{109.54}\ \text{psi}$$

Method Two:

$$v_c = 1.9\sqrt{f'c} + 2{,}500\ \frac{A_s}{12d}\ \frac{(V_{DL} + LLV)d}{M'u}$$ ← Equation [A]

but not greater than $3.5\sqrt{f'c}$ and the quantity $\frac{(V_{DL} + LLV)d}{M'u}$

shall not exceed 1.0.

First, Compute M'u (the factored M_{DL} + LLM occurring simultaneously with V_{DL} + LLV at the section being considered).

Contribution of M'_{DL} to M'u

$$M'_{DL} = (1/2)(W_{DL})(x)(\text{Span Length} - x)$$

$$M'_{DL} = (1/2)(\text{—}\ \text{lb/ft.})(\text{—}\ \text{ft.})(\text{—}\ \text{ft.} - \text{—}\ \text{ft.})$$

$$M'_{DL} = \underline{\quad\text{—}\quad}\ \text{ft.-lbs.}$$

Contribution of LLM to M'u shall be determined by evaluating the moment resulting from the axle placements used to compute LLV for each loading case. Shown below is a general equation to calculate the Live Load Moment for these axle placements. If the load case being examined has less than five axles on the span, simply delete the parts of the equation pertaining to the unused axles. Example: If three axles are being used, delete the terms:

$$\frac{P_4x}{L}(L-x-a-b-c) \text{ and } \frac{P_5x}{L}(L-x-a-b-c-d).$$

General Equation:

$$LLM = \frac{P_1x}{L}(L-x) + \frac{P_2x}{L}(L-x-a) + \frac{P_3x}{L}(L-x-a-b) + \frac{P_4x}{L}(L-x-a-b-c) + \frac{P_5x}{L}(L-x-a-b-c-d)$$

This equation is to be evaluated for each loading case that was used in evaluating shear. The variables x,a,b,c, and d are the same as those used in the shear calculations that were completed as a part of Section 3.3.

Now for each loading case being evaluated, compute the value of the expression:

$$\frac{(V_{DL} + LLV)d}{(M'_{DL} + LLM)}, \text{ but not greater than } 1.0$$

Evaluating equation [A] for each load case we obtain the value of v_c to be used for each load case.

From Plate IV in Appendix A the area for a 1/2" bar is $0.20\ in^2$.

Wb = Width of T-Beam web from Section 2.4.

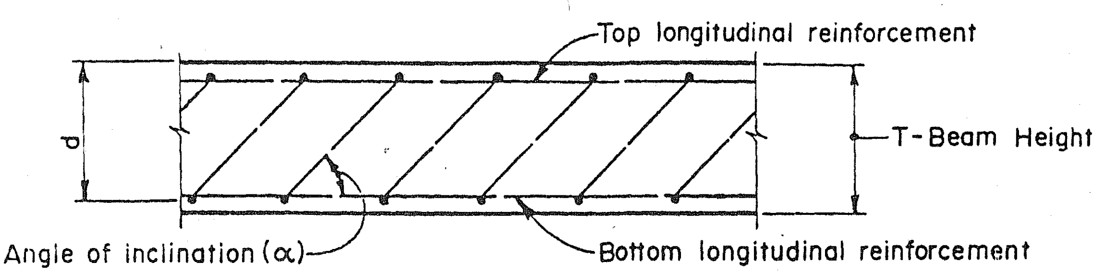
d = Distance from the centroid of the the bottom reinforcing steel to the top of the concrete slab, from Section 2.4.

Av = Area of stirrup. Areas for various bar sizes are tabulated on Plate IV in Appendix A.

s = Stirrup spacing from Section 2.4.

α = Angle of inclination of stirrups with the longitudinal axis of the T-Beam.

d = Distance from centroid of the bottom reinforcing steel to the top of the concrete slab, from Section 2.4.



LONGITUDINAL SECTION OF T-BEAM

Determine Shear carried by concrete:

$$V_c = (v_c)(W_b)(d)(1/1000)$$

$$V_c = (109.54\text{psi})(16 \text{ in.})(38.15\text{in.})(1/1000)$$

$$V_c = \boxed{66.86} \text{ kips}$$

Determine Shear carried by stirrups:

Case I: Stirrups placed perpendicular to longitudinal axis of T-Beam.

$$V_s = \frac{2(Av)(fy)(d)}{(s)(1000)}$$

$$V_s = \frac{2(0.20 \text{ in.}^2)(33000\text{psi})(38.15 \text{ in.})}{(9 \text{ in.})(1000)} = \boxed{55.95} \text{ kips}$$

Case II - Stirrups placed inclined with the longitudinal axis of the T-Beam.

$$V_s = \frac{2(Av)(fy)(\text{Sin a} + \text{Cos a})(d)}{(s)(1000)}$$

$$V_s = \frac{2(\text{—} \text{ in.}^2)(\text{—} \text{ psi.})(\text{—} + \text{—})(\text{—} \text{ in.})}{(\text{—} \text{ in.})(1000)}$$

$$V_s = \boxed{\text{—}} \text{ kips.}$$

GVW = Gross Vehicle Weight from Section 3.2 for each loading type (kips).

LLV = Live Load Shear from Section 3.2 for each loading type (kips).

DF = Distribution Factor from Section 6.3.5

SECTION 6.3.6 - SHEAR CAPACITY RATING FOR T-BEAM (CONT.)

Determine ultimate shear capacity of T-Beam (Vu):

$$Vu = V_c + V_s = (66.86 \text{kips}) + (55.95 \text{kips})$$

$$Vu = \boxed{122.81} \text{ kips.}$$

Calculate Inventory and Operating Shear Ratings for T-Beam:

The following formulae are to be used to compute the Inventory and Operating Shear Capacity Ratings for each load case. The Live Load Shear used in the formulae is from Section 3.2 of this manual. Notice that the numerator of the rating factor equation does not change for the different loading cases, therefore it will need to be calculated only once. The denominator changes and must be recalculated for each different loading case. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the GVW to obtain the Rating Value. This rating value should then be entered under the appropriate type of loading in the "Summary of Ratings" table in Chapter 8 under the heading "Shear Rating for T-Beam".

Inventory Rating

$$RF = \frac{\left[\frac{(0.85)(Vu)}{1.3} - V_{DL}\right](3/5)}{(1/2)(DF)(LLV)(I)}$$

Operating Rating

$$RF = \frac{\left[\frac{(0.85)(Vu)}{1.3} - V_{DL}\right]}{(1/2)(DF)(LLV)(I)}$$

Begin by computing the numerator for the Inventory and Operating Rating Factor equations.

DATE	DESIGN	SEC. 6.3.6-EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

CALCULATE SHEAR RATING FOR T-BEAM

CALCULATE NUMERATOR OF INVENTORY RATING FACTOR EQUATION :

$$\left[\frac{(0.85)(122.81)}{1.3} - 14.24\right]\left(\frac{3}{5}\right) = 39.64$$

CALCULATE NUMERATOR OF OPERATING RATING FACTOR EQUATION :

$$\left[\frac{(0.85)(122.81)}{1.3} - 14.24\right] = 66.06$$

SU2 : INV. $\frac{39.64}{(1/2)(1.0)(23.21)(1.3)} = 2.63$

$34.0\,(2.63) = 89.5$ Kips

OP. $\frac{66.06}{(1/2)(1.0)(23.21)(1.3)} = 4.38$

$34.0\,(4.38) = 148.9$ Kips

SU3 : INV. $\frac{39.64}{(1/2)(1.0)(40.96)(1.3)} = 1.48$

$66.0\,(1.48) = 97.7$ Kips

OP. $\frac{66.06}{(1/2)(1.0)(40.96)(1.3)} = 2.48$

$66.0\,(2.48) = 163.7$ Kips

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Referring to the Live Load Shear computations it is found that a three-axle placement was used in computing maximum Live Load Shear; therefore use GVW equal to 80.0 kips.

DATE	DESIGN	Sec. 6.3.6 - Example Computations Cont.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

SU4 : Inv. $\dfrac{39.64}{(1/2)(1.0)(42.60)(1.3)} = 1.43$

$70.0\ (1.43) \qquad = 100.1$ Kips

Op. $\dfrac{66.06}{(1/2)(1.0)(42.60)(1.3)} = 2.39$

$70.0\ (2.39) \qquad = 167.3$ Kips

C3 : Inv. $\dfrac{39.64}{(1/2)(1.0)(24.41)(1.3)} = 2.50$

$56.0\ (2.50) \qquad = 140.0$ Kips

Op. $\dfrac{66.06}{(1/2)(1.0)(24.41)(1.3)} = 4.16$

$56.0\ (4.16) \qquad = 232.9$ Kips

C4 : Inv. $\dfrac{39.64}{(1/2)(1.0)(33.71)(1.3)} = 1.80$

$73.271\ (1.80) \qquad = 131.9$ Kips

Op. $\dfrac{66.06}{(1/2)(1.0)(33.71)(1.3)} = 3.01$

$73.271\ (3.01) \qquad = 220.5$ Kips

C5 : Inv. $\dfrac{39.64}{(1/2)(1.0)(34.28)(1.3)} = 1.78$

$80.0\ (1.78) \qquad = 142.4$ Kips

Op. $\dfrac{66.06}{(1/2)(1.0)(34.28)(1.3)} = 2.96$

$80.0\ (2.96) \qquad = 236.8$ Kips

H : Inv. $\frac{39.64}{(1/2)(1.0)(29.69)(1.3)} = 2.06$

$40.0\ (2.06)$ $= 82.4$ Kips

Op. $\frac{66.06}{(1/2)(1.0)(29.69)(1.3)} = 3.42$

$40.0\ (3.42)$ $= 136.8$ Kips

HS : Inv. $\frac{39.64}{(1/2)(1.0)(38.55)(1.3)} = 1.58$

$72.0\ (1.58)$ $= 113.8$ Kips

Op. $\frac{66.06}{(1/2)(1.0)(38.55)(1.3)} = 2.63$

$72.0\ (2.63)$ $= 189.4$ Kips

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As the ratings for shear are tabulated, note that in each case the Moment Rating controls; therefore, the computation of Vc by method one was not overly conservative. Had the Shear Rating for any load case been lower than the Moment Rating, Vc would have had to be re-computed. Method two would have had to be employed only for those load cases where the Shear Rating controlled using method one for computing vc.

After computing the Moment and Shear Ratings, determining the cases where Shear Rating controls and re-evaluating the Shear Rating as necessary, the controlling rating is determined by choosing the lowest Inventory and Operating Levels for each type of loading and entering them under the column "Controlling Rating". This marks the completion of the Load Rating for the example problem.

SEC. 6.3.7-EXAMPLE COMPUTATIONS CONT.

Loading Classification	TYPE OF LOADING	RATING LEVEL	MOMENT CAPACITY OF SLAB	MOMENT CAPACITY OF T-BEAM	SHEAR CAPACITY OF T-BEAM								CONTROLLING RATING
Florida Legal Loading	SU2	Inventory	22.6	75.1	89.5								22.6
		Operating	37.6	125.5	148.9								37.6
	SU3	Inventory	43.8	80.5	97.7								43.8
		Operating	73.3	134.6	163.7								73.3
	SU4	Inventory	54.6	79.1	100.1								54.6
		Operating	91.0	131.6	167.3								91.0
	C3	Inventory	37.2	114.2	140.0								37.2
		Operating	62.2	190.4	232.9								62.2
	C4	Inventory	48.6	104.0	131.9								48.6
		Operating	81.3	172.9	220.5								81.3
	C5	Inventory		113.6	142.4								53.1
		Operating	88.8	188.8	236.8								88.8
Design Loading	H	Inventory	18.3	65.6	82.4								18.3
		Operating	30.4	109.2	136.8								30.4
	HS	Inventory	32.8	103.0	113.8								32.8
		Operating	54.8	172.1	189.4								54.8

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

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SUMMARY OF RATINGS

CHAPTER 7

PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM SUPERSTRUCTURE

SECTION 7.1 - OVERVIEW FOR THE STRUCTURAL RATING OF PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM SUPERSTRUCTURE

A Prestressed Concrete Channel and Slab System Superstructure, as the term is used in this manual, is a bridge having a superstructure composed of precast, prestressed concrete channel sections spanning between supports, placed side-by-side and topped with a cast-in-place concrete slab which is conventionally reinforced.

The use of prestressed concrete channels (or prestressed concrete construction in general) is well suited for bridge members since it largely overcomes the limitations of conventionally reinforced concrete members, particularly the presence of tension cracks due to large moments or shears. In conventionally reinforced members the amount of cracking (width and number of cracks) is roughly proportional to the steel strain. Therefore, the available high strength steels (on the order of 3 to 5 times the strength of ordinary reinforcing steels) cannot be utilized because excessive cracking with regard to appearance and corrosion protection would result. Prestressed concrete construction serves to minimize this cracking by applying forces to the member in advance of loading in such a manner as to introduce stresses that are opposite in sign to the stresses the member will be subject to during actual loading. Particularly, the concrete on the tensile side of the neutral axis is placed under an initial compressive stress so that the application of the actual loads will reduce this stress, but not to the point where tension cracks will form. In this manner the available high strength steels may be used to an advantage while maintaining acceptable levels of cracking and deflections in the member.[1].

Although prestressed concrete construction has served to overcome many of the limitations of conventionally reinforced concrete, similar factors cause deterioration, namely:

1. Salt Action - Spray from salt or brackish waterways contributes to weathering through recrystallization. Salt also increases water retention or may chemically attack the concrete if certain compounds are present.

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[1]The following references are cited for those interested in further enlightenment in prestressed concrete:

1. George Winter and Arthur H. Nilson, Design of Concrete Structures, 8th Ed. New York: McGraw-Hill Book Co., 1972, p. 108.

2. H. Kent Preston and Norman J. Sollenberger, Modern Prestressed Concrete, New York: McGraw-Hill Book Co., 1967, p. 1.

3. Narbey Khachaturian and German Gurfinkel, Prestressed Concrete, New York: McGraw-Hill Book Co., 1969, p. 1.

2. Differential Thermal Strains - Large temperature variations which set up severe differential strains between the surface and the interior of a concrete mass are an occasional cause of concrete deterioration. Aggregates with lower coefficients of thermal expansion than the cement paste may also set up high tensile stresses.

3. Unsound Aggregates - Structurally weak and/or readily cleavable aggregates are vulnerable to the weathering effects of moisture and cold.

4. Sulfate Compounds in Soil and Water - Sodium, magnesium, and calcium sulfates have a deletorious effect upon compounds in the cement paste and cause rapid deterioration of the concrete.

5. Leaching - Water seeping through cracks and voids in the hardened concrete leaches or dissolves the calcium hydroxide and other material compounds. The resultant of such action is either efflorescence or incrustation at the surface of the cracks.

6. Wear or Abrasion - Traffic abrasion and impact cause wearing of bridge decks.

7. Shrinkage and Flexure Forces - Both of these kinds of forces set up tensile stresses that result in cracks.

8. Corroded Reinforcing Steel or Prestressing Strands - The increase in volume due to corrosion exerts expansive pressure on concrete.

Deterioration caused by the factors described above will appear as three distinct types: Scaling, spalling, or cracking. Scaling is the gradual and continuing loss of surface mortar and aggregate over a widespread area. Scaling is the most critical form of deterioration since the effective depth of the member is reduced over a relatively large area. Spalling is a roughly circular or oval depression in the concrete surface. Spalling results from the separation and removal of a portion of the surface concrete revealing a fracture roughly parallel, or slightly inclined, to the surface. Usually, a portion of the depression rim is perpendicular to the surface and often reinforcing steel is exposed. When rating a structure based on the effective depth of a spalled area, it must be realized that since the spalled area is well defined and limited to a confined region (and repair is relatively straightforward) a low rating based solely on a spalled area may not reflect the true condition of the structure. For this reason, spalling is not as critical as scaling. Cracking is a linear fracture in the concrete. The Structural Rating of the superstructure as analyzed in this manual is not affected by cracking. It should be noted, however, that cracks extending through the entire member may have serious consequences on the Shear Rating, map cracking may necessitate a significant reduction of the Moment Capacity Rating, and other types of cracks may have varied results on the Structural Ratings. This manual does not attempt to compute a numerical Rating Value for members having cracks since the consequences of cracks are not well suited to generalizations and many factors affect the severity of the cracks effect on the rating. Therefore, cracking must be investigated on an individual basis by a Registered Professional Engineer. Any cracking noted on the Bridge Inspectors Report should be clearly noted on the Summary of Ratings sheet.

The analysis and rating of Prestressed Concrete Channel and Slab System Superstructures presented herein is in accordance with "Standard Specifications for Highway Bridges"[2] and "Manual for Maintenance Inspection of Bridges."[3] As a result of these specifications and generally accepted engineering design practices, the analysis and rating methods described in this section are subject to the following:

1. The analysis is valid for simple spans only.

2. The Moment Capacity Ratings for Prestressed Concrete members will be evaluated for strength and serviceability requirements.

3. The Shear Capacity Ratings for Prestressed Concrete members will be evaluated using Ultimate Strength Methods.[4]

4. Impact Loading is included.

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[2]Standard Specifications for Highway Bridges, Twelfth Edition, 1977, Interim Specifications Bridges, 1978, 1979, 1980.

[3]Manual for Maintenance Inspection of Bridges, 1978.

[4]For a discussion of the design assumptions imposed by employing Ultimate Strength Methods see Standard Specifications for Highway Bridges, p. 101 and 102. For additional description of the ultimate strength method (also called strength method or load factor method) see Design of Concrete Structures, Chapter 3.

5. Loss of prestress is accounted for by decreasing the initial prestress by 45,000 psi. This is in lieu of the more involved computations presented in AASHTO specifications.[5] Accounting for losses by decreasing initial prestress by 45,000 psi is accurate to an acceptable degree providing we have normal weight concrete, normal prestress levels and average exposure conditions. For individuals interested in methods of detailed computation of prestress losses, reference is made to Modern Prestressed Concrete[6] and Estimating Prestress Losses.[7].

6. In the analysis, it is assumed bond is developed between the prestressing strand and the concrete for the entire length of the channel section.

7. In the analysis, composite action between the channel section and the cast-in-place slab is assumed.

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[5] Standard Specifications for Highway Bridges, Twelfth Edition, 1977, p. 121.

[6] H. Kent Preston and Norman J. Sollenberger, Modern Prestressed Concrete, New York: McGraw Hill Book Co., 1967, p. 150.

[7] H. Kent Preston et al., "Estimating Prestress Losses," Concrete International: Design & Construction, June, 1979, Vol. 1, No. 6, Published by American Concrete Institute, p. 32.

SECTION 7.2 - PROCEDURE FOR RATING A PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM SUPERSTRUCTURE

The following steps should be followed to compute the Inventory and Operating Ratings for a Prestressed Concrete Channel and Slab System Superstructure. The Prestressed Concrete Channel and Slab System Inspection Data Sheets and calculaton of Live Load Shears and Moments should already have been computed by the procedures of Chapter 2 and Chapter 3.

STEP 1 CALCULATE DEAD LOAD PER LINEAR FOOT OF CHANNEL. This step is accomplished by completing the forms "Calculation of Dead Load Per Linear Foot of Channel", Section 7.3.3.

STEP 2 DETERMINE THE MOMENT CAPACITY RATING OF THE CHANNEL. This step is accomplished by completing the forms "Moment Capacity Rating at Inventory Level", Section 7.3.4 and "Moment Capacity Rating at Operating Level", Section 7.3.5.

Standard design practice is to proportion prestressed members so that the stresses in the concrete and steel at actual service conditions are within permissible limits. There is merit to this approach since the main purpose of prestressing is to improve the performance of the prestressed member at service conditions. Accordingly, the Inventory Rating will be based on an analysis evaluating the strength of the prestressed member for serviceability requirements. Additonally, AASHTO[8] specifies that members are to be designed based on behavior at all load stages that may be critical during the life of the structure. In order to account for sufficient strength should overloads occur, the maximum permissible steel percentage shall not exceed .30, as determined by AASHTO.[9] Accordingly, the Operating Rating will be based on an analysis evaluating the strength requirements of the prestressed concrete members.

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[8] Standard Specifications for Highway Bridges, Twelfth Edition, 1977, p. 119.

[9] Standard Specifications for Highway Bridges, Twelfth Edition, 1977, Articles 1.6.9 and 1.6.10, p. 126

Spalling of the channel's web below the bottom prestressing strand does not affect the moment capacity of the channel so long as corrosion has not significantly reduced the area of the prestressing strand. Where corrosion has progressed to a point where the area of the prestressing strand has been significantly reduced, that particular prestressing strand should be omitted from the analysis.

Spalling of the top part of the slab does affect the moment capacity of the member. This should be taken into account by using "d" measured from the stable concrete to centroid of the prestressing strands (ie., $d = W_d + \text{effective } T_f - Y_{na}$) and by using the effective T_f in the section property calculations.

Corrosion of the reinforcing steel in the slab portion does not affect the Moment Capacity of the member.

The channel section chosen to be evaluated should be the most deteriorated region within the middle one-half of the channel (See Figure 7-1). When varying degrees of deterioration exist between two or more channels and it is not clear which channel is most critical, all questionable channels should be evaluated by the methods of Section 7.3.4 and 7.3.5.

STEP 3 DETERMINE THE SHEAR CAPACITY RATING OF THE CHANNEL. This step is accomplished by completing the forms "Shear Capacity Ratings for Outer Quarters of Channel", Section 7.3.6 and "Shear Capacity Ratings for Middle Third of Channel", Section 7.3.7

Spalling of the web below the prestressing strands does not affect the Shear Capacity of the channel.

Corrosion of the prestressing strands does not affect the Shear Capacity Rating for the channel. Corrosion of the stirrups does affect the Shear Capacity Rating of a channel and should be accounted for by using the actual area of sound reinforcing steel for Av in the rating calculations.

d = Distance from the centroid of the prestressing force to the extreme compressive fiber

W_d = Web Depth

Effective T_f = Flange thickness, including cast-in-place concrete, that represents the most deteriorated area within the middle one-half of the member (see Figure 7-1).

Y_{na} = Distance from the bottom of the channel to the neutral axis of the channel.

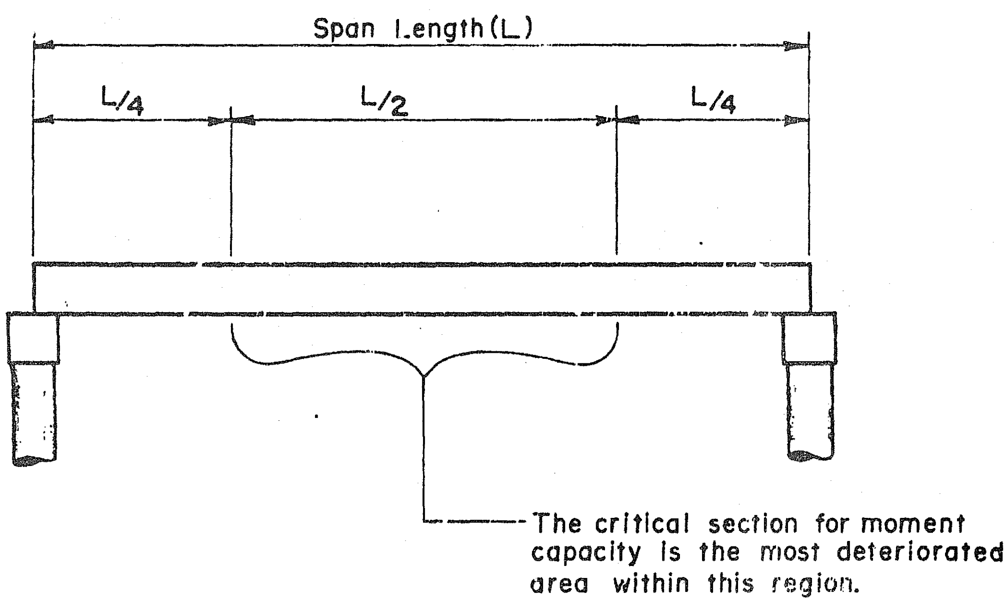


FIGURE 7-1

The channel section chosen to be evaluated should be the most deteriorated region within the L/4 of the support when checking shear in outer quarters (see Figure 7-2) and the most deteriorated region within the middle third when checking shear in the center of the channel (see Figure 7-3). When varying degrees of deterioration exist between two or more channels and it is not clear which channel is most critical, all questionable channels should be evaluated by the methods of Sections 7.3.6 and 7.3.7. For members where the stirrup spacing is uniform across the entire channel length, the shear need only be checked for the outer one quarter of the channel section.

This completes the Structural Rating calculations. The Rating Values should be tabulated in accordance with the instructions at the end of each section and those in Chapter 8.

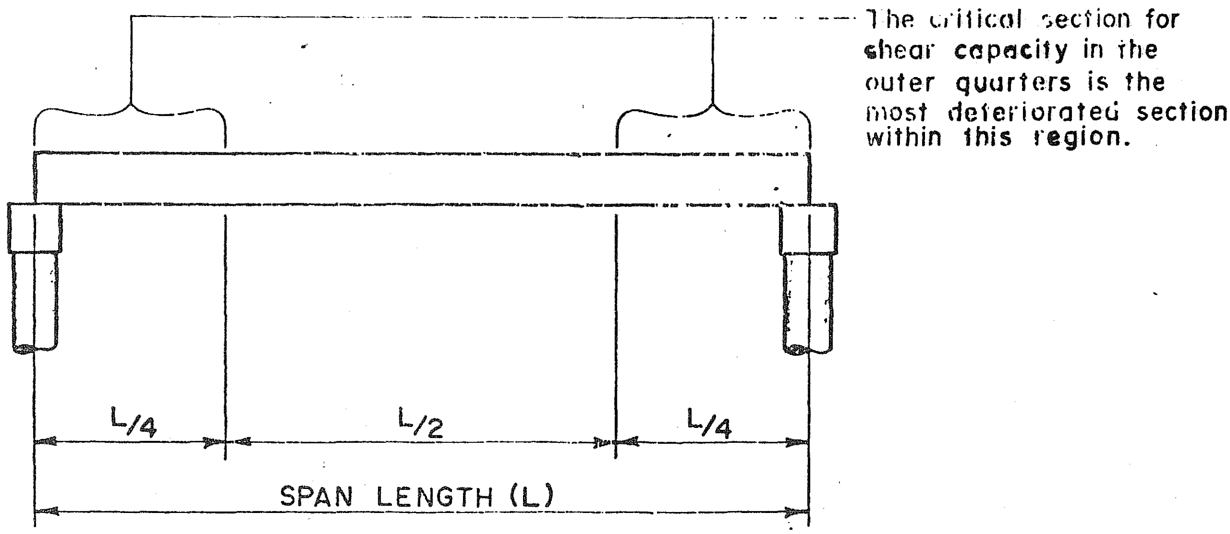


FIGURE 7-2

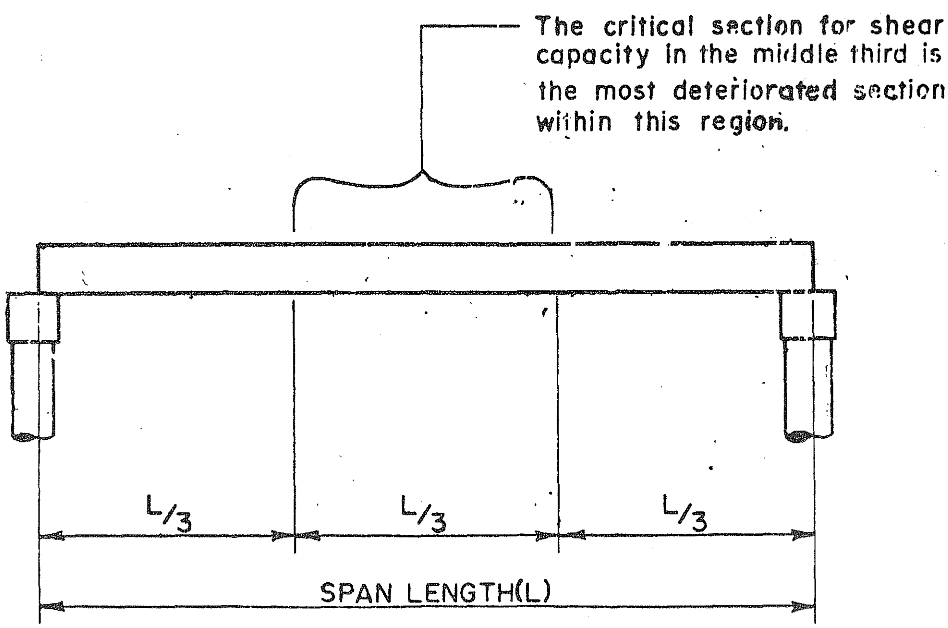


FIGURE 7-3

Section 7.3 illustrates the use of this manual to rate a Prestressed Concrete Channel and Slab System Superstructure. All forms needed to rate this type superstructure are included in Section 12.4 of Appendix C.

STEP 1 OBTAIN DATA

Figure 7-4 is a sample Bridge Inspection Report in which the need for a Structural Rating has been determined by the engineer. By examination of current legal load requirements it is determined that the legal load cases and AASHTO load cases shown on Plate III of Appendix A are applicable. From a copy of the "as-built" construction plans the following is noted:

design loading = H20
f'c (channel) = 5,000 psi
f'c (slab) = 3,400 psi
Size and location of prestressing strands (as entered on Inspection Data Sheet)
Size and location of stirrups (as entered on Inspection Data Sheet)

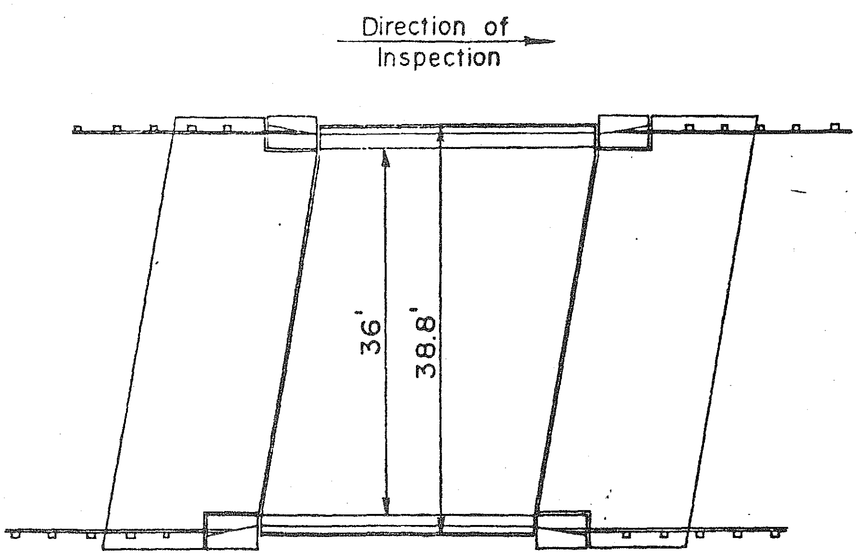
FIGURE 7-4 , EXAMPLE BRIDGE INSPECTION DATA REPORT

Bridge No. XXXXX XX County S.R. XXX

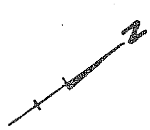
Description: Located on S.R. XXX at M.P. OO. It is a one span all concrete structure composed of prestressed concrete channels with a pour-in-place slab. The bridge was constructed about 1974 and has an estimated ADT of 200.

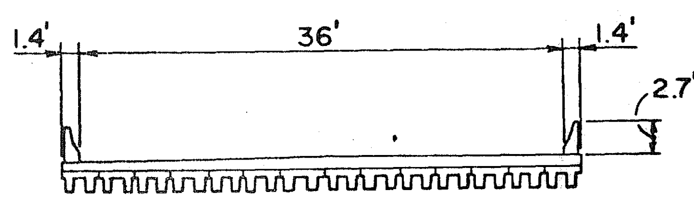
The following measurements were made during the inspection:

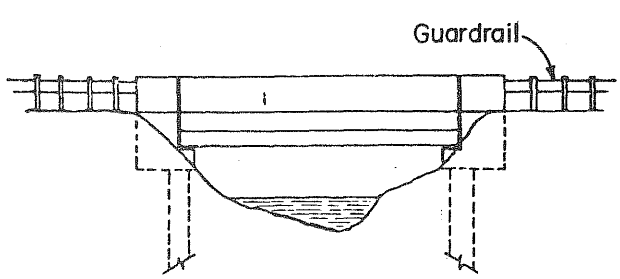
length = 30.5'
no. spans = 1
piling = 12" square
rdwy. approach = 14' ±
skew angle = 15° ±



PLAN



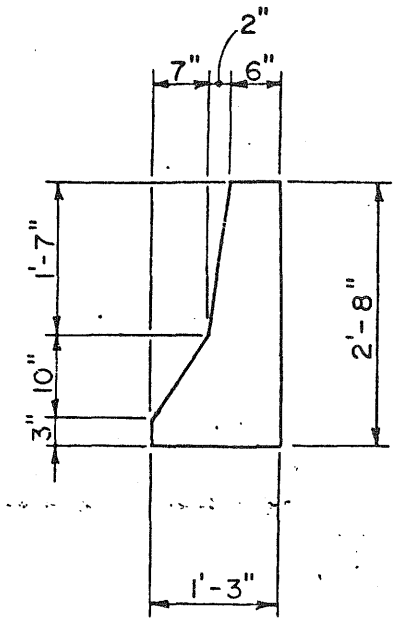




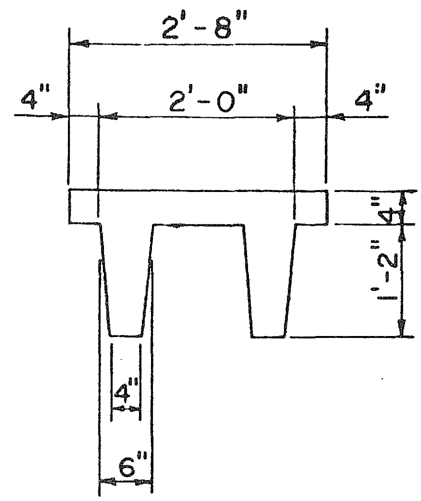
ELEVATION

TYPICAL SECTION

Barrier:



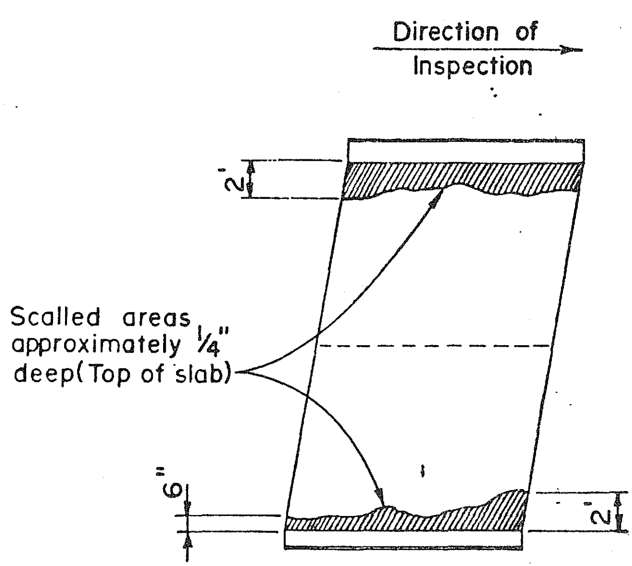
Typical Prestressed Channel:



Approaches: Tangent roadway approaching both ends of the structure with a -0.5% grade. Drainage ditches provide adequate handling of runoff.

Utilities: None.

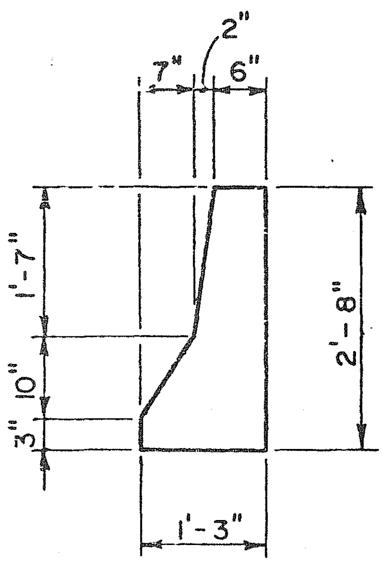
Slab:



STEP 2 DETERMINE TYPE OF SUPERSTRUCTURE

Compare the description of the superstructure given in the Bridge Inspection Data Report with the configurations detailed in Chapter 2. The Prestressed Concrete Channel and Slab System Superstructure shown in Section 2.5 is found to be applicable.

STEP 3 Obtain a photocopy of the "Prestressed Concrete Channel and Slab System Inspection Data Sheets" (Section 12.4) and record the necessary information in the spaces provided. When information requested on the Inspection Data Sheet is not available from the Bridge Inspection Data Report or "as-built" plans, a field visit will be necessary to obtain the information.



SKETCH A

SECTION 7.3.1 - PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM INSPECTION DATA SHEET

NOTE: These sheets should be completed in their entirety before continuing with the rating computations.

By: John Doe Date: xx-xx-xx

Bridge Name: State Road xxx Over x Canal

Bridge No.: xxxxx Road No.: xxx County: x

Bridge Length = 30.5 ft. Number of Spans = 1

Span length (℄ to ℄ of support = 30.5 ft.
(if different, list for each)

Overall Bridge Width = 38.8 ft.

Bridge Width (curb to curb) = 36.0 ft.

Sidewalk: Width = None ft.
Rail Size = ______ in. x ______ in.
Rail Post Size & Length = ______ in. x ______ in. x ______ ft.
Average Post Spacing = ______ ft.

Barrier Rails: Rail Size = ______ in. x ______ in.
See Sketch A Rail Post Size & Length = ______ in. x ______ in. x ______ ft.
Average Post Spacing = ______ ft.

Curb Block: Height x Width = None in. x ______ in.

Wearing Surface: asphalt; thickness = None in.
concrete; thickness = ______ in.

Cast in place concrete deck surface: thickness = 4 in.

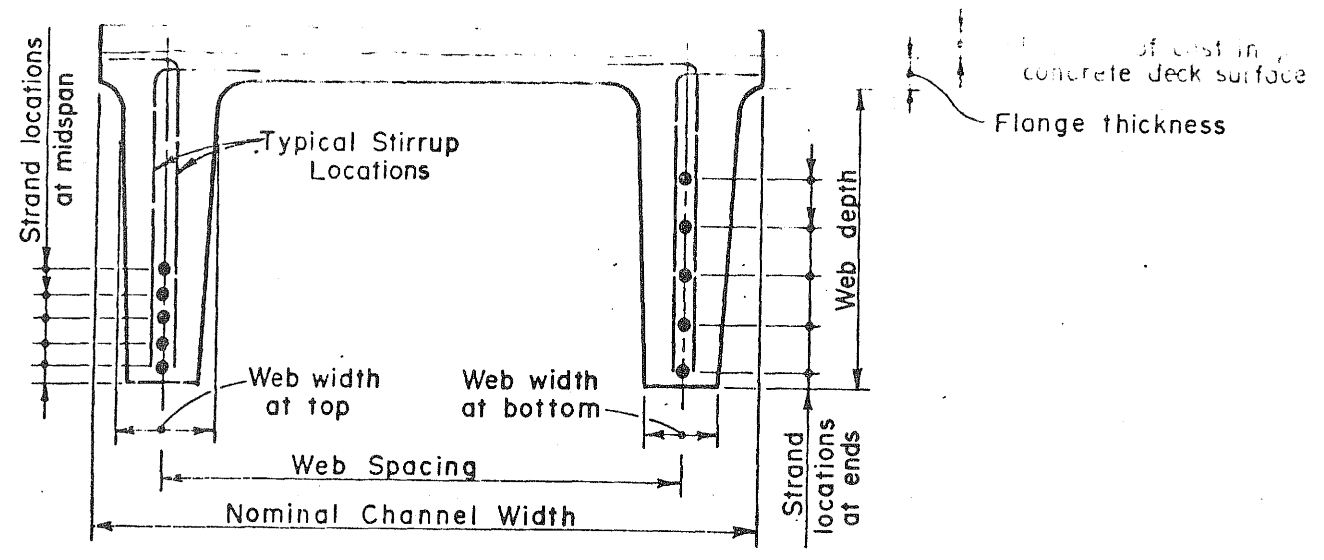
transverse reinf. = #______ at ______ in. spacing.
longitudinal reinf. = #______ at ______ in. spacing.

or, WWF = 6x6x10/10 OR (6x6-W1.4 x W1.4)

Strand locations are obtained from the "as-built" construction plans.

SECTION 7.3.2 - PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM INSPECTION DATA SHEET (Continued)

Prestressed Channel Section:



Number of Channels = 14

Thickness of cast in place concrete deck surface = 4 in.

Flange thickness = 4 in.

Web depth = 14 in.

Web width at top = 6 in.

Web width at bottom = 4 in.

Web spacing = 24 in.

Nominal channel width = 32 in.

Strand locations at ends:

Distance from bottom of web to 1st strand = 2 in.
Distance from bottom of web to 2nd strand = 4 in.
Distance from bottom of web to 3rd strand = 10 in.
Distance from bottom of web to 4th strand = — in.
Distance from bottom of web to 5th strand = — in.

Strand locations at mid span:

Distance from bottom of web to 1st strand = 2 in.
Distance from bottom of web to 2nd strand = 4 in.
Distance from bottom of web to 3rd strand = 10 in.
Distance from bottom of web to 4th strand = — in.
Distance from bottom of web to 5th strand = — in.

Data on strands, concrete strengths, and stirrups are obtained from the "as-built" construction plans.

SECTION 7.3.2 - PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM INSPECTION DATA SHEET (Continued)

Prestressing Strand Size: 1/2 in. diameter; Grade 250^k

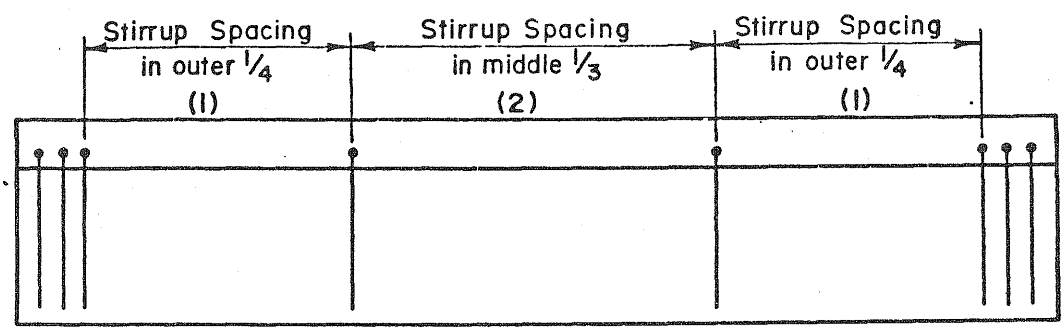
(270^k high-strength or 250^k ASTM designation)

Initial Tensioning Load 25 200 lbs. each

Concrete Strength = 5000 psi. (for prestressed channels)

Concrete Strength = 3400 psi. (for cast in place concrete deck)

Shear Reinforcing (Stirrups):



Side View of Channel

outer 1/4 (1)

Stirrup Spacing = 6 in.
Stirrup Bar Size = #3
fy (yield strength of steel) = 40 000 psi.
(From contract plans[1])

middle 1/3 (2)

Stirrup Spacing = 10 in.
Stirrup Bar Size = #3
fy (yield strength of steel) = 40 000 psi.
(from contract plans)

---

[1]When the Grade is given, or the steel is unknown, use the following yield stresses for reinforcing steel:

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

STEP 4 COMPUTE LIVE LOAD SHEARS AND MOMENTS

The load cases are found to be the same as those on Plate III in Appendix A, therefore, we may use the table on Plate I in Appendix A to obtain the maximum Live Load Moments for each loading case.

H20

Since the design loading is specified as H-15 (H loading), the HS load case will not need to be investigated.

The Live Load Shear for each load case is computed by the method given in Section 3.3. Note that the stirrup spacing in the outer quarters of the member differs from that in the middle third of the member; therefore, Live Load Shear will need to be computed at both the quarter and third points.

SU2: It is evident that by placing the SU2 load case on the span in the opposite direction the smaller load would be placed at the point where shear is being computed thus yielding a smaller Live Load Shear than the placement shown on the following page.

DATE	DESIGN	Sec. 7.3.2 Example Computations	SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Prestressed Concrete Channel and Slab System Superstructure

Span Length = 30.5'

Design Loading = H-20

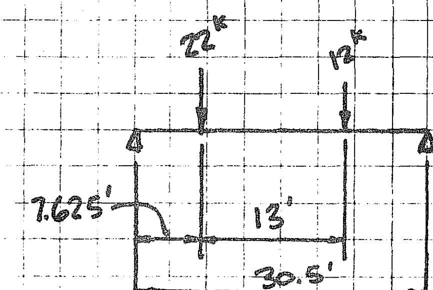
Live Load Moments - From Plate I

SU 2	=	187.1	ft-k
SU3	=	339.2	ft-k
SU4	=	367.2	ft-k
C3	=	202.7	ft-k
C4	=	291.2	ft-k
C5	=	291.2	ft-k
H	=	251.6	ft-k

Live Load Shear at Quarter Point

$X = 1/4\,(30.5) = 7.625$ ft.

Examine SU2 Load Case



$$LLV = 22\left(1-\frac{7.625}{30.5}\right) + 12\left(1-\frac{7.625+13}{30.5}\right) = \boxed{20.39^k}$$

SU3: Note that by placing the SU3 load case on the span in the opposite direction the middle 22^k load would be farther from the point where shear is being computed, thus yielding a smaller Live Load Shear than the placement shown on the following page.

SU4: Note that by placing the SU4 load case on the span in the opposite direction the middle two loads would be farther from the point where shear is being computed and the 13.9 kip load would be at the point where shear is being checked instead of the 18.7 kip load. This would yield a smaller Live Load Shear than the placement shown on the following page.

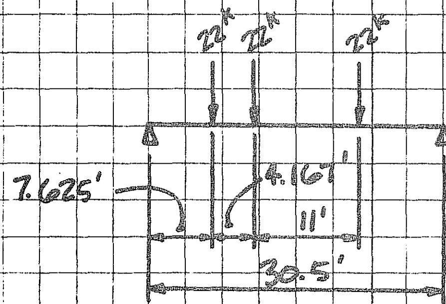
C3: The C3 load case is 30 ft. long; and since we are computing the Live Load Shear 7.625 ft. from the support on a 30.5 ft. span, it is evident that all 3 axles of the load case, cannot be on the span at one time. For the most critical case we want to choose the two axles closest together with the largest load placed at the point where load shear is being checked as shown on the following page.

C4: By examining the C4 load case it is found that it is impossible to place more than two axles on a 30.5 ft. span for computing live load shear 7.625 ft. from the support. Therefore, the two axles of the C4 load case that give the maximum Live Load Shear are the two axles closest together with the greatest loads as shown on the following page.

DATE	DESIGN	Sec. 7.3.2- Example Computations Cont.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

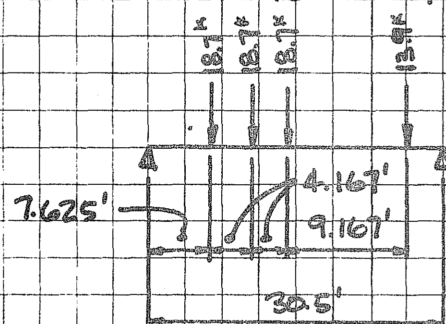
Examine SU3 Load Case



$$LLV = 22\left(1-\frac{7.625}{30.5}\right) + 22\left(1-\frac{7.625+4.167}{30.5}\right) + 22\left(1-\frac{7.625+4.167+11}{30.5}\right)$$

$$LLV = \boxed{35.55^K}$$

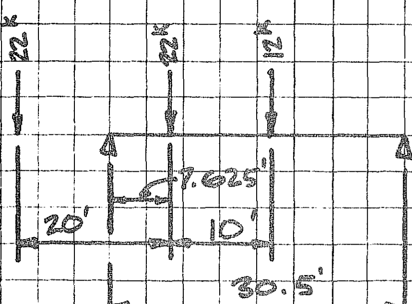
Examine SU4 Load Case



$$LLV = 18.7\left(1-\frac{7.625}{30.5}\right) + 18.7\left(1-\frac{7.625+4.167}{30.5}\right) + 18.7\left(1-\frac{7.625+4.167+4.167}{30.5}\right) + 13.9\left(1-\frac{7.625+4.167+4.167+9.167}{30.5}\right)$$

$$LLV = \boxed{36.86^K}$$

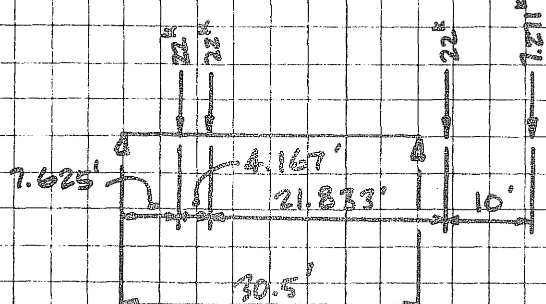
Examine C3 Load Case



$$LLV = 22\left(1-\frac{7.625}{30.5}\right) + 12\left(1-\frac{7.625+10}{30.5}\right)$$

$$LLV = \boxed{21.57^K}$$

Examine C4 Load Case



$$LLV = 22\left(1-\frac{7.625}{30.5}\right) + 22\left(1-\frac{7.625+4.167}{30.5}\right)$$

$$\boxed{LLV = 29.99^K}$$

C5: Referring to Plate III in Appendix A, it is noted that either a two-axle placement or a three-axle placement is possible on the 30.5' span for computing Live Load Shear 7.625 ft. from the support. Generally the three-axle placement would be the obvious choice for maximum Live Load Shear, but in this case the three axle placement has smaller loads than the two-axle placement and it is not obvious which case controls. Therefore, it is necessary to check both cases and choose the greater value of Live Load Shear. The two-axle case was computed for the C4 load case and yielded a live load shear of 29.99^k. The three-axle placement is shown on the following page and yields 30.12^k; therefore, the three axle placement controls.

H: It is evident that by placing the H load case on the span in the opposite direction, the smaller load would be placed at the point where shear is being computed thus yielding a smaller Live Load Shear than the placement shown on the following page.

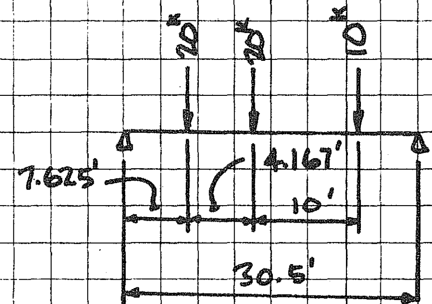
Note that design loading was specified as H-20 in the "as-built" construction plans; therefore the HS load case does not need to be evaluated.

It is good practice to summarize the maximum Live Load Shear for each loading case so they may be easily referred to later.

DATE	DESIGN	Sec. 7.3.2- Example Computations Cont.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

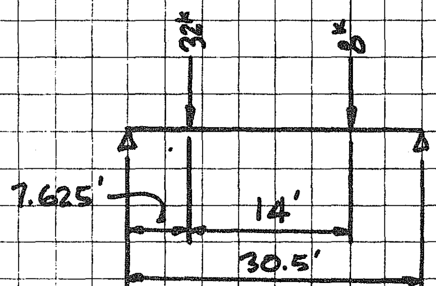
Examine C5 Load Case



$$LLV = 20\left(1-\frac{7.625}{30.5}\right) + 20\left(1-\frac{7.625+4.167}{30.5}\right) + 10\left(1-\frac{7.625+4.167+10}{30.5}\right) = 30.12^k$$

$$LLV = \boxed{30.12^k}$$

Examine H Load Case



$$LLV = 32\left(1-\frac{7.625}{30.5}\right) + 8\left(1-\frac{7.625+14}{30.5}\right)$$

$$LLV = \boxed{26.33^k}$$

Summary Of Maximum Live Load Shears At Quarter Point

SU2 = 20.39^k

SU3 = 35.55^k

SU4 = 36.86^k

C3 = 21.57^k

C4 = 29.99^k

C5 = 30.12^k

H = 26.33^k

Next the Live Load Shear needs to be computed in the middle third of the member. The procedure is the same as that followed to compute the shear at the quarter points except a new "x" is used.

SU2: It is evident that by placing the SU2 load case on the span in the opposite direction the smaller load would be placed at the point where shear is being computed, thus yielding a smaller Live Load Shear than the placement shown on the following page.

SU3: Note that by placing the SU3 load case on the span in the opposite direction the middle 22^k load would be farther from the point where shear is being computed, thus yielding a smaller Live Load Shear than the placement shown on the following page.

SU4: Note that by placing the SU4 load case on the span in the opposite direction the middle two loads would be farther from the point where shear is being computed and the 13.9 kip load would be at the point where the shear is being checked instead of the 18.7 kip load. This would yield a smaller Live Load Shear than the placement shown on the following page.

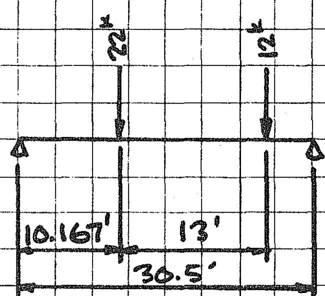
DATE	DESIGN	SEC. 7.3.2 - EXAMPLE COMPUTATIONS CONT.	SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Live Load Shear at Third Point

$X = \frac{1}{3}(30.5) = 10.167'$

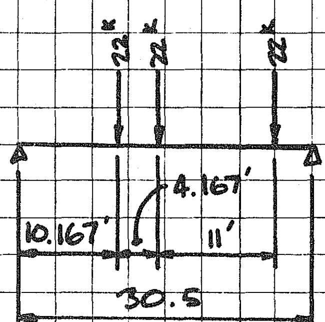
Examine SU2 Load Case



$$LLV = 22\left(1 - \frac{10.167}{30.5}\right) + 12\left(1 - \frac{10.167 + 13}{30.5}\right)$$

$$LLV = \boxed{17.55^k}$$

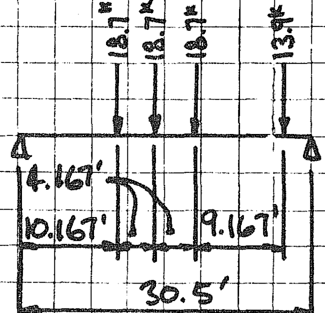
Examine SU3 Load Case



$$LLV = 22\left(1 - \frac{10.167}{30.5}\right) + 22\left(1 - \frac{10.167 + 4.167}{30.5}\right)$$
$$+ 22\left(1 - \frac{10.167 + 4.167 + 11}{30.5}\right)$$

$$LLV = \boxed{30.05^k}$$

Examine SU4 Load Case



$$LLV = 18.7\left(1 - \frac{10.167}{30.5}\right) + 18.7\left(1 - \frac{10.167 + 4.167}{30.5}\right)$$
$$+ 18.7\left(1 - \frac{10.167 + 4.167 + 4.167}{30.5}\right)$$
$$+ 13.9\left(1 - \frac{10.167 + 4.167 + 4.167 + 9.167}{30.5}\right)$$

$$LLV = \boxed{31.03^k}$$

Note that design loading was specified as H-20 in the "as-built" construction plans, therefore, the HS load case does not need to be evaluated.

And finally, summarize the maximum Live Load Shear for each loading case so they may be easily referred to later.

DATE	DESIGN	SEC. 7.3.2- EXAMPLE COMPUTATIONS CONT.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Summary Of Maximum Live Load Shears At Third Point

SU2 = 17.55^k

SU3 = 30.05^k

SU4 = 31.03^k

C3 = 18.73^k

C4 = 26.33^k

C5 = 25.96^k

H = 22.99^k

STEP 5 PERFORM RATING CALCULATIONS

The rating calculations are made in accordance with the procedures given in Section 7.2.

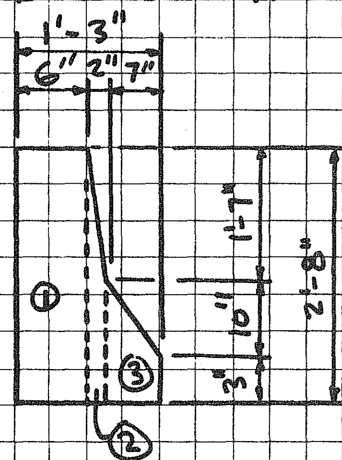
1. Calculate dead load per linear foot of channel. Obtain a photocopy of the forms headed "Section 12.4 - Calculation of Dead Load per Linear Foot of Channel" and compute the total Dead Load by completing the calculations detailed thereon.

   Since the barrier for the bridge being rated is not of the configuration shown on the forms, the first step is to compute the barrier area.

DATE	DESIGN	SEC. 7.3.3-EXAMPLE COMPUTATIONS CONT.	SHEET OF
	CHECK	JOB / FOR	JOB NO.

SUBJECT

CALCULATE BARRIER AREA



Area ① $(0.5\ ft)(2.667\ ft) = 1.334\ ft^2$

Area ② $(\frac{1}{2})(2.667' + 1.083')(0.167') = 0.313\ ft^2$

Area ③ $(\frac{1}{2})(1.083' + 0.250')(0.583') = \underline{0.389}\ ft^2$

Total $= 2.036\ ft^2$

The barrier area as computed on the previous page is entered in the appropriate space.

The intent of the computations shown on the following page is to calculate the weight per linear foot of barrier and then divide it equally between the number of prestressed channels. The rail configuration illustrated here is a typical concrete curb, rail, & railpost type barrier, but actual configurations of existing barriers may vary somewhat.

For barriers constructed of concrete that differ in configuration from the barrier indicated above, the average barrier area should be calculated and a unit weight of 150 lbs/ft^3. used to get a per foot weight of each barrier. The total per foot weight of all barriers should then be equally distributed to the prestressed channel sections. For barriers employing steel, the area of steel per linear foot should be computed and a unit weight of 490 lbs/ft^3. used. If standard rolled steel sections are used, the weights per linear foot may be obtained from the AISC Manual.* For aluminum, a weight of 175 lbs/ft^3. is to be used and for timber a unit weight of 50 lbs/ft^3. is to be used.

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* American Institute of Steel Construction, Inc. Manual of Steel Construction, Part I.

SECTION 7.3.3 - CALCULATION OF DEAD LOAD PER LINEAR FOOT OF CHANNEL

Calculate Barrier Area:

Barrier: (Rail Width)(Rail Height)
(— in.)(— in.)(1/144) = — ft.^{2}
(Curb Width)(Curb Height)
(— in.)(— in.)(1/144) = — ft.^{2}
(Rail Post Length)(Height)(Width)(1/Ave. Post Spacing)
(— ft.)(— in.)(in.)(1/ ft)(1/144)
= — ft.^{2}

Barrier Area = $\boxed{2.036}$ ft.^{2}
FROM PAGE 7-32

Barrier Dead Load per Foot:

= (Barrier Area)(150 lb/ft.^{3})(Number of Barriers)(1/Number of Prestressed Channels)
= (2.036 ft.^{2})(150)(2)(1/ 14) = $\boxed{43.63}$ lb/ft.

Asphalt Wearing Surface Dead Load per foot:

= (Roadway Width)(144 lb/ft^3)(Thickness)(1/Number of Prestressed Channels)

= (— ft.)(144)(— in.)(1/12)(1/ —) = $\boxed{—}$ lb/ft.

Cast In Place Concrete Deck Surface Dead Load Per Foot:

= (Thickness of Cast-In-Place Concrete Deck Surface)(Nominal Channel Width)(1/144)(150 lb/ft^3)

= (4 in.)(32 in.)(1/144)(150) = $\boxed{133.33}$ lb/ft.

Channel Dead Load Per Foot:

Channel Area = (Flange Thickness)(Nominal Channel Width) + [(Web Width at Top) + (Web Width at Bottom)](Web Depth)

= (4 in.)(32 in.) + [(6 in.) + (4 in.)](14 in.)

= $\boxed{268}$ in^2

For flexure the loadings must be divided into 2 catagories:

1. Those acting on the prestressed channel alone, and

2. Those acting on the composite section of the prestressed channel and the cast in place slab.

Loadings corresponding to the first group include those due to prestressing and, the dead load of the channel and the cast in place slab.

W_{DL1} = (Channel DL) + (Cast in Place Slab DL)

Loadings corresponding to the second group include the dead load of the barriers and the wearing surface, and the live load plus impact.

W_{DL2} = (Barrier DL) + (Asphalt Wearing Surface DL)

SECTION 7.3.3 - CALCULATION OF DEAD LOAD PER LINEAR FOOT OF CHANNEL (CONT.)

Channel Dead Load per foot = (Channel Area)(1/144)(150 lb/ft.^{3})

= (268 in.^{2})(1/144)(150) = $\boxed{279.17}$ lb/ft.

Dead Load on Channel Only Per Linear Foot ($W_{DL\ 1}$)

W_{DL1} = (Channel DL) + (Cast in place slab DL)

W_{DL1} = (279.17 lb/ft) + (133.33 lb/ft) = 412.50 lb/ft

Dead Load on Composite Section Per Linear Foot (W_{DL2})

W_{DL2} = (Barrier DL) + (Asphalt Wearing Surface DL)

W_{DL2} = (43.63 lb/ft) + (— lb/ft) = 43.63 lb/ft

Total Dead Load per linear foot of channel (W_{DL})

Barrier DL + Asphalt Wearing Surface DL + Channel DL + Cast In Place Concrete Deck Surface DL

W_{DL} = (43.63 lb/ft.)+(— lb/ft.)+(279.17 lb/ft.)+(133.33 lb/ft)

= $\boxed{456.13}$ lb/ft.

STEP 5 2. DETERMINE INVENTORY RATING FOR MOMENT CAPACITY.

A photocopy of the forms "Section 12.4 - Moment Capacity Rating at Inventory Level" should be obtained and the Inventory Rating computed by performing the computations detailed thereon.

First, the Dead Load Moment on the channel (M_{DL1}) is computed and then entered in the appropriate space in the table on sheet 7-48.

For illustration of areas 1, 2, 3 and 4 refer to the sketch on the following page. The dimension for the thickness of cast in place concrete deck surface (T_d) has been reduced 1/4" to account for scaling. Therefore, use thickness of cast in place concrete deck surface of 3.75 in. in the computation of T_d.

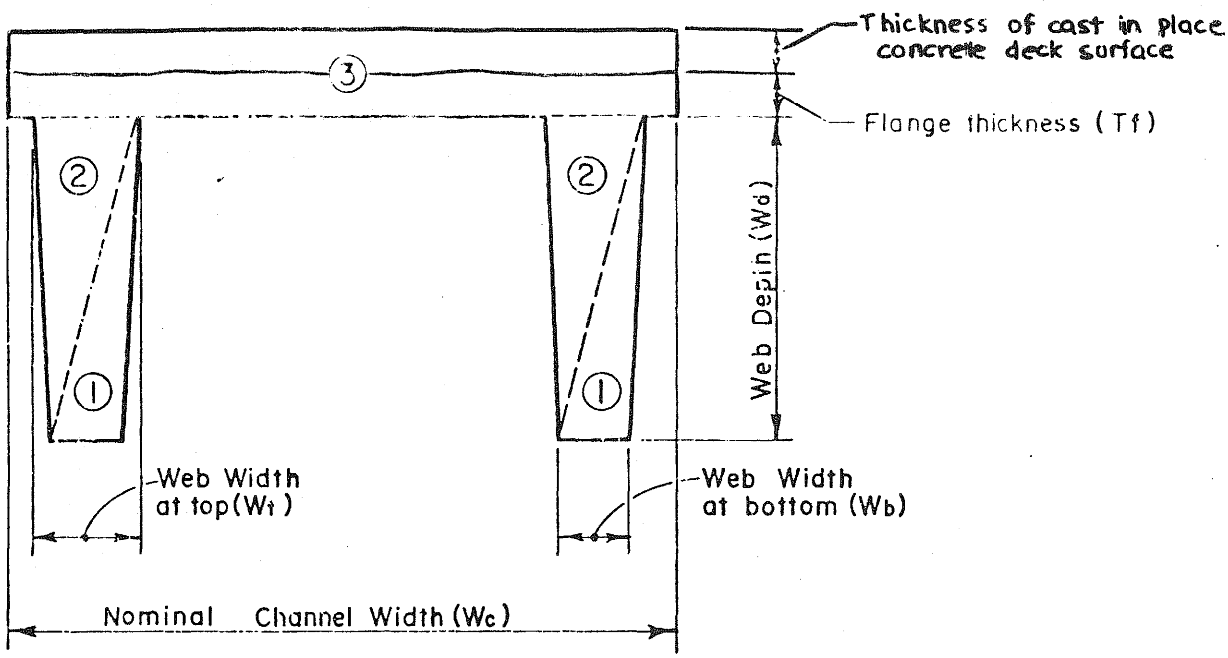


FIGURE 7-4

Generalized Channel Section

SECTION 7.3.4 - MOMENT CAPACITY RATING AT INVENTORY LEVEL (Considering Servicibility Requirements)

Loads and Stresses on Channel Only:

Calculate Applied Dead Load Moment(M_{DL1}):

M_{DL1} = (1/8)(W_{DL1})(Span Length)^2(12)
M_{DL1} = (1/8)(412.50 lb/ft.)(30.5 ft.)^2(12) = $\boxed{575,592}$ in-lbs.

Enter M_{DL1} in the appropriate space in the table on sheet 7-48

Compute Section Properties for Channel Only:

Area of 1 (A_1) = (W_b)(W_d) = (4 in.) (14 in.) = 56.0 in
Area of 2 (A_2) = (W_t)(W_d) = (6 in.) (14 in.) = 84.0 in
Area of 3 (A_3) = (W_c)(T_f) = (32 in.) (4 in.) = 128.0 in

Channel area = $\boxed{268.0}$ in.^{2}

Moment of Inertia*(I_1) of 1 = $\frac{(W_b)(W_d)^3}{36}(2)$

$= \frac{(4 \text{ in.})(14 \text{ in.})^3}{36}(2) = 609.78$ in.^{4}

Moment of Inertia (I_2) of 2 = $\frac{(W_t)(W_d)^3}{36}(2)$

$= \frac{(6 \text{ in.})(14 \text{ in.})^3}{36}(2) = 914.67$ in.^{4}

Moment of Inertia (I_3) of 3 = $\frac{(W_c)(T_f)^3}{12}$

$= \frac{(32 \text{ in.})(4 \text{ in.})^3}{12} = 170.67$ in.^{4}

---

* For those interested in additional detail relating to calculation of moments of inertia, reference is made to Elements of Strength of Materials, p. 346.

Y_b = distance from the bottom of the channel to the neutral axis of 1, 2, or 3.

Y_{na} = distance from the bottom of the channel to the neutral axis of the channel.

SECTION PROPERTIES OF CHANNEL ONLY (CONT.):

$Y_{b1} = 1/3(W_d) = 1/3$ (14 in.) = 4.67 in.

$Y_{b2} = 2/3(W_d) = 2/3$ (14 in.) = 9.33 in.

$Y_{b3} = W_d + 1/2\ (T_f)$ = (14 in.) + (1/2)(4 in.) = 16.00 in.

$$Y_{na} = \frac{(A_1)(Y_{b1}) + (A_2)(Y_{b2}) + (A_3)(Y_{b3})}{A_1 + A_2 + A_3}$$

$$Y_{na} = \frac{(56\text{ in.}^2)(4.67\text{ in.})+(84\text{ in.}^2)(9.33\text{ in.})+(128.0\text{ in.}^2)(16.0\text{ in.})}{(56\text{ in.}) + (84\text{ in.}) + (128.0\text{ in.})}$$

$$= \boxed{11.54}\ \text{in.}$$

Now, the moment of inertia of the prestressed concrete channel section (I_{na}) equals:

$$I_1 + I_2 + I_3 + A_1(Y_{na} - Y_{b1})^2 + A_2(Y_{na} - Y_{b2})^2 + A_3(Y_{b3} - Y_{na})^2$$

$$I_{na} = (609.78\text{ in.}^4)+(914.67\text{ in.}^4)+(170.67\text{ in.}^4)+(56\text{ in.}^2)(11.54\text{ in.} - 4.67\text{ in.})^2$$

$$+ (84\text{ in.})^2\ (11.54\text{ in.} - 9.33\text{ in.})^2 + (128.0\text{ in.}^2)\ (16\text{ in.} - 11.54\text{ in})^2$$

$$= \boxed{7295}\ \text{in.}^4$$

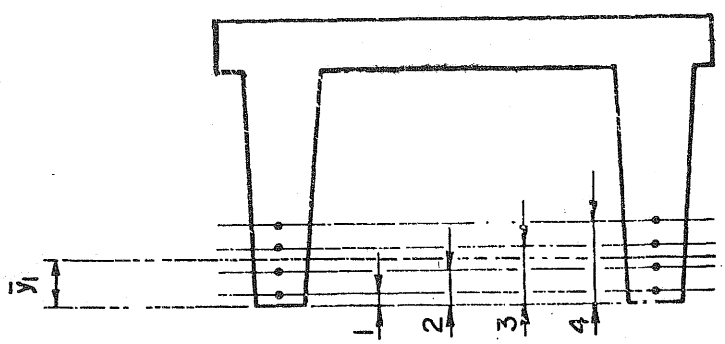
Section Modulus at the top of the channel (S_T):

$$S_T = \frac{I_{na}}{(W_d + T_f - Y)_{na}}$$

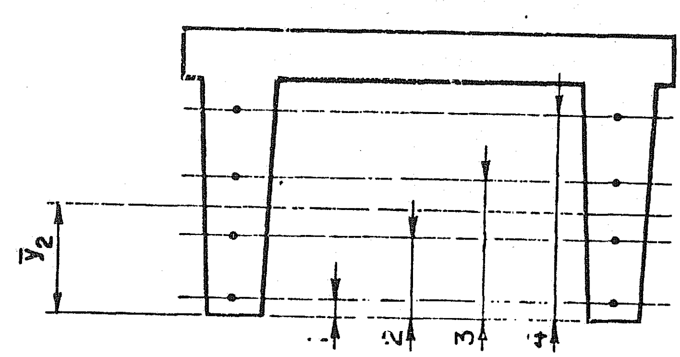
$$S_T = \frac{(7295\text{ in.}^4)}{(14\text{ in.}) + (4\text{ in.}) - (11.54\text{ in.})} = \boxed{1129}\ \text{in.}^3$$

Dimensions 1 thru 4 are the distances from the bottom of the channel to each row of prestressing strands and are illustrated on the following page.

STRAND PATTERN - MIDSPAN



STRAND PATTERN - ENDS



GENERAL SECTIONS SHOWING LOCATION OF PRESTRESSING STRANDS

$\bar{y}$ - Location of centroid of strands from bottom of channel.

SECTION PROPERTIES OF CHANNEL ONLY (Cont.):

Section modulus at the bottom of the channel (S_B)

$$S_B = \frac{I_{na}}{Y_{na}}$$

$$S_B = \frac{(\ 7295\ \text{in.}^4)}{(\ 11.54\ \text{in.})} = \boxed{632}\ \text{in.}^3$$

S_B and S_T should be entered in the appropriate spaces in the table on sheet 7-48.

Calculate Initial Prestressing Load (P_i):

P_i = (number of strands)(tensioning load)

P_i = (6)(25200 lbs.) = 151200 lbs.

Calculate Initial Prestress (f_i):

$$f_i = \frac{-P_i}{\text{Channel Area}} = \frac{(-)\ 151200\ \text{lbs.}}{268\ \text{in.}^2} = \boxed{-564}\ \text{psi*}$$

This value, including the minus sign, should be entered in the appropriate spaces in the table on sheet 7-48.

Determine Prestress Moment (P_{ie}) at Midspan:

$$\bar{y}_1 = \frac{2(\text{dim.}1 + \text{dim.}2 + \text{dim.}3 + \text{dim.}4)}{(\text{total number of strands})}$$

$$\bar{y}_1 = \frac{2(\ 2\ \text{in.} + 4\ \text{in.} + 10\ \text{in.} + 0\ \text{in.})}{(\ 6\ )}$$

$\bar{y}_1$ = 5.333 in.

NOTE: 1. Dimensions 1,2,3 and 4 are illustrated on the opposite page.
2. If there are less than 4 rows of strands, enter zero for the dimension of unused rows.

* The sign convention used in this manual is (-) for compression and (+) for tension.

Y_{NA} is the distance from the bottom of the channel to the neutral axis of the channel from the section properties calculations.

$P_i e_1$ and $P_i e_2$, including their signs, should be entered in the appropriate spaces in the table on sheet 7-48.

This completes the input for the Summary of Stresses table on page 7-48. The table should be completed in accordance with the equations shown in the legend and the columns summed to obtain the totals.

STRESSES ON CHANNEL ONLY:

$e_1 = Y_{na} - \bar{y}_1$

$e_1 = (\ 11.54 \text{ in.}) - (\ 5.33 \text{ in.}) = \underline{\ 6.21\ }$ in.

$P_i e_1 = (Pi)(e_1)$

$= (\ 151{,}200 \text{ lbs.})(\ 6.21 \text{ in.})$

$= \boxed{938{,}952}$ in-lbs.

Determine Prestress Moment ($P_i e_2$) at Ends:

$$\bar{y}_2 = \frac{2(\text{dim.1} + \text{dim.2} + \text{dim.3} + \text{dim.4})}{(\text{Total Number of Strands})}$$

$$\bar{y}_2 = \frac{2(\ 2 \text{ in.} + \ 4 \text{ in.} + 10 \text{ in.} + 0 \text{ in.})}{(\ 6\ )}$$

$\bar{y}_2 = \underline{\ 5.333\ }$ in.

$e_2 = Y_{na} - \bar{y}_2$

$e_2 = (\ 11.54 \text{ in.}) - (\ 5.333 \text{ in.}) = \underline{\ 6.21\ }$ in.

$P_i e_2 = (Pi)(e_2)$

$= (\ 151{,}200 \text{ lbs.})(\ 6.21 \text{ in.})$

$= \boxed{938{,}952}$ in-lbs.

Determine Fractional Loss of Prestress (a):

A^*_s = (Number of Strands)(Area of Strands)*

$= (\ 6\ )(.1438 \text{ in}^2) = \underline{\ .8628\ }$ in^2

$$a = \frac{45{,}000\ A^*_s}{Pi}$$

$$a = \frac{45{,}000\ (.8628 \text{ in.}^2)}{(\ 151{,}200 \text{ lbs.})} = \underline{\ 0.26\ }$$

*Strand sizes and properties are tabulated for common strand sizes on Plate IV in Appendix A.

At this point the data to complete the summary of stresses table should have been entered into the table. This consists of:

- F - the prestress force
- S_T - section modulus at top of channel
- S_B - section modulus at bottom of channel
- Pie mid - prestress moment at midspan
- Pie end - prestress moment at ends

The next step is to complete the equations shown in the upper table and to enter the results in the corresponding spaces in the lower table. This will complete the table except for σ_1, σ_2, σ_3 and σ_4. These are computed by totaling each column.

It is emphasized that all (+) and (-) signs in the equations in the upper table should also be entered in the lower table and used when totaling each column.

SECTION 7.3.4- MOMENT CAPACITY RATING AT INVENTORY LEVEL (CONT.)

LEGEND FOR SUMMARY OF STRESSES						
Item	Section Modulus (in^3)	Moment (in-lbs.)	At Midspan		At Ends	
			Top	Bottom	Top	Bottom
Prestress Force			f_i	f_i	f_i	f_i
Prestress Moment at Midspan	S_T S_B	P_ie_1	$\frac{P_ie_1}{S_T}$	$\frac{-(P_ie_1)}{S_B}$		
Prestress Moment at Ends	S_T S_B	P_ie_2			$\frac{P_ie_2}{S_T}$	$\frac{-(P_ie_2)}{S_B}$
Loss of Prestress Force			$-(a)\left(\frac{P_ie_1}{S_T}+f_i\right)$	$-(a)\left(\frac{-P_ie_1}{S_B}+f_i\right)$	$-(a)\left(\frac{P_ie_2}{S_T}+f_i\right)$	$(a)\left(\frac{-P_ie_2}{S_B}+f_i\right)$
Dead Load	S_T S_B	M_{DL}	$\frac{-M_{DL}}{S_T}$	$\frac{M_{DL}}{S_B}$		
TOTALS			σ_1	σ_2	σ_3	σ_4

SUMMARY OF STRESSES (in psi.)						
Item	Section Modulus (in^3)	Moment (in-lbs.)	At Midspan		At Ends	
			Top	Bottom	Top	Bottom
Prestress Force			−564	−564	−564	−564
Prestress Moment at Midspan	1129 632	938,952	+832	−1485		
Prestress Moment at Ends	1129 632	938,952			+832	−1485
Loss of Prestress Force			−70	+533	−70	+533
Dead Load	1129 632	575,592	−510	+911		
TOTALS *			−312	−605	+198	−1516

* When totaling the columns, pay strict attention to (+) and (−) signs.

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COMPUTE SECTION PROPERTIES OF THE COMPOSITE SECTION:

Reduce the thickness of the cast in place slab by 1/4" to account for scaling . . . .

$$T_d = \left[\begin{array}{c}\text{Adjusted}\\ \text{Thickness of Cast}\\ \text{In Place Slab}\end{array}\right] \quad \frac{\sqrt{\text{Concrete Strength of Slab}}}{\sqrt{\text{Concrete Strength of Channel}}}$$

$$T_d = \left[(4.0 \text{ in.} - .25 \text{ in})\right] \left[\frac{\sqrt{3400} \text{ psi}}{\sqrt{5000} \text{ psi}}\right] = \underline{3.09} \text{ in.}$$

$$\text{Area of 4 (A4)} = (W_c)\ (T_d) = (32 \text{ in})\ (3.09 \text{in}) = \underline{99} \text{ in.}^2$$

$$\text{Moment of Inertia of 4 } (I_4) = \frac{(W_c)(T_d)^3}{12} = \frac{(32 \text{ in})(3.09 \text{ in})^3}{(12)} = \underline{78.68} \text{ in}^4$$

$$Y_{b4} = (W_d) + (T_f) + 1/2\ (T_d) = (14 \text{ in}) + (4 \text{ in}) + 1/2\ (3.09 \text{ in}) = \underline{19.55} \text{ in}$$

$$Y_{na} \text{ comp} = \frac{(\text{Area of Channel})\ (Y_{na} \text{ Channel}) + (A_4)\ (Y_{b4})}{(\text{Area of Channel}) + (A_4)}$$

$$Y_{na} \text{ comp} = \frac{(268.0 \text{ in}^2)\ (11.54 \text{in}) + (99 \text{ in}^2)(19.55 \text{ in})}{(268 \text{ in}^2) + (99 \text{ in}^2)} = \underline{13.70} \text{ in.}$$

The moment of Inertia of the composite section equals:

$$I_{na} \text{ comp} = I_{na} + I_4 + (\text{Area of Channel})(Y_{na} \text{ comp} - Y_{na})^2 + A_4(Y_{b4} - Y_{na} \text{ comp})^2$$

$$I_{na} \text{ comp.}\ (7295 \text{ in}^4) + (78.68 \text{in}^4) + (268 \text{ in}^2)(13.7 \text{ in} - 11.54 \text{in})^2 + (99 \text{ in}^2)\ (19.55 \text{ in} - 13.70 \text{ in})^2$$

$$I_{na} \text{ comp} = \underline{12012} \text{ in}^4$$

Section Modulus at the top of the channel

$$S_T \text{ comp} = I_{na} \text{ comp} / (W_d + T_f - Y_{na} \text{ comp})$$

$$S_T \text{ comp} = (12012 \text{in}^4)/(14 \text{ in} + 4 \text{ in} - 13.70 \text{ in}) = \underline{2793} \text{ in}^3$$

Section Modulus at the bottom of the channel

$$S_B \text{ comp} = I_{na} \text{ comp} / Y_{na} \text{ comp}$$

$$S_B \text{ comp} = (12012 \text{ in}^4)\ /\ (13.70 \text{ in}) = \underline{877} \text{ in}^3$$

LOADS AND STRESSES ON COMPOSITE SECTION:

Calculate Applied Dead Load Moment (M_{DL2}):

M_{DL2} = (1/8) (W_{DL2}) (Span Length)^2 (12)

M_{DL2} = (1/8) (43.63 lb/ft) (30.5 ft)^2 (12) = 60,880 in-lbs

At mid-span the corresponding stresses are:

Top of Channel, σ_1 comp = $-\dfrac{M_{DL2}}{S_T \text{ comp}} = \dfrac{(60{,}880 \text{ in-lbs})}{(2793 \text{ in}^3)}$ = -22 psi

Bottom of Channel, σ_2 comp = $\dfrac{M_{DL2}}{S_B \text{ comp}} = \dfrac{(60{,}880 \text{ in-lbs})}{(877 \text{ in}^3)}$ = +69 psi

Now combining these with the stresses already in the channels (Page 7-48) yields . . . .

At midspan:
Top of Channel, σ_1 Total = σ_1 + σ_1 comp

= (-312 psi) + (-22 psi) = -334 psi

Bottom of Channel, σ_2 Total = σ_2 + σ_2 comp

= (-605 psi) + (+69 psi) = -536 psi

At ends:
Top of Channel, Total = σ_3 (page 7-48) = +198 psi

Bottom of Channel, Total = σ_4 (page 7-48) = -1516 psi

The structure under evaluation is over a fresh water stream, therefore use non-coastal environment for computing σ_t.

Note that σ_3 and σ_4 do not exceed the allowables, therefore the stresses at the ends of the member do not exceed the allowable stresses.

Determine Allowable Maximum Unit Stresses:

Allowable in compression (σ_c):

σ_c = 0.40 (f'c) = 0.40(5000 psi) = $\boxed{-2000}$ psi.

Allowable in tension (σ_t)

for a coastal environment (over salt water)

$\sigma_t = 3\sqrt{f'c} = 3\sqrt{\text{—}\quad \text{psi}} = \boxed{+\ \text{—}}$ psi

or, for a non-coastal environment

$\sigma_t = 6\sqrt{f'c} = 6\sqrt{5000\quad \text{psi}} = \boxed{+424}$ psi

Determine Available Moment Capacity for Live Load(M_ai):

Available Moment Capacity for the two conditions shown below should be computed. The controlling condition for available Live Load Moment (M_{ai}) is the lower value of equations 1 or 2. The stresses σ_3 and σ_4 represent the stresses in the ends of the member due to prestressing forces. Stress σ_3 should not be greater than σ_t and $_4$ should not be greater (in absolute value) than σ_c. If either σ_3 or σ_4 exceed the allowable stress, a special note should be placed on the Summary of Ratings Table as follows:

"The stresses introduced at the ends of the members due to prestressing forces exceed the allowable stresses."

σ_1 Total, σ_2 Total, σ_3 and σ_4 are the summary of stresses due to prestressing and all dead loads as calculated on previous pages.

Include (-) and (+) signs in all calculations.

1. [σ_1 Total - σ_c] (S_T comp) (1/12000)

= [(-334 psi) - (-2000 psi)](2793 in^3)(1/12000)

= $\boxed{+387.76}$ ft-kips

2. [σ_t - σ_2 Total] (S_B comp) (1/12000)

= [(424 psi.) - (-536 psi)](877 in.^{3})(1/12,000)

= $\boxed{+70.16}$ ft.-kips

And, controlling condition (M_a i) = $\boxed{+70.16}$ ft-kips.

Determine Impact Factor (I):

$$I = 1 + \left[\frac{50}{(\text{Span Length} + 125')}\right], \text{ but not greater than } 1.30$$

$$I = 1 + \left[\frac{50}{(30.5 \text{ ft.}) + 125)}\right] = \underline{1.32}$$

use I = $\boxed{1.30}$

Example computation of number of traffic lanes:

For a 17 ft. roadway: number of design traffic lanes = 17/12 = 1.42, therefore, use one design traffic lane.

For a 34 ft. roadway: number of design traffic lanes = 2.83, therefore, use two design traffic lanes.

Determine Distribution Factor (DF):

$$\text{Number of Design Traffic Lanes } (N_L) = \frac{\text{(Roadway Width)}}{12}$$

rounded off to the next lower number of lanes except for roadway widths of 18 to 24 feet use two design traffic lanes.

$$\frac{(\ 36 \text{ ft})}{12} = \underline{\quad 3 \quad}. \quad \text{Use } N_L = \boxed{3} \text{ design lanes}$$

$$N_C = \text{number of channels in span} = \boxed{14}$$

$$c = 2.2\left[\frac{\text{Overall Bridge Width)}}{\text{Design Span Length}}\right]$$

$$= 2.2\ \frac{38.8 \text{ ft.}}{30.5 \text{ ft.}} = \boxed{2.80}$$

If c is less than or equal to 3.0:

$$DF = \frac{12(N_L) + 9}{N_c[5 + \frac{N_L}{10} + (3 - \frac{2N_L}{7})(1 - \frac{C}{3})^2]}$$

$$DF = \frac{12(\ 3\ ) + 9}{(14)[5 + \frac{(3)}{10} + (3 - \frac{2(\ 3\ )}{7})(1 - \frac{(2.8)}{3})^2\ ]}$$

$$DF = \boxed{0.605}$$

If c is greater than 3.0:

$$DF = \frac{12(N_L) + 9}{N_c(5 + \frac{N_L}{10})} = \frac{12(\ —\ ) + 9}{(\ —\ )(5 + \frac{(\ —\ )}{10})}$$

$$DF = \boxed{—}$$

M_ai = Available Live Load Moment Capacity at Inventory level (ft. kips)
LLM = Applied Live Load Moment (from Section 3.2 for each loading type)(ft. kips)
GVW = Gross Vehicle Weight (from Section 3.2 for each loading type)(kips)
I = Impact Factor
DF = Distribution Factor

The formula to be used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(M_a i)(GVW)}{(\frac{1}{2})(LLM)(I)(DF)}$$

The Inventory Rating should be calculated for each loading type described in Section 3.2 and entered in the appropriate spaces in the "Summary of Ratings" table in Chapter 8 under the heading "Channel Moment Rating".

C5: Note from Plate III in Appendix A that for a 30.5 ft. span two axles are placed on the span for maximum moment; therefore, the GVW to be used is 80 kips.

DATE	DESIGN	Sec. 7.3.4 Example Computations Cont.		SHEET OF
	CHECK	JOB	FOR	JOB NO.

SUBJECT

Calculate Inventory Ratings For Moment Capacity

SU2 : $\dfrac{(70.16)(34.0)}{(1/2)(187.1)(1.30)(0.605)} =$ 32.4 Kips

SU3 : $\dfrac{(70.16)(66.0)}{(1/2)(339.2)(1.30)(0.605)} =$ 34.7 Kips

SU4 : $\dfrac{(70.16)(70.0)}{(1/2)(367.2)(1.30)(0.605)} =$ 34.0 Kips

C3 : $\dfrac{(70.16)(56.0)}{(1/2)(202.7)(1.30)(0.605)} =$ 49.2 Kips

C4 : $\dfrac{(70.16)(73.271)}{(1/2)(291.2)(1.30)(0.605)} =$ 44.9 Kips

C5 : $\dfrac{(70.16)(80.0)}{(1/2)(291.2)(1.30)(0.605)} =$ 49.0 Kips

H : $\dfrac{(70.16)(40.0)}{(1/2)(251.6)(1.30)(0.605)} =$ 28.4 Kips

STEP 5. 3. DETERMINE OPERATING RATING FOR MOMENT CAPACITY.

A photocopy of the forms "Section 12.4 - Moment Capacity rating at Operating Level" should be obtained and the Operating Rating computed by performing the computations detailed thereon.

The dimension for the flange thickness (T^f) has been reduced 1/4" to account for scaling.

The area of strands is determined to be 0.1438 sq. in from Plate IV in Appendix A for 1/2 in. dia. ASTM Grade (250^k) strands.

The ultimate strength per strand is found to be 36,000 lbs. from Plate IV in Appendix A for 1/2 in. dia. ASTM Grade (250^k) strands.

"d" is the distance from the centroid of the prestressing force to the extreme compressive fiber (i.e. the top of cast-in-place concrete surface).

W_d = Web depth from Section 2.5

T_f = Flange thickness of P/S Section.

$\overline{Y}_1$, $\overline{Y}_2$ = Distance from the bottom of Channel to the centroid of the prestressing strands from Section 7.3.4, Figure 7-5, on page 7-43.

A*s is the area of the prestressing steel.

P* is the ratio of prestressing steel.

f's is the ultimate strength of the prestressing steel.

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SECTION 7.3.5 - MOMENT CAPACITY RATING AT OPERATING LEVEL
(Considering Strength Requirements)

Begin by computing the following parameters:

Wc = nominal channel width = 32 in.

(From Section 2.5)

$d = W_d - Y_1 + T_f + T_d$

$= (14 \text{ in.}) - (5.333 \text{ in.}) + (4 \text{ in.}) + (3.09 \text{ in.})$

$= 15.757$ in.

A^*_s = (Number of Strands)(Area of Strands)*

$= (6)(.1438 \text{ in.}^2) = .8628 \text{ in.}^2$

$$p^* = \frac{A^*_s}{(Wc)(d)} = \frac{(.8628 \text{ in.}^2)}{(32 \text{ in.})(15.757 \text{ in.})} = .001711$$

$$f's = \frac{\text{Ultimate Strength Per Strand}}{\text{Area Per Strand}}$$

$$= \frac{(36,000 \text{ lb.})}{(.1438 \text{ in}^2)} = 250,348 \text{ lb/in.}^2$$

* Strand areas and properties are tabulated for common strand sizes on Plate IV in Appendix A.

f*su is the average stress in the prestressing steel at ultimate load.

17TH ED

$$f^*_{su} = f_s\left[\left(1-\left(\frac{\gamma}{\beta_1}\right)(p)\left(\frac{f_s}{f_c}\right)\right)\right]$$

$\gamma \begin{cases} 0.28 & \text{LL} \\ 0.40 & \text{SR} \\ 0.55 & \text{BARS} \end{cases}$

$$= f_s\left(1 - \frac{\gamma}{\beta_1} \cdot p \cdot \frac{f_s}{f_c}\right)$$

COMP
CONC
(DECK)
TYP

0.85 @ 4ksi   0.65 @

$$f^*_{su} = (f's)[1 - \frac{(0.5(p^*)(f's)}{f'c}]$$

$$= (250{,}348 \text{ psi.})[1 - \frac{(0.5)(.001711)(250{,}348 \text{ psi})}{(5000 \text{ psi})}]$$

$$= \underline{239{,}624} \text{ psi.}$$

CHECK MAXIMUM STEEL PERCENTAGE:

$p^*\ f^*_{su}/f'c \leq .30$

$(.001711)(\quad \text{psi})/(5000 \text{ psi}) = .082 < .30$ OK

Compute Ultimate Moment Capacity (Mu):

IF STEEL PERCENTAGE $\leq$ .30

$$Mu = A^*_s f^*_{su} d[1 - \frac{(0.6)(p^*)(f^*_{su})}{f'c}]$$

$$Mu = (.8628 \text{in}^2)(239{,}624 \text{psi})(15.757 \text{in})[1 - \frac{(0.6)(.001711)(239{,}624 \text{psi})}{(5000 \text{ psi})}]$$

$$Mu = \underline{3{,}097{,}443} \text{ in-lbs} \div 12{,}000 = \boxed{258.12} \text{ ft.-K}$$

IF STEEL PERECENTAGE > .30:

Since steel percentage was $\leq$ .30, this formula is not used.

$Mu = .25\ f'cW_cd^2$

$Mu = .25\ (.\quad \text{psi})(\quad \text{in})(\quad \text{in})^2\ (1/12000) = \_\_\_\_\_\_ \text{ft.-K.}$

Mu = Ultimate Moment Capacity of the member (ft-kips)
ΣM_{DL} = Total Dead Load Moment = M_{DL1} + M_{DL2} from Section 7.3.4 (in-lbs)
LLM = Applied Live Load Moment (from Section 3.2 for each loading type)(ft-kips)
GVW = Gross Vehicle Weight (from Section 3.2 for each loading type)(kips)
I = Impact Factor (from Section 7.3.4)
DF = Distribution Factor (from Section 7.3.4)

Calculate Operating Moment Ratings:

The formula to be used to compute the Operating Rating Factor (RF) for each loading type is:

$$RF = \frac{(2)\left[\frac{Mu}{1.3} - \frac{\Sigma M_{DL}}{12,000}\right]}{(LLM)(I)(DF)}$$

Notice that the numerator of the Rating Factor equation does not change for the different load cases (only LLM changes). Therefore the numerator needs to be calculated only once and the denominator re-calculated for each load case.

The Operating Rating should be calculated by multiplying the Rating Factor by the GVW for each loading type described in Section 3.2 and entered in the appropriate spaces in the "Summary of Ratings" table in Chapter 8 under the heading "Channel Moment Rating".

As indicated in Section 7.3.5, the numerator of the Rating Factor equation does not change for each loading case, so evaluate this part first.

C5: Note from Plate III in Appendix A that for a 30.5 ft. span two axles are placed on the span for maximum moment, therefore, the GVW to be used is 80 kips.

CALCULATE OPERATING RATINGS FOR MOMENT CAPACITY

Evaluate numerator of rating factor equation:

$$2\left[\frac{1.0(258.12)}{1.3} - \frac{575{,}592+60880}{12{,}000}\right] = 291.03$$

SU2:
$$\frac{291.03}{(187.1)(1.3)(0.605)} = 1.97$$

34.0 (1.97) = 67.0 Kips

SU3:
$$\frac{291.03}{(399.2)(1.3)(0.605)} = 0.927$$

66.0 (0.927) = 61.1 Kips

SU4:
$$\frac{291.03}{(367.2)(1.3)(0.605)} = 1.01$$

70.0 (1.01) = 70.7 Kips

C3:
$$\frac{291.03}{(202.7)(1.3)(0.605)} = 1.83$$

56.0 (1.83) = 102.5 Kips

C4:
$$\frac{291.03}{(291.2)(1.3)(0.605)} = 1.27$$

73.271 (1.27) = 93.05 Kips

C5:
$$\frac{291.03}{(291.2)(1.3)(0.605)} = 1.27$$

80.0 (1.27) = 101.6 Kips

H:
$$\frac{291.03}{(251.6)(1.3)(0.605)} = 1.47$$

40.0 (1.47) = 58.8 Kips

STEP 5 3. DETERMINE INVENTORY AND OPERATING RATINGS FOR SHEAR CAPACITY IN OUTER QUARTERS OF CHANNEL.

A photocopy of the forms "Section 12.4 - Shear Capacity Ratings for Outer Quarters of Channel" should be obtained and the ratings computed by performing the computations detailed thereon.

The deck thickness (T_d) has been reduced 1/4" to account for scaling.

APPLIED DEAD LOAD SHEAR is the shear resulting from the computed dead load per linear foot of channel.

ΣW_{DL} = Total Dead Load per linear foot of channel from Section 7.3.3. ($W_{DL1} + W_{DL2}$)

"d" is the distance from the centroid of the prestressing strands to the extreme compressive fiber of the section (i.e. the top of the cast-in-place concrete surface).

W_d = Web depth in inches from Section 2.5.

T_F = Flange thickness

T_d = Thickness of cast-in-place concrete deck surface from Sec. 7.3.4

$\bar{y}$ midspan = Location of the centroid of the prestressing strands from the bottom of the channel at midspan (in inches) from Section 7.3.4

$\bar{y}$ ends = Location of the centroid of the prestressing strands from the bottom of the channel at ends of channel (in inches), from Section 7.3.4

SECTION 7.3.6 - SHEAR CAPACITY RATINGS FOR OUTER QUARTERS OF CHANNEL

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = \frac{(\Sigma W_{DL}))\ (\text{Span Length})}{4(1000)} = \frac{(456.13\ \text{lb/ft.})(30.5\ \text{ft.})}{4000}$$

$$V_{DL} = \boxed{3.48}\ \text{kips}$$

Compute Impact Factor (I):

$$I = 1 + \frac{50}{(3/4)(\text{Span Length}) + 125},\ \text{but not greater than 1.30}$$

$$I = 1 + \frac{50}{(3/4)(30.5\ \text{ft}) + 125} = \underline{1.34}$$

Use $I = \boxed{1.30}$

Compute "d":

$$d = W_d + T_f + T_d - [\bar{y}\ \text{midspan} + 1/2\ (\bar{y}\ \text{ends} - \bar{y}\ \text{midspan})]$$

$$d = (14\ \text{in}) + (4\ \text{in.}) + (3.09\ \text{in.}) - [(5.333\ \text{in}) + (1/2)(5.333\ \text{in} - 5.333\ \text{in})]$$

$$d = \boxed{15.757}\ \text{in.}$$

"j" is a parameter to be used in the equation for computing the shear carried by the concrete, and represents the ratio of the distance between centroid of compression and centroid of tension to the depth d.

p* is the ratio of prestressing steel, where:

A*s = Area of prestressing steel from Section 7.3.5
Wc = Nominal Channel Width from Section 2.5.
d = Distance from the centroid of the prestressing strands to the extreme compressive fiber of the section from Section 7.3.4.

f*su is the average stress in the prestressing steel at ultimate load, where:

f's = Ultimate strength of prestressing steel from Section 7.3.6.
f'c = Concrete strength for prestressed channels from Section 2.5.

Compute "j":

$$j = 1 - \frac{0.6p^{*}f^{*}_{su}}{f'c}$$

$$p^{*} = \frac{A^{*}s}{(W_c)(d)}$$

$$p^{*} = \frac{(.8628 \text{ in}^2)}{(32 \text{ in})(15.757 \text{ in})}$$

$$p^{*} = \underline{.001711}$$

$$f^{*}_{su} = f's \ 1 - \frac{0.5 \ p^{*}f's}{f'c}$$

$$f^{*}_{su} = (250\ 348 \text{ psi.})\left[1 - \frac{0.5(.001711)\ (250\ 348 \text{ psi.})}{(5000 \text{ psi.})}\right]$$

$$f^{*}_{su} = \underline{239\ 624} \text{ psi.}$$

$$\text{so, } j = 1 - \frac{0.6(.001711)(239\ 624 \text{ psi.})}{(5000 \text{ psi.})} = \boxed{0.951}$$

f'c = Concrete strength for prestressed channels from Section 2.5.

j = Ratio of distance between centroid of compression and centroid of tension to depth d, from Section 7.3.6.

d = Distance from the centroid of the prestressing strands to the extreme compressive fiber of the section, from Section 7.3.6

w_t and w_b = Web width at top and web width at bottom, respectively, from Section 2.5.

Av = Cross-sectional area of one leg of stirrup. Obtain bar size from Section 2.5 and refer to Plate IV in Appendix A for cross-sectional area. (in.^{2})

fy = Yield strength of the stirrups from Section 2.5 (psi).

j and d: See above.

Stirrup spacing is from Section 2.5.

From the Inspection Data Sheet, we find that for the outer quarters the stirrup spacing is 6 inches and the bar size is #3. From Plate IV in Appendix A, it is found that the area of a #3 bar is 0.11 sq. in. The yield strength of the stirrups is given in the Inspection Data Sheet as 40,000 psi.

Compute Shear Carried by Concrete (Vc):

$Vc = 0.06\ f'c\ jd\ (w_t + w_b)$

but not greater than $180\ jd(w_t + w_b)$

$Vc = 0.06\ (5000\text{psi.})(.951)(15.757\text{in})(6\text{ in} + 4\text{ in})$

$Vc = \underline{44\,955}$ lbs ÷ 1000 = $\underline{44.95}$ kips

$Vc = 180\ (.951)(15.757\text{in})(6\text{ in} + 4\text{ in})$

$Vc = \underline{26\,973}$ lbs ÷ 1000 = $\underline{26.97}$ kips

Use Vc = $\boxed{26.97}$ kips

Compute Shear Carried by Stirrups (Vs):

$$Vs = \frac{2(Av)(fy)(j)(d)}{(\text{Stirrup Spacing})(1000)}$$

$$Vs = \frac{2(0.11\text{ in}^2)(40\,000\text{psi.})(.951)(15.757\text{in})}{(6\text{ in})(1000)}$$

Vs = $\boxed{21.98}$ kips

Compute Ultimate Shear Capacity of Channel (Vu):

Vu = Vc + Vs = (26.97 kips) + (21.98 kips)

Vu = $\boxed{48.95}$ kips

SECTION 7.3.6 - SHEAR CAPACITY RATINGS FOR OUTER QUARTERS OF CHANNEL (CONT.)

Calculate Inventory and Operating Shear Ratings for Outer Quarters:

The following formulae are to be used to compute Inventory and Operating Shear Ratings for each loading case. The Live Load Shear (LLV) used in the formulae is to be taken from Section 3.3 and the distribution factor is from Section 7.3.4. Notice that the numerator of the rating factor equations does not change for the different loading cases, therefore, it will need to be calculated only once. The denominator changes and will need to be recalculated for each different loading case. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the Gross Vehicle Weight (GVW) to obtain the rating. The rating should then be entered under the appropriate type of loading in the "Summary of Ratings" table in Chapter 8, under the heading of "Shear Rating for Outer Quarters".

Inventory Ratings:

$$RF = \frac{\left[\frac{(0.90)(Vu)}{1.3} - V_{DL}\right]\left[\frac{3}{5}\right]}{(1/2)(DF)(LLV)(I)}$$

Operating Rating:

$$RF = \frac{\frac{(0.90)(Vu)}{1.3} - V_{DL}}{(1/2)(DF)(LLV)(I)}$$

As indicated in Section 7.3.6, the numerator of the Rating Factor equations does not change for each loading case so evaluate these parts first.

CALCULATE INVENTORY AND OPERATING RATINGS FOR SHEAR CAPACITY IN OUTER QUARTERS OF CHANNEL

Evaluate the numerator of inventory rating factor equation

$$\left[\frac{0.9\times(48.95)}{1.3}-3.48\right]\left(\frac{3}{5}\right)=18.24$$

Evaluate the numerator of operating rating factor equation:

$$\left[\frac{0.9\,(48.95)}{1.3}-3.48\right]=30.40$$

SU2: INV. $\dfrac{18.24}{(1/2)(0.605)(20.39)(1.3)}=2.27$

$34.0\,(2.27) = 77.2$ Kips

O/p. $\dfrac{30.4}{(1/2)(0.605)(20.39)(1.3)}=3.79$

$34.0\,(3.79) = 128.9$ Kips

SU3 INV. $\dfrac{18.24}{(1/2)(0.605)(35.55)(1.3}=1.30$

$66.0\,(1.3) = 85.8$ Kips

Op. $\dfrac{30.4}{(1/2)(0.605)(35.55)(1.3}=2.17$

$66.0\,(2.17) = 143.2$ Kips

SU4 INV. $\dfrac{18.24}{(1/2)(0.605)(36.86)(1.3}=1.25$

$70.0\,(1.25) = 87.5$ Kips

Op. $\dfrac{30.4}{(1/2)(0.605)(36.86)(1.3)}=2.09$

$70.0\,(2.09) = 146.3$ Kips

Referring to the calculation of Live Load Shear at the quarter point for the C5 load case, it is found that three axles were used in computing the maximum Live Load Shear. Therefore, in accordance with Plate III in Appendix A, a GVW of 80 kips is used.

SEC. 7.3.6-EXAMPLE COMPUTATIONS CONT.

SUBJECT

C_3: INV. $\dfrac{18.24}{(1/2)(0.605)(21.57)(1.3)} = 2.15$

$56.0\,(2.15) = 120.4$ Kips

Op. $\dfrac{30.40}{(1/2)(0.605)(21.57)(1.3)} = 3.58$

$56.0\,(3.58) = 200.5$ Kips

C_4: INV. $\dfrac{18.24}{(1/2)(0.605)(29.99)(1.3)} = 1.54$

$73.271\,(1.54) = 112.8$ Kips

Op. $\dfrac{30.40}{(1/2)(0.605)(29.99)(1.3)} = 2.57$

$73.271\,(2.57) = 188.3$ Kips

C_5: INV. $\dfrac{18.24}{(1/2)(0.605)(30.12)(1.3)} = 1.53$

$80.0\,(1.53) = 122.4$ Kips

Op. $\dfrac{30.40}{(1/2)(0.605)(30.12)(1.3)} = 2.56$

$80.0\,(2.56) = 204.8$ Kips

H: INV. $\dfrac{18.24}{(1/2)(0.605)(26.33)(1.3)} = 1.76$

$40.0\,(1.76) = 70.4$ Kips

Op. $\dfrac{30.40}{(1/2)(0.605)(26.33)(1.3)} = 2.93$

$40.0\,(2.93) = 117.2$ Kips

STEP 5 4. DETERMINE INVENTORY AND OPERATING RATINGS FOR SHEAR CAPACITY IN MIDDLE THIRD OF CHANNEL.

A photocopy of the forms "Section 12.4 - Shear Capacity Ratings for Middle Third of Channel" should be obtained and the ratings computed by performing the computations detailed thereon.

The flange thickness has been reduced 1/4" to account for scaling.

ΣW_{DL} = Total Dead Load per linear foot of channel from Section 7.3.3. (W_{DL1} + W_{DL2})

"d" is the distance from the centroid of the prestressing strands to the extreme compressive fiber of the section (i.e. the top of the cast-in-place concrete surface).

W_d = Web depth in inches from Section 2.5.

T_F = Flange thickness

T_d = Thickness of cast-in-place concrete deck surface from Sec. 7.3.4

$\bar{y}$ midspan = Location of the centroid of the prestressing strands from the bottom of the channel at midspan (in inches) from Section 7.3.4

$\bar{y}$ ends = Location of the centroid of the prestressing strands from the bottom of the channel at ends of channel (in inches), from Section 7.3.4

SECTION 7.3.7 - SHEAR CAPACITY RATINGS FOR MIDDLE THIRD OF CHANNEL

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = \frac{(\Sigma W_{DL})\ \ (\text{Span Length})}{6(1000)} = \frac{(456.13\ \text{lb/ft.})(30.5\ \text{ft.})}{6000}$$

$$V_{DL} = \boxed{2.32}\ \text{kips}$$

Compute Impact Factor (I):

$$I = 1 + \frac{50}{(2/3)(\text{Span Length}) + 125}, \text{ but not greater than } 1.30$$

$$I = 1 + \frac{50}{(2/3)(30.5\ \text{ft}) + 125} = \underline{1.34}$$

$$\text{Use } I = \boxed{1.30}$$

Compute "d":

$$d = W_d + T_f + T_d - [\bar{y}\ \text{midspan} + 1/3\ (\bar{y}\ \text{ends} - \bar{y}\ \text{midspan})]$$

$$d = (14\ \text{in}) + (4\ \text{in.}) + (3.09\ \text{in.}) - [(5.333\ \text{in}) + (1/3)(15.333\ \text{in} - 5.33\ \text{in})]$$

$$d = \boxed{15.757}\ \text{in.}$$

"j" is a parameter to be used in the equation for computing the shear carried by the concrete, and represents the ratio of the distance between centroid of compression and centroid of tension to the depth d.

p* is the ratio of the prestressing steel, where:

A*s = Area of prestressing steel from Section 7.3.5.
Wc = Nominal channel width from Section 2.5.
d = Distance from the centroid of the prestressing strands to the extreme compressive fiber of the section from Section 7.3.6.

f^*_{su} is the average stress in the prestressing steel at ultimate load, where:

f's = Ultimate strength of prestressing steel from Section 7.3.5.
f'c = Concrete strength for prestressing channels from Section 2.5.

Compute "j":

$$j = 1 - \frac{0.6p^*f^*_{su}}{f'c}$$

$$p^* = \frac{A^*s}{(W_c)(d)}$$

$$p^* = \frac{.8628\,\text{in}^2}{(32\ \text{in})(15.757\ \text{in})}$$

$$p^* = \underline{\quad .001711 \quad}$$

$$f^*_{su} = f's(1- \frac{0.5\ p^*f's}{f'c})$$

$$f^*_{su} = (250348\ \text{psi.})[1 - \frac{0.5(.001711)(250348\ \text{psi.})}{(5000\ \text{psi.})}]$$

$$f^*_{su} = \underline{\quad 239624 \quad}\ \text{psi.}$$

$$\text{so, } j = 1 - \frac{0.6(.001711)(239624\ \text{psi.})}{(5000\ \text{psi.})} = \boxed{0.951}$$

f'c = Concrete strength for prestressed channels from Section 2.5.

j = Ratio of distance between centroid of compression and centroid of tension to depth d, from Section 7.3.7

d = Distance from the centroid of the prestressing strands to the extreme compressive fiber of the section, from Section 7.3.6.

W_f and W_b = Web width at top and web width at bottom, respectively, from Section 2.5.

Av = Cross-sectional area of one leg of stirrup. Obtain bar size from Section 2.5 and refer to Plate IV in Appendix A for cross-sectional area (in^2).

fy = Yield strength of the stirrups from Section 2.5 (psi).

j and d: See above.

Stirrup spacing is from Section 2.5.

From the Inspection Data Sheet we find that in the outer quarters, the stirrup spacing is 10 inches and the bar size is #3. From Plate III in Appendix A it is found that the area of a #3 bar is 0.11 sq. in. The yield strength of the stirrups is given in the Inspection Data Sheet as 40,000 psi.

SECTION 7.3.7 - SHEAR CAPACITY RATINGS FOR MIDDLE THIRD OF CHANNEL (CONT.)

Compute Shear Carried by Concrete (Vc):

$Vc = 0.06\ f'c\ jd\ (w_t + w_b)$

but not greater than $180\ jd(w_t + w_b)$

Vc = 0.06 (5000 psi.)(.951)(15.757 in)(6 in + 4 in)

Vc = 44955 lbs ÷ 1000 = 44.95 kips

Vc = 180 (.951)(15.757 in)(6 in + 4 in)

Vc = 26973 lbs ÷ 1000 = 26.97 kips

Use Vc = $\boxed{26.97}$ kips

Compute Shear Carried by Stirrups (Vs):

$$Vs = \frac{2(Av)(fy)(j)(d)}{(\text{Stirrup Spacing})(1000)}$$

$$Vs = \frac{2(\ 0.11\ in^2)(40000 psi.)(\ .951\ )(15.757\ in)}{(\ 10\ in)\ (1000)}$$

Vs = $\boxed{13.19}$ kips

Compute Ultimate Shear Capacity of Channel (Vu):

Vu = Vc + Vs = (26.97 kips) + (13.19 kips)

Vu = $\boxed{40.16}$ kips

Calculate Inventory and Operating Shear Ratings for Middle Third:

The following formulae are to be used to compute Inventory and Operating Shear Ratings for each loading case. The Live Load Shear (LLV) used in the formulae is to be taken from Section 3.3 and the distribution factor is from Section 7.3.4. Notice that the numerator of the rating factor equations do not change for the different loading cases; therefore, it will need to be calculated only once. The denominator changes and will need to be recalculated for each different loading case. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the Gross Vehicle Weight (GVW) to obtain the rating. The rating should then be entered under the appropriate type of loading in the "Summary of Ratings" table in Chapter 8, under the heading of "Shear Rating for Outer Quarters".

Inventory Ratings:

$$RF = \frac{\left[\frac{(0.90)(Vu)}{1.3} - V_{DL}\right]\left[\frac{3}{5}\right]}{(1/2)(DF)(LLV)(I)}$$

Operating Rating:

$$RF = \frac{\frac{(0.90)(Vu)}{1.3} - V_{DL}}{(1/2)(DF)(LLV)(I)}$$

As indicated in Section 7.3.7, the numerator of the Rating Factor equations does not change for each loading case so evaluate these parts first.

CALCULATE INVENTORY AND OPERATING RATINGS
FOR SHEAR CAPACITY IN MIDDLE THIRD OF CHANNEL.

EVALUATE the numerator of inventory rating factor equation:

$$\left[\frac{0.9(40.16)}{1.3} - 2.32\right]\left(\frac{3}{5}\right) = 15.28$$

EVALUATE the numerator of operating rating factor equation:

$$\left[\frac{0.9(40.16)}{1.3} - 2.32\right] = 25.48$$

SU2: INV. $\frac{15.28}{(1/2)(0.605)(17.55)(1.3)} = 2.21$

$34.0\,(2.21) = 75.1$ Kips

Op. $\frac{25.48}{(1/2)(0.605)(17.55)(1.3)} = 3.69$

$34.0\,(3.69) = 125.5$ Kips

SU3 INV. $\frac{15.28}{(1/2)(0.605)(30.05)(1.3)} = 1.29$

$66.0\,(1.29) = 85.1$ Kips

Op. $\frac{25.48}{(1/2)(0.605)(30.05)(1.3)} = 2.16$

$66.0\,(2.16) = 142.6$ kips

SU4 INV. $\frac{15.28}{(1/2)(0.605)(31.03)(1.3)} = 1.25$

$70.0\,(1.25) = 87.5$ Kips

Op. $\frac{25.48}{(1/2)(0.605)(31.03)(1.3)} = 2.08$

$70.0\,(2.08) = 145.6$ Kips

Referring to the calculation of Live Load Shear at the quarter point for the C5 load case, it is found that three axles were used in computing the maximum Live Load Shear. Therefore in accordance with Plate III in Appendix A, a GVW of 80 kips is used.

C_3: INV. $\dfrac{15.28}{(1/2)(0.605)(18.73)(1.3)} = 2.07$

$56.0\,(2.07) = 115.9$ Kips

OP. $\dfrac{25.48}{(1/2)(0.605)(18.73)(1.3)} = 3.45$

$56.0\,(3.45) = 193.2$ Kips

C_4: INV. $\dfrac{15.28}{(1/2)(0.605)(26.33)(1.3)} = 1.47$

$73.271\,(1.47) = 107.7$ Kips

OP. $\dfrac{25.48}{(1/2)(0.605)(26.33)(1.3)} = 2.46$

$73.271\,(2.46) = 180.2$ Kips

C_5: INV. $\dfrac{15.28}{(1/2)(0.605)(26.33)(1.3)} = 1.47$

$80.0\,(1.47) = 117.6$ Kips

OP. $\dfrac{25.48}{(1/2)(0.605)(26.33)(1.3)} = 2.46$

$80.0\,(2.46) = 196.8$ Kips

H: INV. $\dfrac{15.28}{(1/2)(0.605)(22.99)(1.3)} = 1.69$

$40.0\,(1.69) = 67.6$ Kips

OP. $\dfrac{25.48}{(1/2)(0.605)(22.99)(1.3)} = 2.82$

$40.0\,(2.82) = 112.8$ Kips

Finally the Inventory and Operating capacities for each load case under each category are entered in the Summary of Ratings Table. The controlling condition is determined by choosing the lowest rating for each type of loading at Inventory and Operating Levels. This completes the Load Rating for the example problem.

SEC. 7.3.8 - EXAMPLE COMPUTATIONS CONT.

Loading Classification	TYPE OF LOADING	RATING LEVEL	MOMENT CAPACITY RATING		SHEAR FOR OUTER 1/4		SHEAR FOR MIDDLE 1/3						CONTROLLING RATING
Florida Legal Loading	SU2	Inventory	32.4		77.2		75.1						32.4
		Operating	67.0		128.9		125.5						67.0
	SU3	Inventory	34.7		85.8		85.1						34.7
		Operating	61.1		143.2		142.6						61.1
	SU4	Inventory	34.0		87.5		87.5						34.0
		Operating	70.7		146.3		145.6						70.7
	C3	Inventory	49.2		120.4		115.9						49.2
		Operating	102.5		200.5		193.2						102.5
	C4	Inventory	44.9		112.8		107.7						44.9
		Operating	93.0		188.3		180.2						93.0
	C5	Inventory	49.0		122.4		117.6						49.0
		Operating	101.6		204.8		196.8						101.6
Design Loading	H	Inventory	28.4		70.4		67.6						28.4
		Operating	58.8		117.2		112.8						58.8
	HS	Inventory											
		Operating											

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

CHAPTER 8

SUMMARY OF RATINGS

SUMMARY OF RATINGS

This section serves to compile the rating results in tabular form. This will facilitate the comparison of the ratings for the different members in order to arrive at the controlling (lowest) Inventory and Operating Ratings for each type of loading.

As the rating values for each individual condition are computed the appropriate heading and the corresponding rating values for each loading type should be entered on the "Summary of Ratings" table. When the rating calculations are finished and the rating values noted in the "Summary of Ratings" table each row should be inspected to determine the controlling rating. The controlling Inventory and Operating Rating for each type of loading should then be entered under the heading "Superstructure Rating". For ratings where sidewalk loading has been determined, those rating values should not be considered when determining the controling rating. They were computed for comparison purposes only and should be circled to identify them. After completion of this table the rating computations should be stapled together or neatly bound for future reference.

At this time, a determination as to the posting or maintenance requirements for the structure should be made by a Registered Professional Engineer. The ratings calculated under "Superstructure Rating" represent the maximum GVW for each loading type at Inventory and Operating Rating Levels but does not necessarily represent the values that the Professional Engineer deems practicable for posting of that particular structure.

"There may be circumstances where an agency responsible for the maintenance of structures maintains a level of inspection and surveillance which greatly exceeds the minimum required; an inspection program which in fact insures the detection of problem areas in advance of actual detrimental behavior. Under this level of inspection and with the assurance that the load history of the bridge will be closely monitored with respect to the frequency of loading with various sized vehicles, to preclude the possibility of fatigue failure, the Professional Engineer may, in his judgment, utilize for posting purposes, load levels higher than those used for Inventory Rating, in order to minimize the need for posting of bridges. In no case should the load levels used be greater than those permitted by the Operating Rating."[1]

Other circumstances which could affect the rating of a structure are items such as Average Daily Traffic (ADT) and frequency of critical vehicle occurence. Structures which are seldom utilized by critical vehicles need not necessarily be posted at the calculated level when the Professional Engineer feels these insignificantly stress the bridge.

[1] Manual for Maintenance Inspection of Bridges, 1978, pgs. 24 & 25.

"A reduced speed limit will be considered where it is desirable to reduce impact loads. It will be found that in some cases, reduced speed limits will reduce impact loads to the extent that lowering the weight limit will not be required. Consideration of a speed posting will require the judgment of the Engineer and much will depend upon alignment, general location, volume and type of traffic. A speed posting should not be considered as a basis for increasing the weight limit in areas where enforcement will be difficult and frequent violations can be anticipated."[2]

"In many cases the physical inspection of the structure will be all that is required for a qualified Engineer to make a judgment that the bridge is safe for all legal loads. Such a case could be a sound concrete bridge which has been carrying normal traffic for many years and shows no distress."[3]

The above examples only represent a few of the areas that the Professional Engineer may consider when making a determination as to the rating of a structure.

The Rating Values calculated by the methods presented herein should supplement the Professional Engineer in making his decisions regarding the structure. They are not intended to solely represent the values at which a structure should be posted.

2 & 3 Manual for Maintenance Inspection of Bridges, 1978, pgs. 24 & 25.

Loading Classification	TYPE OF LOADING	RATING LEVEL											
Florida Legal Loading	SU2	Inventory											
		Operating											
	SU3	Inventory											
		Operating											
	SU4	Inventory											
		Operating											
	C3	Inventory											
		Operating											
	C4	Inventory											
		Operating											
	C5	Inventory											
		Operating											
Design Loading	H	Inventory											
		Operating											
	HS	Inventory											
		Operating											

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

CHAPTER 9

BIBLIOGRAPHY

BIBLIOGRAPHY

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APPENDIX A

Plate I	-	Live Load Moments on Longitudinal Stringers or Girders.
Plate II	-	Wheel Load Distribution Factors
Plate III	-	Maximum Florida Legal Load Cases and AASHTO Design Load Cases and their Placement on Various Simple Spans to Yield Maximum Moment
Plate IV	-	Data on Standard Deformed Reinforcing Bars and Data on Standard Seven-wire Uncoated Stress-relieved Prestressing Strands
Plate V	-	Decimals for a Foot of Each 1/8th of an Inch and Decimals for an Inch for Each 1/8th of an Inch

PLATE I

LIVE LOAD MOMENTS ON LONGITUDINAL STRINGERS OR GIRDERS
(In Foot-Kips; Simple span, One, Lane, Impact Not Included)

Span (ft.) C. to C.	TYPE OF LOADING							
	SU2	SU3	SU4	C3	C4	C5	H-20	HS-20
5.0	27.5	27.5	23.4	27.5	27.5	27.5	40.0	40.0
5.5	30.3	30.3	25.7	30.3	30.3	30.3	44.0	44.0
6.0	33.0	33.0	28.1	33.0	33.0	33.0	48.0	48.0
6.5	35.8	35.8	30.4	35.8	35.8	35.8	52.0	52.0
7.0	38.5	38.5	32.7	38.5	38.5	38.5	56.0	56.0
7.5	41.3	43.0	36.6	41.3	43.0	43.0	60.0	60.0
8.0	44.0	48.1	40.9	44.0	48.1	48.1	64.0	64.0
8.5	46.8	53.3	45.3	46.8	53.3	53.3	68.0	68.0
9.0	49.5	58.5	49.7	49.5	58.5	58.5	72.0	72.0
9.5	52.3	63.7	55.3	52.3	63.7	63.7	76.0	76.0
10.0	55.0	68.9	62.3	55.0	68.9	68.9	80.0	80.0
10.5	57.8	74.2	69.3	57.8	74.2	74.2	84.0	84.0
11.0	60.5	79.5	76.4	60.5	79.5	79.5	88.0	88.0
11.5	63.3	84.8	83.4	63.3	84.8	84.8	92.0	92.0
12.0	66.0	90.1	90.4	66.0	90.1	90.1	96.0	96.0
12.5	68.8	95.5	97.4	68.8	95.5	95.5	100.0	100.0
13.0	71.5	100.8	104.4	71.5	100.8	100.8	104.0	104.0
13.5	74.3	106.2	111.4	74.3	106.2	106.2	108.0	108.0
14.0	77.0	111.6	118.4	77.0	111.6	111.6	112.0	112.0
14.5	79.8	117.0	125.4	79.8	117.0	117.0	116.0	116.0
15.0	82.5	122.3	132.5	82.5	122.3	122.3	120.0	120.0
15.5	85.3	127.7	139.5	85.3	127.7	127.7	124.0	124.0
16.0	88.0	133.2	146.5	88.0	133.2	133.2	128.0	128.0
16.5	90.8	138.6	153.5	90.8	138.6	138.6	132.0	132.0
17.0	93.5	144.0	160.5	93.5	144.0	144.0	136.0	136.0
17.5	96.3	149.4	167.5	96.3	149.4	149.4	140.0	140.0
18.0	99.0	154.8	174.5	99.0	154.8	154.8	144.0	144.0
18.5	101.8	160.2	181.5	103.0	160.2	160.2	148.0	148.0
19.0	104.5	165.7	188.6	107.1	165.7	165.7	152.0	152.0
19.5	107.3	171.1	195.6	111.2	171.1	171.1	156.0	156.0
20.0	110.0	176.6	202.6	115.3	176.6	176.6	160.0	160.0
20.5	112.8	182.0	209.6	119.4	182.0	182.0	164.0	164.0
21.0	115.5	187.4	216.6	123.5	187.4	187.4	168.0	168.0
21.5	118.3	192.9	223.6	127.7	192.9	192.9	172.0	172.0
22.0	121.0	200.1	230.6	131.8	198.3	198.3	176.0	176.0
22.5	123.8	208.2	237.6	136.0	203.8	203.8	180.0	180.0
23.0	126.5	216.4	244.7	140.1	209.2	209.2	184.0	184.0
23.5	129.4	224.6	251.7	144.3	214.7	214.7	188.0	188.0
24.0	133.5	232.7	258.7	148.4	220.2	220.2	192.0	192.7
24.5	137.6	240.9	265.7	152.6	225.6	225.6	196.0	200.0

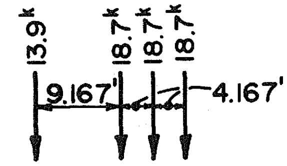
PLATE I - Continued

Span (ft.) C. to C.	TYPE OF LOADING							
	SU2	SU3	SU4	C3	C4	C5	H-20	HS-20
25.0	141.7	249.1	272.7	156.7	231.1	231.1	200.0	207.4
25.5	145.8	257.3	280.5	160.9	236.5	236.5	204.0	214.7
26.0	149.9	265.5	289.1	165.1	242.0	242.0	208.0	222.2
26.5	154.0	273.6	297.8	169.2	247.5	247.5	212.0	229.6
27.0	158.1	281.8	306.5	173.4	252.9	252.9	216.9	237.0
27.5	162.3	290.0	315.1	177.6	258.4	258.4	221.8	244.5
28.0	166.4	298.2	323.8	181.8	263.9	263.9	226.8	252.0
28.5	170.5	306.4	332.5	186.0	269.3	269.3	231.7	259.5
29.0	174.7	314.6	341.1	190.2	274.8	274.8	236.7	267.0
29.5	178.8	322.8	349.8	194.3	280.3	280.3	241.6	274.6
30.0	183.0	331.0	358.5	198.5	285.8	285.8	246.6	282.1
30.5	187.1	339.2	367.2	202.7	291.2	291.2	251.6	289.7
31.0	191.3	347.4	375.9	206.9	296.7	296.7	256.5	297.3
31.5	195.4	355.6	384.6	211.1	302.2	302.2	261.5	304.9
32.0	199.6	363.8	393.2	215.3	307.7	308.4	266.4	312.5
32.5	203.8	372.0	401.9	219.5	313.1	314.6	271.4	320.1
33.0	207.9	380.3	410.6	223.7	318.6	320.9	276.4	327.8
33.5	212.1	388.5	419.3	227.9	324.1	327.1	281.3	335.4
34.0	216.3	396.7	428.0	232.1	329.6	333.4	286.3	343.5
34.5	220.4	404.9	436.7	236.3	335.1	339.6	291.3	352.4
35.0	224.6	413.1	445.4	240.5	340.5	345.9	296.2	361.2
35.5	228.8	421.3	454.1	244.7	346.0	352.1	301.2	370.0
36.0	233.0	429.5	462.8	248.9	351.5	358.4	306.2	378.9
36.5	237.2	437.8	471.5	253.2	357.0	364.6	311.1	387.7
37.0	241.3	446.0	480.2	257.4	362.5	370.9	316.1	396.6
37.5	245.5	454.2	488.9	261.8	367.9	377.1	321.1	405.5
38.0	249.7	462.4	497.6	265.8	373.4	383.4	326.1	414.3
38.5	253.9	470.6	506.4	270.9	378.9	389.6	331.0	423.2
39.0	258.1	478.9	515.1	277.7	384.4	395.9	336.0	432.1
39.5	262.3	487.1	523.8	284.6	389.9	402.1	341.0	440.9
40.0	266.5	495.3	532.5	291.4	395.4	408.4	346.0	449.8
40.5	270.7	503.5	541.2	298.3	400.8	414.6	350.9	458.7
41.0	274.9	511.8	549.9	305.1	406.3	420.9	355.9	467.6
41.5	279.1	520.0	558.6	312.0	412.5	427.1	360.9	476.4
42.0	283.3	528.2	567.3	318.9	420.6	433.4	365.9	485.3
42.5	287.5	536.4	576.1	325.8	428.7	439.6	370.8	494.2
43.0	291.7	544.7	584.8	332.6	436.8	445.9	375.8	503.1
43.5	295.9	552.9	593.5	339.5	444.9	452.1	380.8	512.0
44.0	300.1	561.1	602.2	346.4	453.0	458.4	385.8	520.9
44.5	304.3	569.3	610.9	353.3	461.1	464.6	390.8	529.8
45.0	308.5	557.6	619.6	360.2	469.2	470.9	395.7	538.7
45.5	312.7	585.8	628.4	367.0	477.3	477.1	400.7	547.6
46.0	316.9	594.0	637.1	373.9	485.4	483.4	405.7	556.5
46.5	321.1	602.3	645.8	380.8	493.6	489.6	410.7	565.4
47.0	325.3	610.5	654.5	387.7	501.7	495.9	415.7	574.3
47.5	329.5	618.7	663.2	394.6	509.8	502.1	420.6	583.3
48.0	333.7	626.9	672.0	401.5	517.9	508.4	425.6	592.2
48.5	337.9	635.2	680.7	408.4	526.0	514.6	430.6	601.1
49.0	342.2	643.4	689.4	415.3	534.2	520.9	435.6	610.0
49.5	346.4	651.6	698.1	422.2	542.3	527.1	440.6	618.9
50.0	350.6	659.9	706.9	429.1	550.4	533.4	445.6	627.8

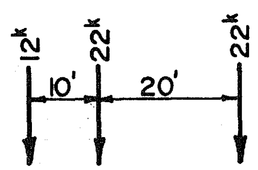
WHEEL LOAD DISTRIBUTION FACTORS (DF)		
Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for two or More Traffic Lanes
Timber Plank	s/4.0	s/3.75
Timber Strip 4" thick or multiple layer floors over 5"	s/4.5	s/4.0
Timber Strip 6" or more thick	s/5.0	s/4.25
Concrete T-Beams	s/6.5	s/6.0

where s = stringer or beam spacing in feet.

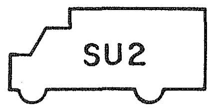
SU4 LOAD CASE:



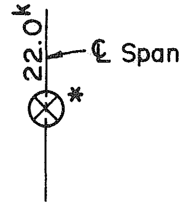
C3 LOAD CASE:



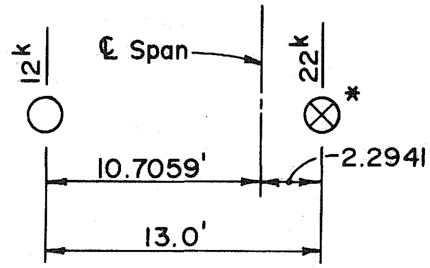
SINGLE UNIT - 2 AXLE (SU 2)



GVW = 34.0 Kips

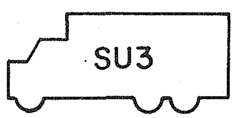


Spans less than 23.5'

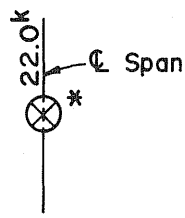


Spans 23.5' and greater

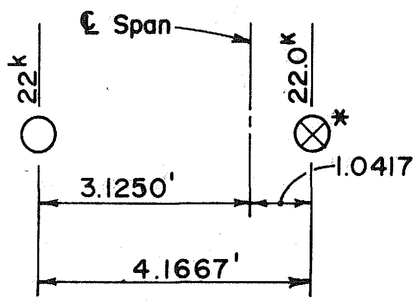
SINGLE UNIT - 3 AXLE (SU 3)



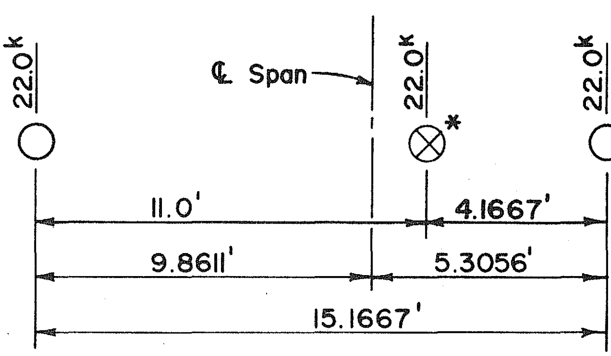
GVW = 66.0 Kips



Spans less than 7.1'



Spans 7.1' to 21.7'

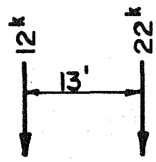


Spans 21.7' and greater

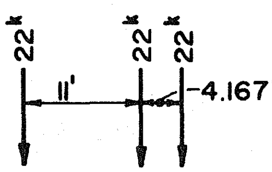
*Denotes the axle under which maximum bending moment occurs.

MAXIMUM FLORIDA LEGAL LOAD CASES AND AASHTO DESIGN LOAD CASES AND THEIR PLACEMENTS ON VARIOUS SIMPLE SPANS TO YIELD MAXIMUM MOMENT

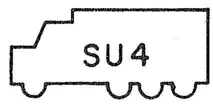
SU2 LOAD CASE:



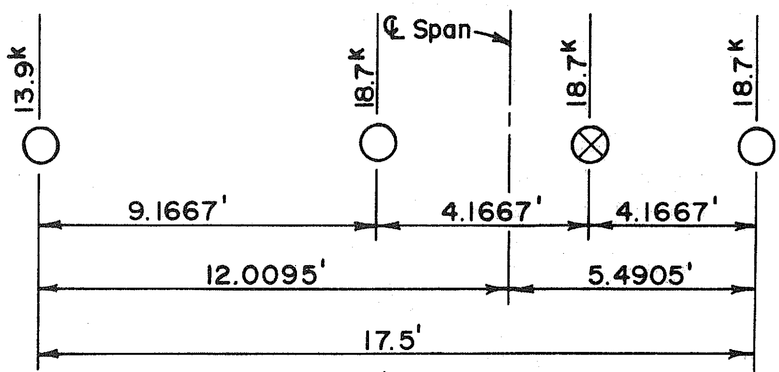
SU3 LOAD CASE:



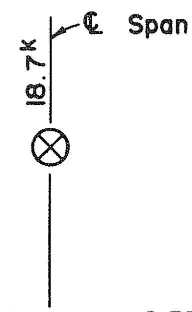
SINGLE UNIT - 4 AXLE (SU4)



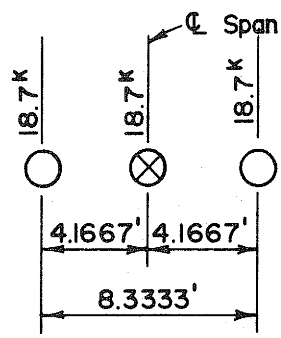
GVW = 70.0 Kips



Spans 25.25' and greater

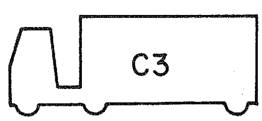


Spans less than 8.33'

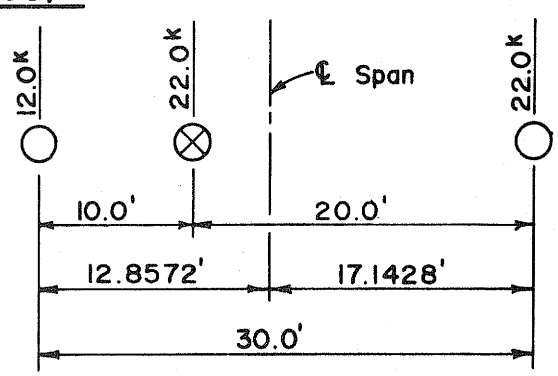


Spans 8.33' to 25.25'

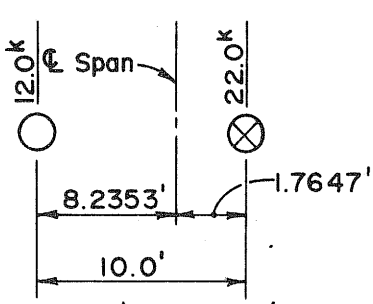
COMBINATION - 3 AXLE (C3)



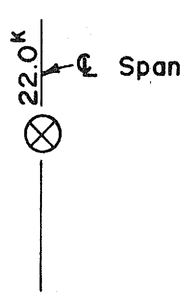
GVW = 56.0 Kips



Spans 38.33' and greater

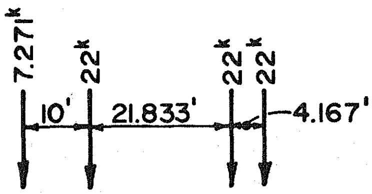


Spans 18' to 38.33'



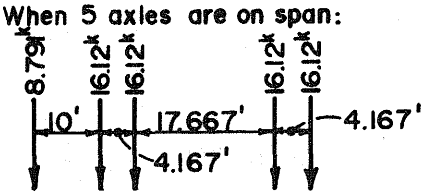
Spans less than 18'

C4 LOAD CASE:



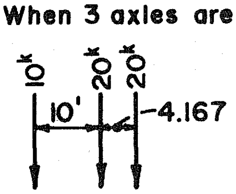
C5 LOAD CASE:

When 5 axles are on span:



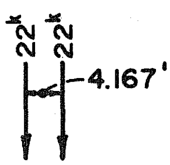
GVW = 73.271 Kips

When 3 axles are on span(Tractor axles only on span):



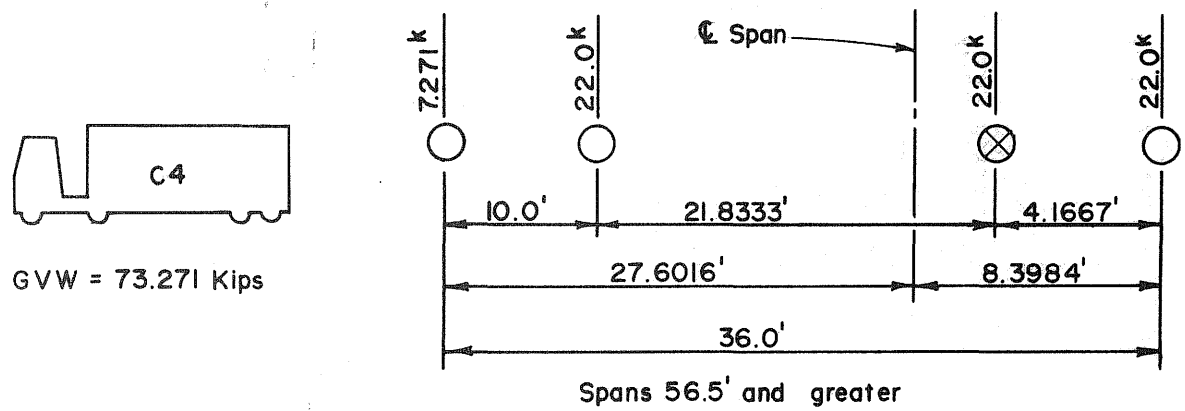
GVW = 80.0 Kips

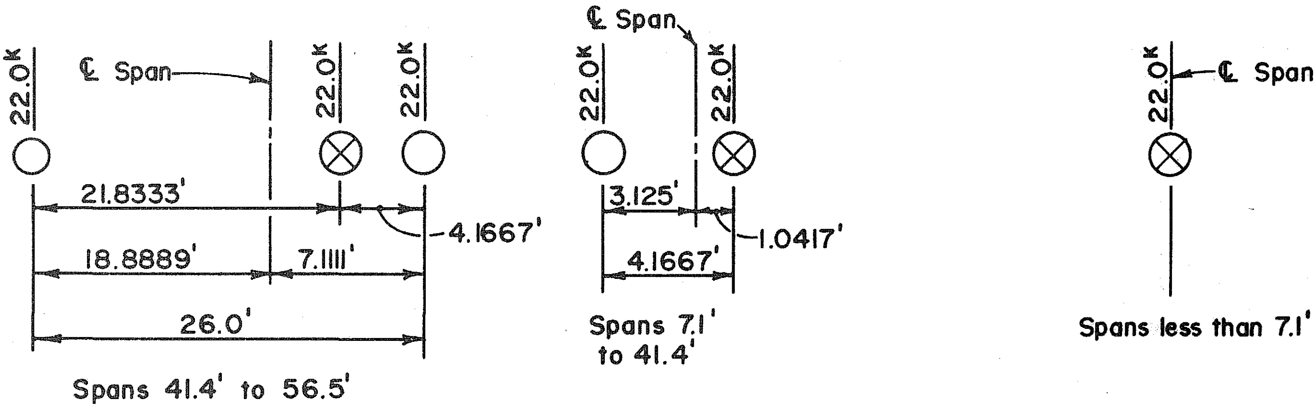
When 2 axles are on span (Trailer axles only on span):



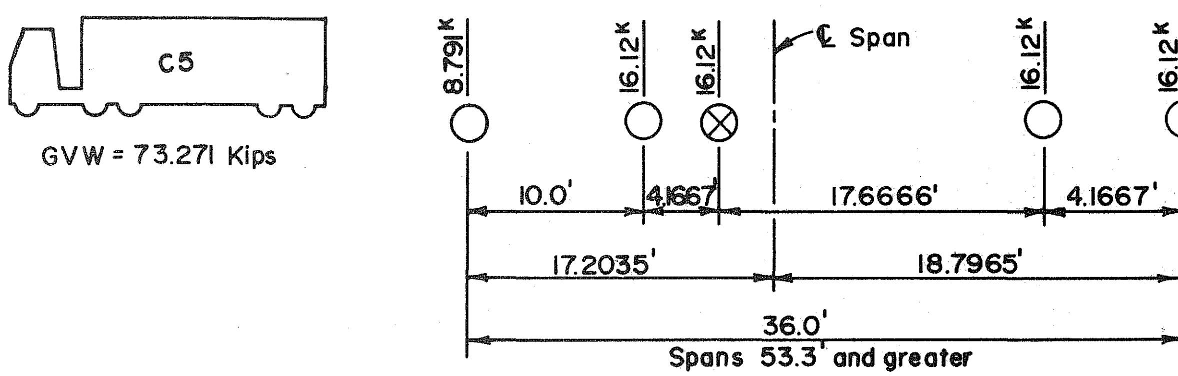
GVW = 80.0 Kips

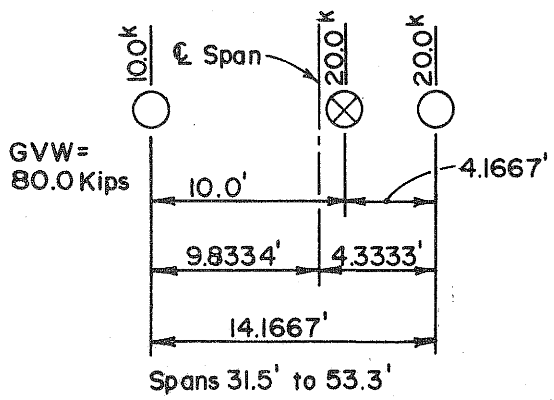
COMBINATION- 4 AXLE (C4)

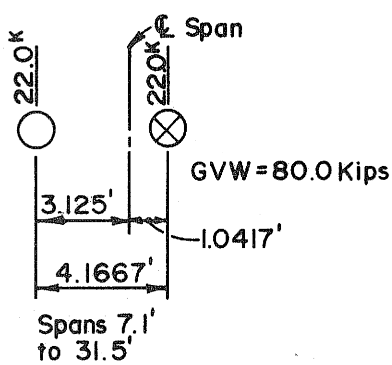


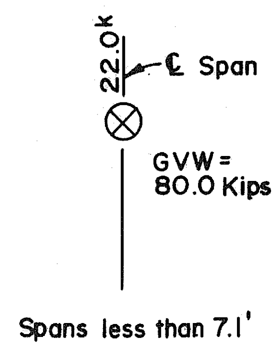


COMBINATION - 5 AXLE (C5)

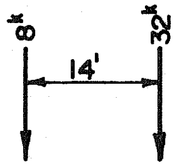








H LOADING:



HS LOADING:

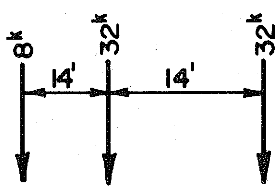
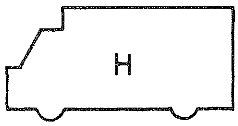
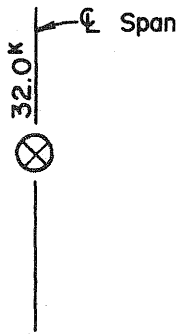


PLATE III
(Continued)

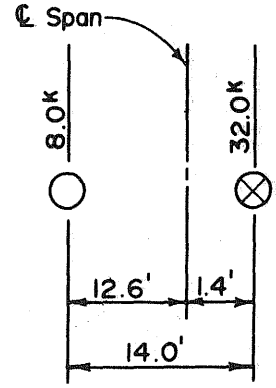
H LOADING (H)



GVW = 40 Kips

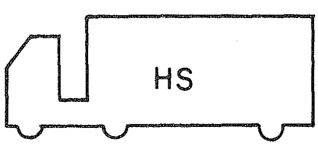


Spans less than 26.5'

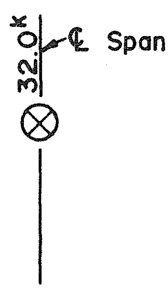


Spans 26.5' and greater

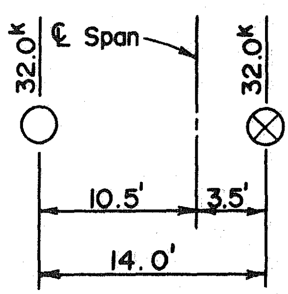
HS LOADING (HS)



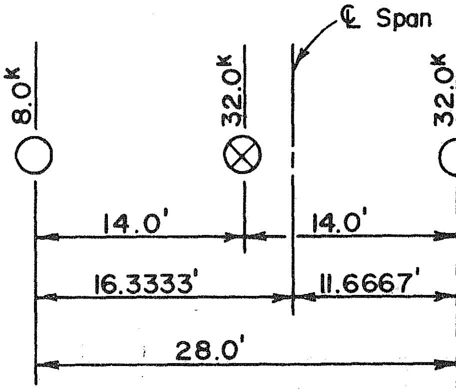
GVW = 72 Kips



Spans less than 23.9'



Spans 23.9' to 33.8'



Spans 33.8' and greater

Plate IV

DATA ON STANDARD DEFORMED REINFORCING BARS

Obsolete Bar Designation (size in in.)		Bar Designation		Diameter (in.)	Cross-Sectional area (A_b) (in^2)
Round Bars	1/4	Round Bar	# 2*	0.250	0.05
	3/8		# 3	0.375	0.11
	1/2		# 4	0.500	0.20
	5/8		# 5	0.625	0.31
	3/4		# 6	0.750	0.44
	7/8		# 7	0.875	0.60
	1		# 8	1.000	0.79
Square Bars	1		# 9	1.128	1.00
	1-1/8		#10	1.270	1.27
	1-1/4		#11	1.410	1.56

*#2 in plain round bars only.

DATA ON STANDARD SEVEN-WIRE UNCOATED STRESS-RELIEVED PRESTRESSING STRANDS

	3/8 in. dia.		7/16 in. dia.		1/2 in. dia.	
Property	ASTM Grade	Type 270K	ASTM Grade	Type 270K	ASTM Grade	Type 270K
Area (sq.in.)	0.0799	0.0854	0.1089	0.1167	0.1438	0.1531
Minimum Ultimate Strength	20,000 lb.	23,000 lb.	27,000 lb.	31,000 lb.	36,000 lb.	41,300 lb.
Initial Tension (70% ult)	14,000 lb.	16,100 lb.	18,900 lb.	21,700 lb.	25,200 lb.	28,910 lb.

DECIMALS OF A FOOT FOR EACH 1/8th OF AN INCH						
INCH	0	1	2	3	4	5
0	0	.0833	.1667	.2500	.3333	.4167
1/8	.0104	.0938	.1771	.2604	.3438	.4271
1/4	.0208	.1042	.1875	.2708	.3542	.4375
3/8	.0313	.1146	.1979	.2812	.3646	.4479
1/2	.0417	.1250	.2083	.2917	.3750	.4583
5/8	.0521	.1354	.2188	.3021	.3854	.4688
3/4	.0625	.1458	.2292	.3125	.3958	.4792
7/8	.0729	.1563	.2396	.3229	.4063	.4896

INCH	6	7	8	9	10	11
0	.5000	.5833	.6667	.7500	.8333	.9167
1/8	.5104	.5938	.6771	.7604	.8438	.9271
1/4	.5208	.6042	.6875	.7708	.8542	.9375
3/8	.5313	.6146	.6979	.7813	.8646	.9479
1/2	.5417	.6250	.7083	.7917	.8750	.9583
5/8	.5521	.6354	.7188	.8021	.8854	.9688
3/4	.5625	.6458	.7292	.8125	.8958	.9792
7/8	.5729	.6563	.7396	.8229	.9036	.9896

DECIMALS OF AN INCH FOR EACH 1/8TH OF AN INCH			
FRACTION	DECIMAL	FRACTION	DECIMAL
1/8	.125	5/8	.625
1/4	.250	3/4	.750
3/8	.375	7/8	.875
1/2	.500	1	1.000

APPENDIX B

11.1 Notation

11.2 Glossary of Terms

SECTION 11.1 - NOTATION

AASHTO	American Association of State Highway and Transportation Officials
A_b	Cross Sectional area of a reinforcing bar (in.^{2}). (Sec. 5.3.4, 6.3.4, 6.3.5.
As	Cross-sectional area of reinforcing steel (in.^{2})(Sec. 5.3.4, 6.3.4, 6.3.5, 6.3.6.
A^*_s	Area of prestressing Steel (Section 7.3.5).
As_b	Area of reinforcement required for balanced condition of strain in ultimate strength design (in.^{2})(Sec. 5.3.4, 6.3.4, 6.3.5).
A_{sf}	Area of reinforcement to develop compressive strength of overhanging flanges of a T-Beam Section (in.^{2})(Sec. 6.3.5).
A_v	Cross-sectional area of each leg of a stirrup (in.^{2}) (Sec. 6.3.6, 7.3.6, 7.3.7).
b	Width of the compression face of a member (in.) (Section 7.3.5).
c	Distance from the neutral axis of a member in flexure to the extreme compressive fiber. (in.)(Sec. 5.3.4, 6.3.4, 6.3.5).
c_L	Centerline.
d	Distance from the extreme compression fiber to the centroid of tension reinforcement (in.).
DF	Distribution Factor. The fraction of a wheel load that is to be applied to the stringer or beam (Sec. 4.3.4, 4.3.6, 5.3.4, 6.3.5, 7.3.4).
DL	Dead Load. Loads due to the weight of the components making up the structure.
e	Eccentricity of prestressing force. Distance from centroid of prestressing force to centroid of channel (in.) (Sec. 7.3.4).
f_i	Initial Prestressing Force (psi) (Sec. 7.3.4).
f_{bi}	Allowable bending stress for timber at inventory rating (psi.)(Sec. 2.2, 4.3.3, 4.3.4, 4.3.6).
f_{bo}	Allowable bending stress for timber at operating rating (psi.)(Sec. 2.2, 4.3.3, 4.3.4, 4.3.6)
f'c	Specified minimum compressive strength of the type of concrete being used. (psi)(Sec. 2.3, 2.4, 2.5)

FDOT	Florida Department of Transportation
f's	Ultimate strength of prestressing steel (Section 7.3.5).
f^*_{su}	Average stress in prestressing steel at ultimate load (Section 7.3.5, 7.3.6, 7.3.7
f_{vi}	Allowable horizontal shear stress at inventory rating (psi.) (Sec. 2.2, 4.3.5, 4.3.7).
f_{vo}	Allowable horizontal shear stress at operating rating (psi.) (Sec. 2.2, 4.5, 4.7).
fy	Specified minimum yield point or yield strength of the type of steel being used. (psi)(Sec. 2.3, 2.4, 2.5)
GVW	Gross Vehicle Weight. (kips)
I	Impact factor (Section 5.3.4, 5.3.5, 6.3.5, 6.3.6, 7.3.4, 7.3.6).
I_o or I_{na}	Moment of Inertia of a geometric shape about its neutral axis(in.^{4})(Sec. 4.4, 4.6, 7.4).
j	Ratio of the distance between the centroid of the compressive forces and the centroid of the tensile forces to the depth (d) in a flexural member. (Section 7.3.6, 7.3.7).
LLM	Applied live load moment (ft-kips)(Sec. 3.2)
LLM'	That part of M'u resulting from live load (ft-lbs)(Sec. 5.3.5, 6.3.6).
LLV	Applied live load shear (kips)(Sec. 3.3)
$M_a i$	Available live load moment capacity at Inventory Level (Section 7.3.4).
$M_a o$	Available live load moment capacity at Operating Level (Section 7.3.5).
M_{DL}	Applied Dead Load Moment (ft.-lbs. or ft.-kips)(Sec. 4.3.4, 4.3.6, 5.3.4, 6.3.4, 6.3.5, 7.3.3).
M'_{DL}	That part of M'u resulting from dead load (ft-lbs)(Sec. 5.3.5, 6.3.6).
M_{LLi}	Available live load moment capacity at Inventory Rating (ft.-lbs)(Sec. 4.3.3, 4.3.4, 4.3.6).

M_{LLo}	Available live load moment capacity at Operating Rating (ft.-lbs)(Sec. 4.3.3, 4.3.4, 4.3.6).
M_{Ri}	Resisting Moment of a member at Inventory Rating (ft.-lbs)(Sec. 4.3.3).
M_{Ro}	Resisting Moment of a member at Operating Rating (ft.-lbs)(Sec. 4.3.3).
M'u	The live load plus dead load moment applied in conjuction with maximum applied live load plus dead load shear. (ft-lbs)(Sec. 5.3.5, 6.3.6).
Mu	Ultimate moment capacity of a member (ft-kips) (Sec. 7.3.5).
N_L	Number of design traffic lanes (Sec. 7.3.4).
p*	A*s/bd, ratio of prestressing steel (Sec. 7.3.5, 7.3.6, 7.3.7).
pcf	Pounds per cubic foot
Pi	Available live load wheel capacity at Inventory Rating (kips)(Sec. 4.3.3).
P_i	Prestressing load (lbs.) (Sec. 7.3.4).
P_{ie_1}	Prestress moment at end of channel section (in. lbs.).
P_{ie_2}	Prestress moment of midspan (in-lbs) (Sec. 7.3.4).
Po	Available live load wheel capacity at Operating Rating (kips)(Sec. 4.3.3).
psf	Pounds per square foot.
psi	Pound per square inch
RF	Rating Factor. The ratio of the load carrying capacity of a member to the applied load. RF shall not be taken as greater than 1.0.
S_B	Section modulus at the bottom of a channel section (in. (Sec. 7.3.4).
S_T	Section modulus at the top of a channel section (in.$_3$) (Sec. 7.3.4).
T_d	Thickness of cast in place concrete deck surface for a channel section.
T_f	Flange thickness of a channel section.

v_c — Allowable shear stress in concrete (psi.) (Sec. 5.3.5, 6.3.6, 7.3.6, 7.3.7).

V_c — Shear carried by concrete (kips) (Section 6.3.6, 7.3.6, 7.3.7).

V_{DL} — Applied Dead Load Shear (lbs. or kips)(Sec. 4.3.5, 4.3.7, 5.3.5, 6.3.6, 7.3.6, 7.3.7).

V_{LLi} — Available horizontal shear capacity for live load at inventory rating (lbs)(Sec. 4.3.5, 4.3.7).

V_{LLo} — Available horizontal shear capacity for live load at operating rating (lbs)(Sec. 4.3.5, 4.3.7).

VR_o — Resisting horizontal shear for operating rating (lbs) (Sec. 4.3.5, 4.3.7).

VR_i — Resisting horizontal shear for inventory rating (lbs.) (Sec. 4.3.5).

V_s — Shear carried by the stirrups (kips) (Section 6.3.6, 7.3.6).

Vu — Ultimate shear capacity of a member (kips) (Sec. 5.3.3, 6.3.6, 7.3.6, 7.3.7).

W_b — Bottom width of the web for a channel section (in.).

W_c — Nominal width of a channel section (in.).

W_d — Web depth for a channel section (in.).

W_{DL} — Uniform Dead Load weight along a member (lb/ft.)(Sec. 4.3.4, 4.3.5, 4.3.6, 4.3.7, 5.3.3, 6.3.3, 6.3.6, 7.3.3).

W_F — Effective width of timber floor to be considered to support wheel loads.(Sec. 4.3.3).

W_i — Available live load capacity per foot at inventory rating (lb/ft.)(Sec. 4.3.4, 4.3.5).

W_o — Available live load capacity per foot at operating rating (lb/ft.)(Sec. 4.3.4, 4.3.5).

WS_{DL} — Dead Load per one foot strip of slab (lb/ft) (Sec. 6.3.3)

W_t — Top width of the web for a channel section (in.).

x — Distance from the support to the nearest axle load as defined in Sec. 3.3 (ft.).

$\overline{Y}$ — Distance from the centroid of a segment of a member to the centroid of the member. (Sec. 4.3.4, 4.3.6, 7.3.4).

Y_b Distance from the bottom of a prestressed channel to the neutral axis of a particular segment of the channel (in.) (Sec. 7.3.4).

Y_{na} Distance from the bottom of a member to its neutral axis. (in.)(Sec. 4.3.4, 4.3.6), 7.3.4).

Z Section Modulus (in.^{3})(Sec. 4.3.3, 4.3.4, 4.3.6).

α The angle describing the inclination of the stirrups with the longitudinal axis of the member (Sec. 6.3.6).

β A factor describing the stress curve of a concrete flexural member as a function of Kd. (Sec. 5.3.4, 6.3.4, 6.3.5).

σ_c Allowable concrete stress in compression for a prestressed member (psi.) (Sec. 7.3.4).

σ_t Allowable concrete stress in tension for a prestressed member (psi.) (Sec. 7.3.4).

σ_1 Concrete stress at the top of the member at midspan due to applied prestress forces plus dead load. (psi.) (Sec. 7.3.4).

σ_2 Concrete stress at the bottom of the member at midspan due to applied prestress forces plus dead load (psi) (Sec. 7.3.4).

σ_3 Concrete stress at the top of the member at ends due to applied prestress forces (psi) (Sec. 7.3.4).

σ_4 Concrete stress at the bottom of the member at ends due to applied prestress forces (psi) (Sec. 7.3.4).

SECTION 11.2 - GLOSSARY OF TERMS

Abutment - A substructure composed of stone, concrete, brick, or timber supporting the end of a single span or the extreme end of a multispan superstructure and, in general, retaining or supporting the approach embankment placed in contact therewith.

Allowable Unit Stress - As applied to the investigation of an existing structure in determining its adequacy for existing or prospective service; it is the stress per unit area of the material of the entire structure or any portion or member thereof which is determined to be a safe unit for service use, due consideration being given to the quality of the material, physical condition, the adequacy of the construction details or other physical factors incident or pertinent to the service conditions to which they are or will be subjected and, if necessary, to the conditions contemplated to exist in the event of repair, replacement or strengthening operations.

Applied Live Load Moment - Moment resulting from the weight of the live loads (trucks).

Applied Live Load Shear - Shear force resulting from the weight of the live loads (trucks).

Bearing or Bearing Area - The region of the stringer, pour-in-place slab, channel flange, or T-Beam flange where the transmittal of the loads from the superstructure to substructure occurs.

Bridge Inspection Data Report - The report prepared by the bridge inspector, recording the condition and physical dimensions of the structure.

Cap - The topmost piece or member of a pile bent serving to distribute the loads uppon the piles and to hold them in their proper relative positions.

Centroid - The geometric center of an object.

Clear Span - The unobstructed space or distance between the substructure elements measured, by common practice, between faces of abutments and/or piers.

Cracking - Deterioration of concrete by a linear fracture of the concrete mass.

Continuous Span - Span that is continuous over intermediate supports.

Dead Load - Dead Load consists of the weight of the members comprising the structure.

Denominator - The part of a fractional equation below the line.

Deck - That portion of a bridge which provides direct support for vehicular and pedestrian traffic. The deck may be either a reinforced concrete slab or timber flooring. While normally distributing load to a system of beams or stringers, a deck may also be the main supporting member of a bridge, as with the reinforced concrete slab type superstructure.

Design Loading - The loading comprising magnitudes and distributions of wheel, axle or other concentrations used in the determination of the stresses, stress distributions and ultimately the cross sectional areas and compositions of the various portions of the bridge structure. The design loading is fixed by specifications and is very commonly composite rather than actual, but is predicated upon a study of various vehicle types. In rating bridges in this manual H and HS trucks are used in the design loading cases.

Distribution Factor - A numerical factor applied to the live load for the purpose of distributing the wheel loads (longitudinally or laterally) to the bridge members.

Flange - The portion of a beam which carries the compressive and tensile forces that comprise the internal resisting moment of the beam. The flange extends transversely across the top and bottom edges of the web.

Floor or Flooring - See Deck.

Horizontal Shear - Shear along the axis of the member in the horizontal plans. In particular, this term applies to timber stringers where the shear strength along the grain is much less than the shear strength across the grain.

Impact - A dynamic increment of stress equivalent in magnitude to the difference between the stresses produced by a static load when at rest and by a load moving in a straight line.

Impact Factor - A numerical factor applied to the live load for the purpose of providing for the effects of impact.

Impact Load - A load allowance or increment intended to provide for the dynamic effect of a load applied in a manner other than statically.

Inventory Rating - The load level that can safely utilize a structure an indefinite period of time, based on the load rating calculations.

Kip - A unit of force equal to 1,000 pounds.

Live Load - A dynamic load such as traffic load that is supplied to a structure suddenly of that is accompanied by vibration, oscillation or other physical condition affecting its intensity.

Load Case - Representation of a truck that uses Florida highways. A load case is a particular set of axle loads and axle spacings designed to produce the greatest stresses in the bridge members for that given number of axles.

Longitudinal Reinforcement - Reinforcement placed parallel with the centerline of a bridge.

Loss in Section - A decrease in the cross-sectional area of a member due to deterioration or damage.

Moment - The product of distance and force (or load) about a point.

Moment of Inertia - A geometric property of an object defined as the integral over the beam section of each element of area multiplied by the square of its distance from the neutral axis.

Neutral Axis - The axis of a member in bending along which the strain is zero. On one side of the neutral axis the fibers are in tension, on the other side in compression.

Numerator - The part of a fractional equation above the line.

Operating Rating - The absolute maximum permissible load level to which a structure may be subjected, based on the load rating calculations.

Reinforced Concrete Beam - A member in which the tensile stresses, whether resulting from bending, shear, or combinations thereof produced by transverse loading, are by design carried by the steel reinforcement. The concrete takes compression (and some shear) only.

Reinforcing Bar - A steel bar, smooth or with a deformed surface, which bonds to the concrete and supplies tensile strength to the concrete.

Safe Load - The maximum loading determined by a consideration of its magnitudes and distributions of wheel and axle loads as productive of unit stresses in the various members permissible for service use, due consideration being given to the physical condition of the structure resulting from its previous service use.

Scaling - Deterioration of concrete by the gradual and continuing loss of surface mortar and aggregate over a widespread area.

Section Modulus - The ratio of the moment of inertia of a member to the distance between the neutral axis and the outer compressive fiber of the member.

Shear - The result of transverse loads or forces which tend to separate one part of the member from the other along a lateral plane. Shear is a scissor-like action.

Simple Span - Span from end support to end support, with no intermediate supports.

Skew Angle - As applied to oblique bridges; the skew angle, angle of skew, or simply "skew" is the acute angle subtended by a line normal to the longitudinal axis of the structure and a line parallel to or coinciding with the alignment of its end.

Spalling - Deterioration of concrete by a localized separation and removal of a portion of the surface of the concrete revealing a fracture roughly parallel, or slightly inclined, to the concrete surface. Usually, a portion of the depression rim is perpendicular to the surface and often reinforcing steel is exposed.

Span - The distance center to center of the end bearings or the distance between the lines of action of the reactions.

Stirrup - Name given to reinforcement placed perpendicular to, or at some angle of inclination with, the longitudinal axis of a member. Stirrups serve the purpose of providing reinforcement against diagonal tension (shear).

Strain - The distortion of a body produced by the application of one or more external forces and measured in units of length per unit length. In common usage, this is the proportional relation of the amount of distortion divided by the original length.

Stress - The resistance of a body to distortion when in a solid or plastic state and when acting in an unconfined condition. Stress is produced by the strain (distortion) and holds in equilibrium the external forces causing the distortion. Stress is measured in pounds or kips per unit area. Within the elastic limit the strain in a member of a structure is proportional to the stress in that member.

Stringer - A longitudinal beam supporting the brdige deck.

Substructure - The abutments, bents, piers, or other constructions built up to support the span or spans of the superstructure.

Superstructure - The entire portion of a bridge structure which primarily receives and supports the highway traffic loads and in turn transmits them to the substructure. That portion of the bridge which spans between supports.

Transverse Reinforcement - Reinforcement placed perpendicular with the centerline of a bridge.

Wearing Surface - (Wearing Course) A topmost layer or course of material applied upon a roadway to receive the traffic service loads and to resist the abrading, crushing or other disintegrating action resulting therefrom.

Web - The portion of a beam located between and connected to the flange. It serves mainly to resist shear stress.

Wheel Guard - A timber piece placed longitudinally along the side limit of the roadway to guide the movements of vehicle wheels and safeguard the bridge railings and other constructions existing outside the roadway limit from collision with vehicles and their loads.

APPENDIX C

12.1 Timber Deck and Stringer Superstructure Rating Forms

12.2 Reinforced Concrete Slab Superstructure Rating Forms

12.3 Reinforced Concrete T-Beam Superstructure Rating Forms

12.4 Prestressed Concrete Channel and Slab System Superstructure Rating Forms

TIMBER DECK AND STRINGER SUPERSTRUCTURE

RATING FORMS

SECTION 12.1 - TIMBER DECK AND STRINGER SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE:

1. These sheets should be completed in their entirety before continuing with the rating computations.
2. Sketches of the bridge superstructure under consideration should be made and dimensions, spacings, and direction of inspection should be shown thereon. These sketches should include a section and plan similar to Figures 2-1 and 2-2.

By: ______________________________ Date: __________

Bridge Name: ______________________________

Bridge No.: __________ Road No.: __________ County: __________

Bridge Length = __________ ft. Number of Spans = __________

Span Length ($\mathrm{C_L}$ support to $\mathrm{C_L}$) = ________ ft. (if different list each)

Overall Bridge Width (out-to-out) = ______________ ft.

Roadway Width (curb to curb) = ______________ ft.

Sidewalks:

LEFT SIDE (looking in direction of inspection):

Width = ____________ ft.

Rail Size = _________in. x ________ in.

Rail Post Size & Length = ________ in. x _______ in. x ________ ft.

Average Post Spacing = ___________ ft.

RIGHT SIDE (looking in direction of inspection):

Width = ______________ ft.

Rail Size = _________in. x ________ in.

Rail Post Size & Length = ________ in. x _______ in. x ________ ft.

Average Post Spacing = ____________ ft.

Barrier Rails: Rail Size = ________ in. x _______ in.

Rail Post Size & Length = _______ in. x _______ in. x _______ ft.

Average Post Spacing = ____________ ft.

Wheel Guard: Width = _____________ in.

Height = _____________ in.

Wearing Surface:

asphalt; thickness = _____________ in.

concrete; thickness = _____________ in.

Deck:

Type; Strip (tongue & groove or dowled) or Plank

Continuous over three or more stringers/or simple span between two stringers

thickness = ________ in. (average thickness not considering deterioration)

thickness = ________ in. (average thickness taking into account deterioration.)

Width of Strip or Plank = ___________ in.

STRINGER DATA:

Stringer Number	Stringer Cross-Sectional Dimensions (Height and Width) (inches)	Spacing Between Adjacent Stringers (ft.)

STRINGER DATA NOTES:

1. Deterioration of the stringer should be accounted for by using an average height x width of the sound material. If interior deterioration has occurred (the stringer is a "shell") the average exterior height x width along with the approximate interior height x width of deterioration and dimensions from center of deteriorated area to bottom of stringer should be recorded under "stringer cross-sectional dimensions".

2. The location of deteriorated sections as measured from the beginning of stringer (as defined by the direction of inspection) should be noted.

3. It should be specifically noted if a loss of stringer section has occurred near the end of the stringer. If no specific comments were noted in the Bridge Inspectors Report, it should be verified that the stringer has suffered no loss of section due to deterioration or crushing, as this condition will be significant in the horizontal shear rating of the stringer.

Material Strengths:

Allowable stress in bending (f_b) as determined by the Bridge

Engineer* = __________ psi.

Allowable stress in shear (f_v) as determined by the Bridge

Engineer* = __________ psi.

Allowable bending stress at inventory rating level (f_{bi})

$= f_b = \boxed{}$ psi.

Allowable bending stress at operating rating level (f_{bo})

$= 1.33\ f_b = \boxed{}$ psi.

Allowable shear stress at inventory rating level (f_{vi})

$= f_v = \boxed{}$ psi

Allowable shear stress at operating rating level (f_{vo})

$= 1.33\ f_v = \boxed{}$ psi.

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*Standard Specifications for Highway Bridges, Twelfth Edition, 1977; Interim Specifications Bridges 1978, 1979 and 1980; Section 1.10.1 and Table 1.10.1A will be used in determining the allowable stresses for timber members when, in the judgment of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the allowable stresses shall be fixed by a Registered Professional Engineer.

Compute Dead Load (DL) per foot of flooring:

Asphalt wearing surface = (thickness)(144 lb/ft^3)(1/12)
= (in.)(144 lb/ft^3)(1/12) = ______ lb/ft

Concrete wearing surface = (thickness)(150 lb/ft^3)(1/12)
= (in.)(150 lb/ft^3)(1/12) = ______ lb/ft

Timber floor = (thickness)(50 lb/ft^3)(1/12)
(in.)(50 lb/ft^3)(1/12) = ______ lb/ft

Total DL = $\boxed{}$ lb/ft

Determine Effective Deck Span(s):

s = Stringer Spacing (C_L to C_L)
s = __________ ft.

but not greater than:

(Stringer Spacing - Stringer Width)+(Floor thickness*)
(-   )ft. + (in. x 1/12) = ______ ft.

So, s = $\boxed{}$ ft.

*not including wearing surface

Determine Applied Dead Load Moment (M_{DL}):

M_{DL} = (Total DL)(s)^2(1/8)
M_{DL} = (lb/ft.)(ft.)^2(1/8)   = $\boxed{}$ ft-lbs

Determine distribution of Wheel Loads on Flooring Parallel to Bridge:

W_F = Effective Width of Floor to be considered to support wheel load

for Strip Floor W_F = 4 x (Floor Thickness*) but not less than 5-1/2 "

W_F = 4 x (   )in.   = ________ (in.)

for Plank Floor W_F = width of plank   = ________ (in.)

So, W_F = $\boxed{}$ (in.)

If W_F come out less than 5-1/2, use 5-1/2

Calculate Section Modulus (Z):

$$Z = \frac{W_F \times (T_F)^2}{6}$$

$$Z = \frac{(\quad in)(\quad in)^2}{6} = \boxed{} \text{ in.}^3$$

* Not including wearing surface

Calculate Resisting Moments:

Inventory Resisting Moment (MR_i) = $\frac{f_{bi} \times Z}{12}$ x (continuity factor)*

$= \frac{(\quad lb/in^2)(\quad in^3)}{12} (\quad) = \boxed{\quad}$ ft-lbs.

Operating Resisting Moments (MR_o) = $\frac{f_{bo} \times Z}{12}$ x (continuity factor)*

$= \frac{(\quad lb/in^2)(\quad in^3)}{12} (\quad) = \boxed{\quad}$ ft-lbs.

*Note: If the flooring is continuous over stringers (i.e., the floor timbers span 3 or more stringers) the continuity factor is 1.25. If the flooring is not continuous over stringers (i.e., the floor timbers span between 2 stringers only) the continuity factor is 1.0.

Determine Resisting Moments Available for Live Load:

Available Live Load Inventory Moment

(MLLi) = (MR_i) - (M_{DL})
(MLLi) = ( ) - ( ) = $\boxed{\quad}$ ft-lbs.

Available Live Load Operating Moment

(MLLo) = (MR_o) - (M_{DL})
(MLLo) = ( ) - ( ) = $\boxed{\quad}$ ft-lbs.

Determine Available Live Load per Wheel (P):

For Inventory Rating: $P_i = \frac{4 \text{ x } (\text{MLLi})}{(s)(1000)}$

$$P_i = \frac{(4)(\quad\quad)}{(\quad\quad)(1000)} = \boxed{\quad\quad} \text{ Kips}$$

For Operating Rating: $P_o = \frac{4 \text{ x } (\text{MLLo})}{(s)(1000)}$

$$P_o = \frac{(4)(\quad\quad)}{(\quad\quad)(1000)} = \boxed{\quad\quad} \text{ Kips}$$

Calculate Inventory and Operating Ratings:

The formula to be used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{\text{Pi}}{1/2(\text{max. axle load})}(\text{GVW})$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{\text{Po}}{1/2\ (\text{Max. axle load})}(\text{GVW})$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 3.2 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Floor Rating".

SECTION 12.1 - MOMENT CAPACITY RATING OF AN EXTERIOR STRINGER

Compute Dead Load DL per foot of Stringer:

Rail: (Number of Rails)(Width)(Height)($50\#/ft^3$)(1/144)

= (   )(in.)(in.)($50\#/ft^3$)(1/144) =

__________lb/ft.

Rail Posts: (Width)(Height)(Length)($\frac{1}{\text{Post Spacing}}$)($50\#/ft^3$)(1/144)

= (in.)(in.)(ft.)(1/   ft.)($50\#/ft.^3$)(1/144) =

__________lb/ft.

Wheel Guard: (Width)(Height)(50#/ft(1/144)

= (in.)(in.)($50\#/ft^3$)(1/144) =

__________lb/ft.

s = 1/2 (spacing between exterior stringer & first interior stringer, $\mathrm{C_L}$ to $\mathrm{C_L}$)

s = __________ ft.

Wearing Surface:

asphalt: (Thickness)(S)($144/ft^3$)(1/12)
(in.)(ft.)($144/ft^3$)(1/12) = __________lb/ft.

concrete: (Thickness)(S)($150\#/ft^3$)(1/12)
(in.)(ft.)($150\#/ft^3$)(1/12) = __________lb/ft.

Timber Deck: (Thickness)(S)($50\#/ft^3$)(/12)
(in.)(ft.)($50\#/ft^3$)(1/12) = __________lb/ft.

Stringer: (Width)(Height)($50\#/ft^3$)(1/144)
(in.)(in.)($50\#/ft^3$)(1/144) = __________lb/ft.

Total DL per foot of stringer (W_{DL}) = $\boxed{}$ lb/ft.

Calculate Applied Dead Load Moment (M_{DL})

M_{DL} = (1/8)(W_{DL})(Span Length)^2

= (1/8)(lb/ft.)(ft.)^2 = $\boxed{}$ ft.lb.

Compute Section Modulus (Z) of Exterior Stringer:

Due to the nature of deterioration in timber members, two conditions may exist: deterioration of the surfaces of the member, and internal deterioration. The computation of the section modulus is different for the two conditions; therefore, the deterioration condition should be determined and the section modulus calculated using the appropriate case below. Where no interior deterioration is present, use Case I.

CASE I: No interior deterioration. Height and width are measurements of the "sound" material, taking into account deterioration. The section modulus for the top of the stringer equals the section modulus for the bottom of the stringer.(ie. $Z_{top} = Z_{bottom} = Z$).

$$Z = \frac{(\text{Width})(\text{Height})^2}{6}$$

$$Z = \frac{(\quad \text{in.})(\quad \text{in.})^2}{6} = \boxed{} \text{ in.}^3$$

CASE II: Interior deterioration is present. Void size is to be approximated and represented by a void height, void width, and the distance from the center of the void to the bottom of the stringer. Section modulus will be calculated for the "shell" of sound material.

$$Y_{na} = \frac{(1/2)(\text{Width})(\text{Height})^2 - (\text{Void Width})(\text{Void Height})(\text{Dist. bot. stringer to } ℄ \text{ Void})}{(\text{Width})(\text{Height}) - (\text{Void Width})(\text{Void Height})}$$

$$Y_{na} = \frac{(1/2)(\quad \text{in.})(\quad \text{in.})^2 - (\quad \text{in.})(\quad \text{in.})(\quad \text{in.})}{(\quad \text{in.})(\quad \text{in.}) - (\quad \text{in.})(\quad \text{in.})}$$

Y_{na} = ______________ in.

CASE II (Continued)

$$I_o \text{ Stringer} = \frac{(\text{Width})(\text{Height})^3}{12}$$

$$= \frac{(\quad \text{in.})(\quad \text{in.})^3}{12} = \underline{\qquad\qquad} \text{in.}^4$$

$$I_o \text{ Void} = \frac{(\text{Void Width})(\text{Void Height})^3}{12}$$

$$= \frac{(\quad \text{in.})(\quad \text{in.})^3}{12} = \underline{\qquad\qquad} \text{in.}^4$$

$$\bar{Y} \text{ Stringer} = Y_{na} - 1/2\ (\text{height})$$

$$= (\quad \text{in.}) - 1/2\ (\quad \text{in.}) = + \underline{\qquad} \text{in.*}$$

$$\bar{Y} \text{ Void} = Y_{na} - (\text{dist. bot. stringer to } ℄ \text{ Void})$$

$$= (\quad \text{in.}) - (\quad \text{in.}) = + \underline{\qquad} \text{in.*}$$

$$I_{na} = (I_o \text{ Stringer}) - (I_o \text{ Void}) + (\text{Width})(\text{Height})(\bar{Y} \text{ Stringer})^2$$
$$- (\text{Void Width})(\text{Void Height})(\bar{Y} \text{ Void})^2$$

$$= (\quad \text{in}^4) - (\quad \text{in.}^4) + (\quad \text{in.})(\quad \text{in.})(\quad \text{in.})^2$$
$$- (\quad \text{in.})(\quad \text{in.})(\quad \text{in.})^2 = \underline{\qquad\qquad} \text{in.}^4$$

---

* If negative, the negative sign should be dropped. Both values should be positive.

CASE II (Continued)

$$Z_{bottom} = \frac{I_{na}}{Y_{na}} = \frac{\quad in.^4}{\quad in.} = \boxed{\qquad\qquad} \text{ in.}^3$$

$$Z_{top} = \frac{I_{na}}{(\text{Height} - Y_{na})} = \frac{\quad in.^4}{(\quad in. - \quad in.)} = \boxed{\qquad\qquad} \text{ in.}^3$$

Calculate Resisting Moments:

CASE I: No interior deterioration.

Inventory Resisting Moment $(MR_i) = \frac{(f_{bi})(Z)}{12}$

$$= \frac{(\quad psi)(\quad in^3)}{12} = \boxed{\qquad\qquad} \text{ ft-lbs.}$$

Operating Resisting Moment $(MR_o) = \frac{(f_{bo})(Z)}{12}$

$$= \frac{(\quad psi)(\quad in^3)}{12} = \boxed{\qquad\qquad} \text{ ft-lbs.}$$

CASE II: Interior deterioration is present.

Inventory Resisting Moment (MR_i) =

$$\text{smaller of } \frac{(f_{bi})(Z_{top})}{12} \text{ or } \frac{(f_{bi})(Z_{bottom})}{12}$$

$$= \frac{(\quad psi)(\quad in^3)}{12} = \boxed{\qquad\qquad} \text{ ft-lbs.}$$

Use $\boxed{\qquad\qquad}$ ft-lbs.

$$= \frac{(\quad psi)(\quad in^3)}{12} = \boxed{\qquad\qquad} \text{ ft-lbs.}$$

Operating Resisting Moment (MR_o) =

smaller of $\frac{(f_{bo})(Z_{top})}{12}$ or $\frac{(f_{bo})(Z_{bottom})}{12}$

$= \frac{(\quad lb/in^2)(\quad in.^3)}{12} =$ ________ ft-lbs.

use $\boxed{\quad\quad}$ ft-lbs

$= \frac{(\quad lb/in^2)(\quad in.^3)}{12} =$ ________ ft-lbs.

Determine Resisting Moments Available for Live Load:

Available Capacity for Live Load Inventory Moment

$(MLL_i) = (MR_i) - (M_{DL})$

$(MLL_i) = (\quad\quad) - (\quad\quad) = \boxed{\quad\quad}$ ft-lbs.

Available Capacity for Live Load Operating Moment

$(MLL_o) = (MR_o) - (M_{DL})$

$(MLL_o) = (\quad\quad) - (\quad\quad) = \boxed{\quad\quad}$ ft-lbs.

Calculate Inventory and Operating Moment Ratings:

CASE I: Sidewalk at Exterior Stringer

Available Live Load Capacity per ft. at Inventory Level Rating = (W_i)

$$(W_i) = \frac{8\ (MLL_i)}{(\text{Des. Span Length})^2}$$

$$= \frac{8(\quad \text{ft-lbs})}{(\quad \text{ft.})^2} = \boxed{\quad\quad} \text{ lb/ft.}$$

Available Live Load Capacity per ft. at Operating Rating = (W_o)

$$(W_o) = \frac{8\ (MLL_o)}{(\text{Des. Span Length})^2}$$

$$= \frac{8(\quad \text{ft-lbs})}{(\quad \text{ft.})^2} = \boxed{\quad\quad} \text{ lb/ft.}$$

Inventory Rating =

$$\frac{W_i}{s} = \frac{(\quad \text{lb/ft})}{(\quad \text{ft})} = \boxed{\quad\quad} \text{ psf}$$

Operating Rating =

$$\frac{W_o}{s} = \frac{(\quad \text{lb/ft})}{(\quad \text{ft})} = \boxed{\quad\quad} \text{ psf}$$

These Inventory and Operating Ratings are the same for all loading types and should be entered in each row under "Exterior Stringer (Moment)" on the "Summary of Ratings" table in Chapter 8.

CASE II: No sidewalk at exterior stringer.

Determine distance from centerline of exterior stringer to centerline of wheel load (x_1):

x_1 = 2' + Wheel Guard Width - 1/2 stringer width (only if outer face of stringer and wheel guard line up)

x_1 = 2' + (in)(1/12) - (1/2)(in.)(1/12)

x_1 = ________ ft.

Determine Distribution Factor (DF):

$$DF = \frac{\text{Stringer Spacing} - x_1}{\text{Stringer Spacing}}$$

$$DF = \frac{(\quad \text{ft.}) - (\quad \text{ft.})}{(\quad \text{ft.})} = \_\_\_\_\_\_\_\_$$

Calculate Inventory and Operating Ratings:

The formula to be used to calcualte the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(M_{LLi})(GVW)}{(LLM)(1/2)(DF)(1000)}$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{(M_{LLo})(GVW)}{(LLM)(1/2)(DF)(1000)}$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 2.2 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Ext. Stringer (Moment)".

Calculate Applied Dead Load Horizontal Shear (V_{DL})

$V_{DL} = W_{DL}$[(Span Length)(1/2) - (x)]

V_{DL} = (lb/ft.)[(ft.)(1/2) - (ft.)]

V_{DL} = $\boxed{}$ lbs.

Calculate Resisting Horizontal Shear:

Inventory Resisting Horizontal Shear = (VR_i)

VR_i = (2/3)(Stringer Width)(Stringer Height)(f_{vi})

VR_i = (2/3)(in.)(in.)(psi.) = $\boxed{}$ lbs.

Operating Resisting Horizontal Shear = (VR_o)

VR_o = (2/3)(Stringer Width)(Stringer Height)(f_{vo})

VR_o = (2/3)(in.)(in.)(psi.) = $\boxed{}$ lbs.

Determine Resisting Horizontal Shear Available for Live Load:

Available Live Load Inventory Horizontal Shear = (V_{LLi})

$$(V_{LLi}) = (VR_i) - (V_{DL})$$

(V_{LLi}) = (lbs.) - (lbs.) = $\boxed{}$ lbs.

Available Live Load Operating Horizontal Shear = (V_{LLo})

$$(V_{LLo}) = (VR_o) - (V_{DL})$$

(V_{LLo}) = (lbs.) - (lbs.) = $\boxed{}$ lbs.

Calculate Inventory and Operating Horizontal Shear Ratings:

CASE I: Sidewalk at Exterior Stringer

Available Live Load Capacity per ft. at Inventory Rating (W_i)

$$(W_i) = \frac{2(V_{LLi})}{(\text{Span Length})}$$

$$(W_i) = \frac{2(\quad \text{lbs})}{(\quad \text{ft.})} = \boxed{} \text{ lb/ft.}$$

Available Live Load Capacity per ft. at Operating Rating (W_o)

$$(W_o) = \frac{2(V_{LLo})}{(\text{Span Length})}$$

$$(W_o) = \frac{2(\quad \text{lbs})}{(\quad \text{ft.})} = \boxed{} \text{ lb/ft.}$$

Inventory Rating

$$\frac{2\ (W_i)}{(\text{Stringer Spacing})} = \frac{(\qquad \text{lb/ft})}{(\qquad \text{ft.})} = \boxed{\qquad\qquad} \text{ lb/ft}^2$$

Operating Rating:

$$\frac{2\ (W_o)}{(\text{Stringer Spacing})} = \frac{(\qquad \text{lb/ft})}{(\qquad \text{ft.})} = \boxed{\qquad\qquad} \text{ lb/ft}^2$$

These Inventory and Operating Ratings are the same for all loading types and should be entered in each row under "Ext. Stringer Horizontal Shear)" on the "Summary of Ratings" table in Chapter 8.

CASE II: No Sidewalk at Exterior Stringer

Calculate Inventory and Operating Ratings:

the formula to be used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(V_{LLi})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{(V_{LLo})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 3.3 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Ext. Stringer (Horiz. Shear)".

Compute Dead Load (DL) per foot of Stringer:

s = 1/2[(Spacing of Stringer to Right) + (Spacing of Stringer to Left)]

s = 1/2[(ft.) + (ft.)] = ________ ft.

Wearing Surface:

Asphalt: (Thickness)(S)(144 lb/ft^3)(1/12)

= (in.)(ft.)(144 lb/ft^3)(1/12) = ________ lb/ft.

Concrete: (Thickness)(S)(150 lb/ft^3)(1/12)

= (in.)(ft.)(150 lb/ft^3)(1/12) = ________ lb/ft.

Timber Deck: (Thickness)(S)(50 lb/ft^3)(1/12)

= (in.)(ft.)(50 lb/ft^3)(1/12) = ________ lb/ft.

Stringer: (Width)(Height)(50 lb/ft^3)(1/144)

= (in.)(in.)(50 lb/ft^3)(1/144) = ________ lb/ft.

Total DL per ft. stringer (W_{DL}) = $\boxed{}$ lb/ft.

Calculate Applied Dead Load Moment (M_{DL}):

M_{DL} = (1/8)(W_{DL})(Span Length)^2

= (1/8)(lb/ft)(ft.)^2 = $\boxed{}$ ft-lbs.

Compute Section Modulus (Z) of Interior Stringer:

Due to the nature of deterioration in timber members, two conditions may exist; deterioration of the surfaces of the member, and internal deterioration. The computation of the section modulus is different for the two conditions; therefore the deterioration condition should be determined and the section modulus calculated using the appropriate case below. Where no interior deterioration is present use Case I.

CASE I: No interior deterioration. Height and width are measurements of the "sound" material, taking into account deterioration.

$$Z = \frac{(\text{Width})(\text{Height})^2}{6}$$

$$Z = \frac{(\quad \text{in.})(\quad \text{in.})^2}{6} = \boxed{\quad\quad} \text{ in.}^3$$

CASE II: Interior deterioration is present. Void size is to be approximated and represented by a void height, void width and the distance from the center of the void to the bottom of the stringer. Section modulus will be calculated for the "shell" of sound material.

$$Y_{na} = \frac{(1/2)(\text{Width})(\text{Height})^2 - (\text{Void Width})(\text{Void Height})(\text{dist. bot. stringer to } \mathrm{C\!\!\!L} \text{ Void})}{(\text{Width})(\text{Height}) - (\text{Void Width})(\text{Void Height})}$$

$$Y_{na} = \frac{(1/2)(\quad \text{in.})(\quad \text{in.})^2 - (\quad \text{in.})(\quad \text{in.})(\quad \text{in.})}{(\quad \text{in.})(\quad \text{in.}) - (\quad \text{in.})(\quad \text{in.})}$$

Y_{na} = ________ in.

$$I_o \text{ Stringer} = \frac{(\text{Width})(\text{Height})^2}{12}$$

$$= \frac{(\quad\quad \text{in.})(\quad\quad \text{in.})^3}{12} = \underline{\hspace{3cm}} \text{ in.}^4$$

$$I_o \text{ Void} = \frac{(\text{Void Width})(\text{Void Height})^3}{12}$$

$$= \frac{(\quad\quad \text{in.})(\quad\quad \text{in.})^3}{12} = \underline{\hspace{4cm}} \text{ in.}^4$$

$$\bar{Y} \text{ Stringer} = Y_{na} - 1/2(\text{Height})$$

$$= (\quad\quad \text{in.}) - 1/2(\quad\quad \text{in.}) = + \underline{\hspace{3cm}} \text{ in.}^*$$

$$\bar{Y} \text{ Void} = Y_{na} - (\text{dist. bot. Stringer to } \mathbb{C}_L \text{ Void})$$

$$= (\quad\quad \text{in.}) - (\quad\quad \text{in.}) = + \underline{\hspace{3cm}} \text{ in.}^*$$

$$I_{na} = (I_o \text{ Stringer}) - (I_o \text{ Void}) + (\text{Width})(\text{Height})(\bar{Y} \text{ Stringer})^2$$
$$- (\text{Void Width})(\text{Void Height})(\bar{Y} \text{ Void})^2$$

$$I_{na} = (\quad\quad \text{in}^4) - (\quad\quad \text{in.}^4) + (\quad\quad \text{in.})(\quad\quad \text{in.})(\quad\quad \text{in.})^2$$
$$- (\quad\quad \text{in.})(\quad\quad \text{in.})(\quad\quad \text{in.})^2 = \underline{\hspace{3cm}} \text{ in}^4$$

$$Z_{bottom} = \frac{I_{na}}{Y_{na}} = \frac{\quad\quad \text{in.}^4}{\quad\quad \text{in.}} = \boxed{\hspace{2.5cm}} \text{ in.}^3$$

$$Z_{top} = \frac{I_{na}}{(\text{Height} - Y_{na})} = \frac{\quad\quad \text{in.}^4}{(\quad\quad \text{in.} - \quad\quad \text{in.})} = \boxed{\hspace{2.5cm}} \text{ in.}^3$$

---

*If negative, the negative sign should be dropped. Both values should be positive.

Calculate Resisting Moments:

CASE I: No interior deterioration.

Inventory Resisting Moment $(MR_i) = \dfrac{(f_{bi})(Z)}{12}$

$= \dfrac{(\quad lb/in.^2)(\quad in.^3)}{12} = \boxed{\quad}$ ft-lbs.

Operating Resisting Moment $(MR_o) = \dfrac{(f_{bo})(Z)}{12}$

$= \dfrac{(\quad lb/in.^2)(\quad in.^3)}{12} = \boxed{\quad}$ ft-lbs.

CASE II: Interior deterioration is present.

Inventory Resisting Moment (MR_i) =

smaller of $\dfrac{(f_{bi})(Z_{top})}{12}$ or $\dfrac{(f_{bi})(Z_{bottom})}{12}$

$= \dfrac{(\quad lb/in.^2)(\quad in.^3)}{12} = \_\_\_\_\_\_$ ft-lb.

Use $\boxed{\quad}$ ft.lbs.

$= \dfrac{(\quad lb/in.^2)(\quad in.^3)}{12} = \_\_\_\_\_\_$ ft-lbs.

Operating Resisting Moment (MR_o) =

smaller of $\dfrac{(f_{bo})(Z_{top})}{12}$ or $\dfrac{(f_{bo})(Z_{bottom})}{12}$

$= \dfrac{(\quad lb/in.^2)(\quad in.^3)}{12} = \_\_\_\_\_\_$ ft-lb.

Use $\boxed{\quad}$ ft.lbs.

$= \dfrac{(\quad lb/in.^2)(\quad in.^3)}{12} = \_\_\_\_\_\_$ ft-lb.

Determining Resisting Moments Available for Live Load:

Available Capacity for Live Load Inventory Moment

$$(M_{LLi}) = (MR_i) - (M_{DL})$$

$$(M_{LLi}) = (\qquad\qquad) - (\qquad\qquad) = \boxed{\qquad\qquad} \text{ ft-lbs.}$$

Available Capacity for Live Load Operating Moment

$$(M_{LLo}) = (MR_o) - (M_{DL})$$

$$(M_{LLo}) = (\qquad\qquad) - (\qquad\qquad) = \boxed{\qquad\qquad} \text{ ft-lbs.}$$

Determine Distribution Factor (DF) =

$$\text{Number of design traffic lanes} = \frac{(\text{Roadway width})}{12}$$

rounded off to the next lower number of lanes except roadway widths from 18 to 24 ft. shall have two design traffic lanes.

$$\frac{(\qquad\quad \text{ft.})}{12 \text{ ft.}} = \underline{\qquad\qquad} \qquad \text{use } \boxed{\qquad\qquad} \text{ design lanes}$$

Based on type of floor and number of design lanes, determine Wheel Load Distribution Factor from Plate II in Appendix A:

$$DF = \frac{(\qquad\quad \text{ft.})}{(\qquad\quad)\text{ft.}} = \boxed{\qquad\qquad}$$

SECTION 12.1 - SHEAR CAPACITY RATING OF AN INTERIOR STRINGER

Calculate Applied Dead Load Horizontal Shear (V_{DL})

$V_{DL} = W_{DL}$[(Span Length)(1/2) - (X)]

V_{DL} = (lb/ft.)[(ft.)(1/2) - (ft.)]

V_{DL} = $\boxed{}$ lbs.

Calculate Resisting Horizontal Shear:

Inventory Resisting Horizontal Shear (VR_i)

VR_i = (2/3)(Stringer Width)(Stringer Height)(f_{vi})

VR_i = (2/3)(in.)(in.)(psi.) = $\boxed{}$ lbs.

Operating Resisting Horizontal Shear:

VR_o = (2/3)(Stringer Width)(Stringer Height)(f_{vo})

VR_o = (2/3)(in.)(in.)(psi.) = $\boxed{}$ lbs.

Determine Resisting Horizontal Shear Available for Live Load

Available Live Load Inventory Horizontal Shear

$(V_{LLi}) = (VR_i) - (V_{DL})$

$(V_{LLi}) = (\qquad \text{lbs.}) - (\qquad \text{lbs.}) = \boxed{\qquad}$ lbs.

Available Live Load Operating Horizontal Shear

$(V_{LLo}) = (VR_o) - (V_{DL})$

$(V_{LLo}) = (\qquad \text{lbs.}) - (\qquad \text{lbs.}) = \boxed{\qquad}$ lbs.

Calculate Inventory and Operating Horizontal Shear Ratings:

The formula to be used to calculate the Inventory Rating for each loading type is:

$$\text{Inventory Rating} = \frac{(V_{LLi})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

and the formula to be used to calculate the Operating Rating for each loading type is:

$$\text{Operating Rating} = \frac{(V_{LLo})(GVW)}{(1/4)[(.6)(LLV) + (DF)(LLV)](1000)}$$

The Inventory and Operating Ratings should be calculated on separate sheets of paper for each loading type described in Section 3.3 and entered in the "Summary of Ratings" table in Chapter 8 under the heading "Int. Stringer (Horiz. Shear)".

Loading Classification	TYPE OF LOADING	RATING LEVEL											
Florida Legal Loading	SU2	Inventory											
		Operating											
	SU3	Inventory											
		Operating											
	SU4	Inventory											
		Operating											
	C3	Inventory											
		Operating											
	C4	Inventory											
		Operating											
	C5	Inventory											
		Operating											
Design Loading	H	Inventory											
		Operating											
	HS	Inventory											
		Operating											

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

REINFORCED CONCRETE SLAB SUPERSTRUCTURE

RATING FORMS

SECTION 12.2 - REINFORCED CONCRETE SLAB SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE: These sheets should be completed in their entirety before continuing with the rating computations.

By: ______________________________ Date: ______________

Bridge Name: ____

Bridge No.: __________ Road No.: __________ County: ______________

Bridge Length: __________ ft. Number of Spans: ______________

Span Length ($\mathbb{C}_L$ support to $\mathbb{C}_L$ Support) = __________
(if different, list each)

Overall Bridge Width = __________ ft.

Roadway Width (curb to curb) = __________ ft.

Sidewalk: Width = ________ ft.
Rail Size = ________ in. x ________ in.
Rail Post Size & Length = _______ in. x _______ in. x ______ ft.
Average Post Spacing = __________ ft.

Barrier Rails: Rail Size = ________ in. x _______ in.
Rail Post Size & Length = _______ in. x _______ in. x _______ ft.
Average Post Spacing = _______ ft.

Wearing Surface:

Thickness = __________ in.

Concrete Slab:

Thickness = __________ in.
Longitudinal Rein. (Bottom) Size ______ Avg. Spacing = _______ in.
Longitudinal Rein. (Top) Size ______ Avg. Spacing = ________ in.
Transverse Rein. (Bottom) Size ______ Avg. Spacing = _______ in
Transverse Rein. (Top) Size ______ Avg. Spacing = ________ in.
d = ________ in. (distance from top of slab to centerline of bottom longitudinal reinforcing).

Material Strengths:

f'c (compressive strength of concrete) = __________ psi.
(from contract plans*)

fy (yield strength of reinforcing steel) = __________ psi.
(from contract plans**)

These unit stresses shall only be used when, in the judgement of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the maximum yield stresses and compressive strength of concrete shall be fixed by the Registered Professional Engineer, based on the field investigation, and shall be substituted for the previously given stresses. These stresses shall in no case be greater than the previously given stresses.

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* When the compressive strength of the concrete is unknown, f'c may be taken as 3,000 psi., subject to the above paragraph.

**When the Grade is given, or the steel is unknown, use the following yield stresses for reinforcing steel:

Reinforcing Steel	Yield Point (fy) psi
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Intermediate Grade and unknown after 1954 (GRADE 40)	40,000
Hard grade (GRADE 50)	50,000
(GRADE 60)	60,000

Dead Load (DL) per linear foot for a one foot strip of slab:

Barrier:

(Rail Width)(Rail Height)
(in.)(in.)(1/144) = ________ $ft.^2$ -(1)

(Curb Width)(Curb Height)
(in.)(in.)(1/144) = ________ $ft.^2$ -(2)

(Rail Post Length)(Height)(Width)(1/Avg. Post Spacing)
(ft.)(in.)(in.)(1/ ft.)(1/144)
= ________ $ft.^2$ -(3)

Barrier Weight per foot =

[(1)+(2)+(3)](150 $lb/ft.^3$) (Number of Barriers)(1/Overall Bridge Width)
= [($ft.^2$)+($ft.^2$)+($ft.^2$)](150)( )

(1/ ft.) = $\boxed{}$ lb/ft. per one foot strip

Asphalt Wearing Surface:

(Roadway Width)(144 $lb/ft.^3$)(Thickness)(1/Overall Bridge Width)
= (ft.)(144)(in.)(1/12)(1/ ft.)

= $\boxed{}$ lb/ft. per one foot strip

Concrete Slab:

(Thickness)(150 $lb/ft.^3$)

= (in.)(1/12)(150) = $\boxed{}$ lb/ft. per one foot strip

Total DL per one foot strip (W_{DL}) = $\boxed{}$ lb/ft.

SECTION 12.2 - MOMENT CAPACITY RATING

Calculate Applied Dead Load Moment (M_{DL}):

M_{DL} = (1/8)(W_{DL})(Span Length)^2(1/1000)

= (1/8)(lb/ft)(ft.)^2(1/1000) = $\boxed{}$ ft-kips

Determine Area of Reinforcing Steel (As) to be used in computing Moment Capacity of Slab:

actual As = (Ab)*(1/Avg. spacing for Longitudinal Reinf.)(12)

= (in.^{2})(1/ in.)(12) = $\boxed{}$ in.^{2}/ft.

Calculate Asb:

$$A_{sb} = \frac{.85(\ )(f'c)(d)(12\text{ in.})}{(fy)}\ \frac{87{,}000}{87{,}000 + fy}$$

$$.75\ A_{sb} = \frac{(.75)(.85)(\quad)(\quad \text{psi})(\quad \text{in.})(12)}{(\quad \text{psi})}\ \frac{87{,}000}{87{,}000 + (\quad \text{psi})}$$

$.75\ A_{sb}$ = $\boxed{}$ in.^{2}/ft.

If, as most instances should, the actual area of steel (A_s) is less than the balanced steel area (A_{sb}) use the lesser of the actual A_s or .75 A_{sb}.

As = $\boxed{}$ in.^{2}/ft.

If the actual area of steel (A_s) is greater than the balanced steel area (A_{sb}), compression controls the analysis and a professional engineer should be consulted.

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*Area of reinforcing bar, see Plate IV in Appendix A for area of bar, given the bar designation from the Inspection Data Sheet.

Determine Ultimate Moment Capacity (Mu) of Member:

$$Mu = \frac{(As)(fy)(d)}{12} - \frac{(As)^2(fy)^2}{2(.85)(f'c)(144)}$$

$$Mu = \frac{(\quad in.^2)(\quad psi.)(\quad in.)}{12} - \left[\frac{(\quad in.^2)^2(\quad psi)^2}{(1.7)(\quad psi.)(144)}\right]$$

Mu = (________ ft-lbs)(1/1000)

Mu = $\boxed{\quad}$ ft-kips.

Determine Impact Factor:

$$I = 1 + \left[\frac{50}{(\text{Span Length} + 125)}\right] \text{, but not greater than 1.30}$$

$$I = 1 + \left[\frac{50}{(\quad ft. + 125)}\right] = \underline{\qquad}$$

use I = $\boxed{\quad}$

Determine Distribution Factor:

Span Length = center to center of supports = ______ ft. but not greater than:

($\mathbb{C}_L$ to $\mathbb{C}_L$ of supports) - (Pier Cap Width) + (slab thickness)

= (ft.) - (in.)(1/12) + (in.)(1/12)

= __________ ft.

use span length = $\boxed{\quad}$ ft.

DF = 4 + 0.06 (Span Length), but not greater than 7.0

DF = 4 + 0.06 (ft.) = ________

use DF = $\boxed{\quad}$

SECTION 12.2 - SHEAR CAPACITY RATING

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = (W_{DL})[(\text{Span Length})(1/2) - (x)](1/1000)$$

$$V_{DL} = (\quad \frac{lb}{ft})[(\quad ft)[\frac{1}{2}] - (\quad ft)](\frac{1}{1000}) = \boxed{\quad\quad} \text{ kips}$$

Determine Impact Factor

$$I = 1 + [\frac{50}{(\text{Span Length}) - (x) + 125}]$$, but not greater than 1.30

$$I = 1 + \frac{50}{(\quad ft.) - (\quad ft.) + 125} = \_\_\_\_\_\_\_\_;\ \text{use } I = \boxed{\quad\quad}$$

Determine the Maximum permitted Shear Stress for the concrete slab (v_c):

Method One:

$$v_c = 2\sqrt{f'c}$$

$$v_c = 2\sqrt{\quad psi} = \_\_\_\_\_\_\_\_\_\_ \text{ psi}$$

Method Two:

$$v_c = 1.9\sqrt{f'c} + 2{,}500\ [\frac{As}{12d}][\frac{[V_{DL} + LLV\ d]}{M'u}] \leftarrow \text{equation [A]}$$

but not greater than $3.5\sqrt{f'c}$, and the quantity $\frac{(V_{DL} + LLV\ d)}{M'u}$ shall not exceed 1.0

First, compute M'u (the factored M_{DL} + LLM occurring simultaneously with V_{DL} + LLV at the section being considered).

contribution of M'_{DL} to M'u:

$$M'_{DL} = \frac{(W_{DL})(X)^*}{2}(\text{Span Length} - x)$$

$$M'_{DL} = \frac{(\quad\quad \text{lb/ft.})(\quad\quad \text{ft.})}{2}(\quad\quad \text{ft.} - \quad\quad \text{ft.})$$

M'_{DL} = ____________ ft-lbs.

Contributions of LLM' to M'u shall be determined by evaluating the moment resulting from the axle placements used to compute LLV for each loading case. Shown below is a general equation to calculate the Live Load Moment for these axle placements. If the load case being examined has less than five axles on the span, simply delete the parts of the equation pertaining to the unused axles. Example: if three axles are being used, delete the terms $\frac{P_4X}{L}(L-x-a-b-c)$ and $\frac{P_5X}{L}(L-x-a-b-c-d)$.

This equation is to be evaluated for each loading case that was used in evaluating shear. The variables x, a, b, c, & d are the same as those used in the shear calculations.

$$LLM' = \frac{P_1X}{L}(L-X)+\frac{P_2X}{L}(L-X-a)+\frac{P_3X}{L}(L-X-a-b)+\frac{P_4X}{L}(L-X-a-b-c)+\frac{P_4X}{L}(L-X-a-b-c-d)$$

now, for each loading case compute the value of the expression:

$$\frac{(V_{DL} + LLV)d}{(M'_{DL} + LLM')}$$

The largest value is the one to be used in equation [A]; unless that value exceeds 1.0, in which case 1.0 should be used in equation [A] in place of that computed above.

---

*X is from the live load shear computations completed as a part of Section 3.3

Evaluating equation [A]

$$v_c = 1.9 \sqrt{\qquad\quad \text{lb/in}} + 2500\left[\frac{(\qquad\quad \text{in})}{12(\qquad\quad \text{in})}\right](\qquad\qquad)$$

$$v_c = \underline{\qquad\qquad} \text{ lb/in}$$

but not greater than 3.5 $\sqrt{f'c}$

$$= \sqrt{\qquad\quad \text{lb/in}}$$

$$= \underline{\qquad\qquad} \text{ lb/in}$$

therefore, vc = $\boxed{\qquad\qquad}$ lb/in. by method two.

Calculate Ultimate Shear Capacity (Vu) of member:

$$Vu = (v_c)(d)(12)$$

$$Vu = (\qquad \text{psi})(\qquad \text{in.})(12 \text{ in.})(1/1000)$$

$$Vu = \boxed{\qquad\qquad} \text{ kips}$$

Calculate Inventory and Operating Shear Ratings:

Inventory Rating:

$$RF = \frac{\left[\frac{(0.85)(Vu)}{1.3} - V_{DL}\right]\left(\frac{3}{5}\right)(DF)}{(1/2)(LLV)(I)}$$

Operating Rating:

$$RF = \frac{\left[\frac{(0.85)(Vu)}{1.3} - V_{DL}\right](DF)}{(1/2)(LLV)(I)}$$

Loading Classification	TYPE OF LOADING	RATING LEVEL											
Florida Legal Loading	SU2	Inventory											
		Operating											
	SU3	Inventory											
		Operating											
	SU4	Inventory											
		Operating											
	C3	Inventory											
		Operating											
	C4	Inventory											
		Operating											
	C5	Inventory											
		Operating											
Design Loading	H	Inventory											
		Operating											
	HS	Inventory											
		Operating											

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

REINFORCED CONCRETE T-BEAM SUPERSTRUCTURE

RATING FORMS

SECTION 12.3 - REINFORCED CONCRETE T-BEAM SUPERSTRUCTURE INSPECTION DATA SHEET

NOTE: These sheets should be completed in their entirety before continuing with the rating computations.

By: ______________________________ Date: ______________

Bridge Name: ____

Bridge No.: __________ Road No.: __________ County: ______________

Bridge Length: __________ ft. Number of Spans: ______________

Span Length ($\mathbb{C}_L$ support to $\mathbb{C}_L$ Support) = __________
(if different, list each)

Overall Bridge Width = __________ ft.

Roadway Width (curb to curb) = ____________ ft.

Sidewalk: Width = ________ ft.
Rail Size = ________ in. x ________ in.
Rail Post Size & Length = _______ in. x _______ in. x ______ ft.
Average Post Spacing = ___________ ft.

Barrier Rails: Rail Size = ________ in. x _______ in.
Rail Post Size & Length = _______ in. x _______ in. x _______ ft.
Average Post Spacing = _______ ft.

Wearing Surface:

Thickness = ___________ in.

Concrete Slab:

Thickness = ___________ in.
Longitudinal Rein. (Bottom) Size ______ Avg. Spacing = _______ in.
Longitudinal Rein. (Top) Size ______ Avg. Spacing = ________ in.
Transverse Rein. (Bottom) Size ______ Avg. Spacing = _______ in
Transverse Rein. (Top) Size ______ Avg. Spacing = ________ in.
d = ________ in. (distance from top of slab to centerline of bottom longitudinal reinforcing).

Concrete Beam:

Height = __________ in.
Width = __________ in.
Spacing Between Beams __________ ft.
Tension Reinforcement (reinforcement in bottom of beam)

size __________
number of bars = __________
d (distance from centroid of reinforcement to top of concrete slab) = __________ in.

Shear reinforcement in outer quaters of T-beam.

stirrup size = __________
stirrup spacing = __________ in.

Material Strengths:

f'c (compressive strength of concrete) = __________ psi.
(from contract plans*)

fy (yield strength of reinforcing steel) = __________ psi.
(from contract plans**)

These unit stresses shall be used when, in the judgement of a Registered Professional Engineer, the materials under consideration are sound and reasonably equivalent in strength to new materials of the grade and qualities that would be used in first class construction. When the materials are deemed substandard (by grading, manufacture, or deterioration) the maximum yield stresses and compressive strength of concrete shall be fixed by the Registered Professional Engineer, based on the field investigation, and shall be substituted for the previously given stresses.

---

* When the compressive strength of the concrete is unknown, f'c may be taken as 3,000 psi, subject to the above paragraph.

** When the Grade is given or the steel is unknown, use the following yield stresses for reinforcing steel:

Reinforcing Steel	Yield Point (fy) psi
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Intermediate Grade and Unknown after 1954 (GRADE 40)	40,000
Hard grade (GRADE 50)	50,000
(GRADE 60)	60,000

Dead Load (DL) per linear foot for slab:

Asphalt Wearing Surface:

(Thickness)(144 lb/ft^3)(1/12)

= (in.)(144 lb/ft^3)(1/12) = ________ lb/ft per 1 ft. strip

Slab:

(Thickness)(150 lb/ft^3)(1/12)

= (in.)(150 lb/ft^3)(1/12) = ________ lb/ft per 1 ft. strip

Total DL per foot per 1 ft. strip (WS_{DL}) for Slab

= $\boxed{}$ lb/ft per 1 foot strip

Dead Load per linear foot for T-beam:

Barrier:

(Rail Width)(Rail Height)

(in.)(in.) (1/144) = __________ ft.^{2} -(1)

(Curb Width)(Curb Height)

(in.)(in.)(1/144) = __________ ft.^{2} -(2)

(Rail Post Length)(Height)(Width)(1/Avg. Post Spacing)

(ft.)(in.)(in.)(1/   ft.)(1/144)

= __________ ft.^{2} -(3)

Barrier Weight per Foot:

= [(1) + (2) + (3)](150 lb/ft^3)(Number of Barriers)
(1/Number of T-Beam Sections)

= [(ft^2) + (ft^2) + (ft^2)](150)( )(1/ ft.)

= $\boxed{}$ lb per foot.

Determine Nominal Flange Width (s):

s = (Beam Spacing)(12)

= (ft)(12) = ____________ in.

Asphalt Wearing Surface:

(S)(Thickness)(144 lb/ft^3)(1/144)

= (in)(in) = $\boxed{}$ lb/ft.

Concrete Slab:

(S)(Thickness)(150 lb/ft^3)(1/144)

= (in)(in)(150)(1/144) = $\boxed{}$ lb/ft

Beam:

(Beam Width)(Beam Height)(150 lb/ft^3)(1/144)

= (in)(in)(150)(1/144) = $\boxed{}$ lb/ft.

Total DL per foot of T-Beam Section (W_{DL}) = $\boxed{}$ lb/ft.

Calculate Applied Dead Load Moment (M_{DL}):

Effective Slab Span (S) = (Beam Spacing) - (Beam Width)

s = (ft) - (in)(1/12) = ________ ft.

$M_{DL} = (1/10)(WS)_{DL}(S)^2(1/1000)$

$= (1/10)(1/1000)(\quad \text{lb/ft})(\quad \text{ft})^2 = \boxed{\quad\quad}$ ft-kips

Determine Area of Reinforcing Steel (A_s) to be used in computing Moment Capacity of Slab

If, as most instances should, the actual area of steel (A_s) is less than the balanced steel area (A_{sb}), the lesser of the actual A_s or .75 A_{sb} should be used.

If the actual area of steel (A_s) is greater than the balanced steel area (A_{sb}), compression controls the analysis and a professional engineer should be consulted.

Actual A_s = (Ab)(1/Max. Spacing for Transverse Reinf.)(12)

= (in^2)(1/   in)(12) = ________ in^2/ft.

Calculate A_{sb}:

$$A_{sb} = \frac{.85(\ \ )(f'c)(d)(12)}{(fy)} \frac{87000}{87000 + fy}$$

$$.75\ A_{sb} = \frac{(.75)(.85)(\qquad)(\qquad psi)(\qquad in)(12)}{(\qquad psi)} \frac{87000}{87000 + (\qquad psi)}$$

$$.75\ A_{sb} = \boxed{\qquad\qquad} \ in^2/ft.$$

now, taking the lesser of the A_s or $.75\ A_{sb}$, $A_s = \boxed{\qquad\qquad}$ $in^2/ft.$

Determine Ultimate Moment Capacity (Mu) of Slab:

$$Mu = \frac{(A_s)(fy)(d)}{12} - \frac{(A_s)^2(fy)^2}{2(.85)(f'c)(144)}$$

$$Mu = \frac{(\qquad in^2)(\qquad psi)(\qquad in)}{12} - \frac{(\qquad in^2)^2(\qquad psi)^2}{(1.7)(\qquad psi)(144)}$$

Mu = __________ ft-lbs ÷ 1000 = $\boxed{\qquad\qquad}$ ft-kips.

SECTION 12.3 - MOMENT CAPACITY RATING FOR T-BEAM

Calculate Applied Dead Load Moment(M_{DL}):

$$M_{DL} = (1/8)(W_{DL})(\text{Span Length})^2(1/1000)$$

$$M_{DL} = (1/8)(\quad\quad \text{lb/ft})(\quad\quad \text{ft})^2(1/1000) = \boxed{\quad\quad} \text{ ft-kips}$$

Determine Effective Flange Width (s):

$$s = \text{Beam Spacing} = (\quad\quad \text{ft})(12 \text{ in./ft}) = \boxed{\quad\quad} \text{ in.}$$

But not greater than:

$$\frac{\text{Span Length}}{4} = \frac{(\quad\quad \text{ft})}{4} \; \frac{12 \text{ in}}{\text{ft.}} = \underline{\quad\quad\quad} \text{ in.}$$

And not greater than:

$$Wb + 12(\text{Slab Thickness}) = (\quad\quad \text{in}) + 12(\quad\quad \text{in}) = \underline{\quad\quad\quad} \text{ in.}$$

$$\text{Use } s = \boxed{\quad\quad} \text{ in.}$$

Determine Area of Reinforcing Steel (A_s) to be used in computing Moment Capacity of the member:

Actual $A_s = (A_b)$(number of bars)

(for reinforcement in bottom of beam)

$$= (\quad\quad \text{in}^2)(\quad\quad) = \boxed{\quad\quad} \text{ in}^2$$

Locate Neutral Axis:

$$c = \frac{(A_s)\ (f_y)}{.85\ (f'_c)\ (\beta)\ (s)}$$

$$c = \frac{(\quad\quad \text{in}^2)\ (\quad\quad \text{psi.})}{.85\ (\quad\quad \text{psi.})\ (\quad\quad)\ (\quad\quad \text{in.})} = \underline{\quad\quad} \text{ in.}$$

If c is less than or equal to the slab thickness:

$$A_{sb} = \left[\frac{.85\ (\beta)(f'c)(d)(W_b)}{(f_y)}\right] \left[\frac{87000}{87000 + f_y}\right]$$

$$A_{sb} = \left[\frac{.85\ (\quad)(\quad psi)(\quad in)\ (\quad in)}{(\quad psi)}\right] \left[\frac{87000}{87000 + (\quad psi)}\right]$$

A_{sb} = __________ in^2

If the actual steel area (A_s) is less than the balanced steel area (A_{sb}), the lesser of the actual A_s or .75 A_{sb} should be used.

If the actual steel area (A_s) is greater than the balanced steel area (A_{sb}), compression controls the analysis and a professional engineer should be consulted.

.75 A_{sb} = __________ in^2

now, taking the lesser of A_s or .75 A_{sb}, A_s = ________ in^2

Determine Ultimate Moment Capacity (M_u):

$a = (\beta)\ (c)$

$a = (\quad)\ (\quad in)$ = __________ in

$$M_u = [(A_s)(f_y)][(d) - \frac{(a)}{2}]$$

$$M_u = \left[\frac{(\quad in^2)(\quad psi)}{(12)}\right] \left[(\quad in) - \frac{(\quad in)}{2}\right]$$

M_u = __________ ft-lbs ÷ 1000 = ____________ ft-kips

If c is greater than the slab thickness:

$$A_{sf} = \frac{.85\ (f'c)(s-W_b)(\text{slab thickness})}{f_y}$$

$$A_{sf} = \frac{.85\ (\quad \text{psi})(\quad \text{in} - \quad \text{in})(\quad \text{in})}{(\quad \text{psi})}$$

A_{sf} = __________ in^2

Calculate A_{sb}:

$$A_{sb} = \left[\frac{.85(\beta)(f'c)(d)(W_b)}{fy}\ \frac{87000}{87000 + fy} + A_{sf}\right] \frac{W_b}{s}$$

$$.75\ A_{sb} = .75\ \left[\frac{(.85)(\quad)(\quad \text{psi})(\quad \text{in.})(\quad \text{in.})}{(\quad \text{psi})}\right.$$

$$\left.\frac{87000}{87000 + (\quad \text{psi})} + (\quad \text{in.}^2)\right]\ \frac{(\quad \text{in})}{(\quad \text{in})}$$

$.75A_{sb}$ = $\boxed{}$ in^2

If the actual area of steel (A_s) is less than the balanced steel area (A_{sb}), the lesser of the actual A_s or .75 A_{sb} should be used.

If the actual area of steel (A_s) is greater than the balanced steel area (A_{sb}), a professional engineer should be consulted.

now, taking the lesser of A_s or .75 A_{sb}, A_s = $\boxed{}$ in^2

Determine Ultimate Moment Capacity (Mu) of T-Beam:

$$c = \frac{(A_s - A_{sf})(fy)}{.85(f'c)(\beta)(W_b)}$$

$$= \frac{(\quad in^2 - \quad in^2)(\quad psi)}{.85(\quad psi)(\quad)(\quad in)} = \_\_\_\_\_\_\_\_ \text{ in.}$$

$a = (\beta)(c)$

$a = (\quad)(\quad in) = \_\_\_\_\_\_\_\_$ in.

$$M_u = [(A_s - A_{sf})(fy)(d - \frac{a}{2}) + (A_{sf})(f_y)(d - \frac{\text{slab thickness}}{2})]$$

$$M_u = [(\quad in^2 - \quad in^2)(\quad psi)(\quad in - \frac{in}{2}) + (\quad in^2)$$
$$(\quad psi)(\quad in - \frac{in}{2})]\ (\frac{1}{12})$$

$M_u = \_\_\_\_\_\_\_\_$ ft-lbs ÷ 1000 = $\_\_\_\_\_\_\_\_$ ft-kips

Determine Impact Factor (I):

$$I = 1 + \frac{50}{\text{Span Length} + 125}\text{, but not greater than } 1.30$$

$$I = 1 + \frac{50}{(\qquad \text{ft}) + (125)} = \underline{\qquad\qquad}$$

Use I = $\boxed{\qquad\qquad}$

Determine Distribution Factor (DF):

$$\text{Number of design traffic lanes} = \frac{\text{Roadway Width}}{12}$$

rounded off to the next lower number of lanes except roadway widths of 18 to 24 feet shall have two design lanes.

$$\frac{(\qquad \text{ft.})}{12 \text{ ft.}} = \underline{\qquad\qquad}$$

Use $\boxed{\qquad\qquad}$ design lanes

Based on the number of design lanes, determine the Wheel Load Distribution Faction equation from Plate II in Appendix A:

$$DF = \frac{(\qquad \text{ft.})}{(\qquad \text{ft.})} = \boxed{\qquad\qquad}$$

SECTION 12.3 - SHEAR CAPACITY RATING FOR T-BEAM

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = (W_{DL})[(\text{Span Length})(1/2) - (x)](1/1000)$$

$$V_{DL} = (\quad\quad \text{lb/ft})[(\quad\quad \text{ft.})(1/2) - (\quad\quad \text{ft.})](1/1000)$$

$$V_{DL} = \boxed{\quad\quad\quad} \text{ kips}$$

Calculate Impact Factor (I):

$$I = 1 + \frac{50}{(\text{Span Length}) - (x) + (125)}$$, but not greater than 1.30

$$I = 1 + \frac{50}{(\quad\quad \text{ft.}) - (\quad\quad \text{ft.}) + (125)}$$

$I =$ __________ Use $I = \boxed{\quad\quad\quad}$

Determine the maximum permitted Shear Stress for concrete (v_c):

Method One:

$$v_c = 2\sqrt{f'c}$$

$$v_c = 2\sqrt{\quad\quad\quad \text{psi}} = \underline{\quad\quad\quad\quad} \text{ psi}$$

Method Two:

$$v_c = 1.9\sqrt{f'c} + 2{,}500\,\frac{A_s}{12d}\,\frac{(V_{DL} + LLV)d}{M'u} \leftarrow \text{Equation [A]}$$

but not greater than $3.5\sqrt{f'c}$ and the quantity $\frac{(V_{DL} + LLV)d}{M'u}$

shall not exceed 1.0.

First, Compute M'u (the factored M_{DL} + LLM occurring simultaneously with V_{DL} + LLV at the section being considered).

Contribution of M'_{DL} to M'u

$$M'_{DL} = (1/2)(W_{DL})(x)(\text{Span Length} - x)$$

$$M'_{DL} = (1/2)(\qquad \text{lb/ft.})(\qquad \text{ft.})(\qquad \text{ft.} - \qquad \text{ft.})$$

$$M'_{DL} = \underline{\qquad\qquad} \text{ ft.-lbs.}$$

Contribution of LLM' to M'u shall be determined by evaluating the moment resulting from the axle placements used to compute LLV for each loading case. Shown below is a general equation to calculate the live load moment for these axle placements. If the load case being examined has less than five axles on the span, simply delete the parts of the equation pertaining to the unused axles. Example: If three axles are being used, delete the terms:

$$\frac{P_4 x}{L}(L-x-a-b-c) \text{ and } \frac{P_5 x}{L}(L-x-a-b-c-d).$$

General Equation:

$$LLM' = \frac{P_1 x}{L}(L-x) + \frac{P_2 x}{L}(L-x-a) + \frac{P_3 x}{L}(L-x-a-b) + \frac{P_4 x}{L}(L-x-a-b-c) + \frac{P_5 x}{L}(L-x-a-b-c-d)$$

This equation is to be evaluated for each loading case that was used in evaluating shear. The variables x,a,b,c, and d are the same as those used in the shear calculations that were completed as a part of Section 3.3.

Now for each loading case being evaluated, compute the value of the expression:

$$\frac{(V_{DL} + LLV)d}{(M'_{DL} + LLM')}, \text{ but not greater than 1.0}$$

Evaluating equation [A] for each load case we obtain the value of v_c to be used for each load case.

Determine Shear carried by concrete:

$$V_c = (v_c)(W_b)(d)(1/1000)$$

$$V_c = (\quad\quad \text{psi})(\quad\quad \text{in.})(\quad\quad \text{in.})(1/1000)$$

$$V_c = \boxed{\quad\quad\quad} \text{ kips}$$

Determine Shear carried by stirrups:

Case I: Stirrups placed perpendicular to longitudinal axis of T-Beam.

$$V_s = \frac{2(Av)(fy)(d)}{(s)(1000)}$$

$$V_s = \frac{2(\quad\quad \text{in.}^2)(\quad\quad \text{psi})(\quad\quad \text{in.})}{(\quad\quad \text{in.})(1000)} = \boxed{\quad\quad\quad} \text{ kips}$$

Case II - Stirrups placed inclined with the longitudinal axis of the T-Beam.

$$V_s = \frac{2(Av)(fy)(\text{Sin} \quad + \text{Cos} \quad)(d)}{(s)(1000)}$$

$$V_s = \frac{2(\quad\quad \text{in.}^2)(\quad\quad \text{psi.})(\quad\quad + \quad\quad)(\quad\quad \text{in.})}{(\quad\quad \text{in.})(1000)}$$

$$V_s = \boxed{\quad\quad\quad} \text{ kips.}$$

Determine ultimate shear capacity of T-Beam (Vu):

$$Vu = V_c + V_s = (\qquad \text{kips}) + (\qquad \text{kips})$$

$$Vu = \boxed{\qquad\qquad} \text{ kips.}$$

Calculate Inventory and Operating Shear Ratings for T-Beam:

The following formulae are to be used to compute the Inventory and Operating Shear Capacity Ratings for each load case. The Live Load Shear used in the formulae is from Section 3.2 of this manual. Notice that the numerator of the rating factor equations does not change for the different loading cases, therefore it will need to be calculated only once. The denominator changes and must be recalculated for each different loading case. As the Rating Factor (RF) is computed for each loading case it should be multiplied by the GVW to obtain the Rating Value. This Rating Value should then be entered under the appropriate type of loading in the "Summary of Ratings" table in Chapter 8 under the heading "Shear Rating for T-Beam".

Inventory Rating

$$RF = \frac{\left[\dfrac{(0.85)(Vu)}{1.3} - V_{DL}\right](3/5)}{(1/2)(DF)(LLV)(I)}$$

Operating Rating

$$RF = \frac{\dfrac{(0.85)(Vu)}{1.3} - V_{DL}}{(1/2)(DF)(LLV)(I)}$$

Loading Classification	TYPE OF LOADING	RATING LEVEL											
Florida Legal Loading	SU2	Inventory											
		Operating											
	SU3	Inventory											
		Operating											
	SU4	Inventory											
		Operating											
	C3	Inventory											
		Operating											
	C4	Inventory											
		Operating											
	C5	Inventory											
		Operating											
Design Loading	H	Inventory											
		Operating											
	HS	Inventory											
		Operating											

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM SUPERSTRUCTURE RATING FORMS

SECTION 12.4 - PRESTRESSED CONCRETE CHANNEL AND SLAB SYSTEM INSPECTION DATA SHEET

NOTE: These sheets should be completed in their entirety before continuing with the rating computations.

By: ______________________________ Date: ______________

Bridge Name: ____

Bridge No.: __________ Road No.: ________ County: ______________

Bridge Length = ___________ ft. Number of Spans = ______________

Span length ($\mathbb{C}_L$ to $\mathbb{C}_L$ of support = ___________ft.
(if different, list for each)

Overall Bridge Width = ________________ ft.

Bridge Width (curb to curb) = _________________ ft.

Sidewalk: Width = ___________ ft.
Rail Size = ________ in. x _______ in.
Rail Post Size & Length = _______ in. x ______ in. x _____ ft.
Average Post Spacing = ____________ ft.

Barrier Rails: Rail Size = ________ in. x ______ in.
Rail Post Size & Length = ______ in. x ______ in. x _______ ft.
Average Post Spacing = ___________ ft.

Curb Block: height x width = _______ in. x ______ in.

Wearing Surface: asphalt; thickness = ________ in.
concrete; thickness = ________ in.

Cast in place concrete deck surface: thickness = _______ in.

transverse reinf. = #_______ at _______ in. spacing.
longitudinal reinf. = #______ at _______ in. spacing.

or, WWF = __________

Prestressed Channel Section:

Number of Channels = ____________

Thickness of cast in place concrete deck surface = ___________ in.

Flange thickness = _____________ in.

Web depth = _____________ in.

Web width at top = ______________ in.

Web width at bottom = _____________ in.

Web spacing = ____________ in.

Nominal channel width = ___________ in.

Strand locations at ends:

Distance from bottom of web to 1st strand = ____________ in.
Distance from bottom of web to 2nd strand = ____________ in.
Distance from bottom of web to 3rd strand = ____________ in.
Distance from bottom of web to 4th strand = ____________ in.
Distance from bottom of web to 5th strand = ____________ in.

Strand locations at mid span:

Distance from bottom of web to 1st strand = ____________ in.
Distance from bottom of web to 2nd strand = ____________ in.
Distance from bottom of web to 3rd strand = ____________ in.
Distance from bottom of web to 4th strand = ____________ in.
Distance from bottom of web to 5th strand = ____________ in.

Prestressing Strand Size: __________ in. diameter; Grade ______________

(270^k high-strength or 250^k ASTM designation)

Initial Tensioning Load _____________ lbs. each

Concrete Strength = ___________ psi. (for prestressed channels)

Concrete Strength = ___________ psi. (for cast in place concrete deck)

Shear Reinforcing (Stirrups):

Side View of Channel

outer 1/4 (1)	Stirrup Spacing = ____________ in. Stirrup Bar Size = ____________ fy (yield strength of steel) = _____________ psi. (From contract plans[1])
middle 1/3 (2)	Stirrup Spacing = ____________ in. Stirrup Bar Size = ____________ fy (yield strength of steel) = _____________ psi. (from contract plans)

[1]When the Grade is given, or the steel is unknown, use the following yield stresses for reinforcing steel:

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

SECTION 12.4 - CALCULATION OF DEAD LOAD PER LINEAR FOOT OF CHANNEL

Calculate Barrier Area:

Barrier: (Rail Width)(Rail Height)
(in.)(in.)(1/144) = ________ ft.^{2}
(Curb Width)(Curb Height)
(in.)(in.)(1/144) = ________ ft.^{2}
(Rail Post Length)(Height)(Width)(1/Ave. Post Spacing)
(ft.)(in.)(in.)(1/ ft)(1/144)
= ________ ft.^{2}

Barrier Area = $\boxed{}$ ft.^{2}

Barrier Dead Load per Foot:

= (Barrier Area)(150 lb/ft.^{3})(Number of Barriers)(1/Number of Prestressed Channels)
= (ft.^{2})(150)( )(1/) = $\boxed{}$ lb/ft.

Asphalt Wearing Surface Dead Load per foot:

= (Roadway Width)(144 lb/ft^3)(Thickness)(1/Number of Prestressed Channels)

= (ft.)(144)(in.)(1/12)(1/) = $\boxed{}$ lb/ft.

Cast In Place Concrete Deck Surface Dead Load Per Foot:

= (Thickness of Cast-In-Place Concrete Deck Surface)(Nominal Channel Width)(1/144)(150 lb/ft^3)

= (in.)(in.)(1/144)(150) = $\boxed{}$ lb/ft.

Channel Dead Load Per Foot:

Channel area = (Flange Thickness)(Nominal Channel Width) + [(Web Width at Top) + (Web Width at Bottom)](Web Depth)

= (in.)(in.) + [(in.) + (in.)](in.)

= $\boxed{}$ in^2

Channel Dead Load per foot = (Channel Area)(1/144)(150 lb/ft.^{3})

= (in.^{2})(1/144)(150) = ______ lb/ft.

Dead Load on Channel Only Per Linear Foot (W_{DL1})

W_{DL1} = (Channel DL) = (Cast in place slab DL)

W_{Dl1} = (lb/ft) + (lb/ft) = ______lb/ft

Dead Load on Composite Section Per Linear Foot (W_{DL2})

W_{DL2} = (Barrier DL) + (Asphalt Wearing Surface DL)

W_{DL2} = (lb/ft) + (lb/ft) = ______lb/ft

Total Dead Load per linear foot of channel (ΣW_{DL})

Barrier DL + Asphalt Wearing Surface DL + Channel DL + Cast In Place Concrete Deck Surface DL

ΣW_{DL} = (lb/ft.)+(lb/ft.)+(lb/ft.)+(lb/ft)

= ______ lb/ft.

SECTION 12.4 - MOMENT CAPACITY RATING AT INVENTORY LEVEL
(Considering Servicibility Requirements)

LOADS AND STRESSES ON CHANNEL ONLY:

Calculate Applied Dead Load Moment(M_{DL1}):

M_{DL1} = (1/8)(W_{DL1}) (Span Length)^2(12)
M_{DL1} = (1/8)(lb/ft.)(ft.)^2(12) = $\boxed{}$ in-lbs.

Enter M_{DL1} in the appropriate space in the table on sheet 12.4-10.

Compute Section Properties for Channel Only:

Area of 1 (A_1) = (W_b)(W_d) = (in.)(in.) = ________ in.^{2}

Area of 2 (A_2) = (W_t)(W_d) = (in.)(in.) = ________ in.^{2}

Area of 3 (A_3) = (W_c)(T_f) = (in.)(in.) = ________ in.^{2}

Channel area = $\boxed{}$ in.^{2}

Moment of Inertia*(I_1) of 1 = $\dfrac{(W_b)(W_d)^3}{36}(2)$

$= \dfrac{(\quad\text{in.})(\quad\text{in.})^3}{36}(2) =$ ________ in.^{4}

Moment of Inertia (I_2) of 2 = $\dfrac{(W_t)(W_d)^3}{36}(2)$

$= \dfrac{(\quad\text{in.})(\quad\text{in.})^3}{36}(2) =$ ________ in.^{4}

Moment of Inertia (I_3) of 3 = $\dfrac{(W_c)(T_f)^3}{12}$

$= \dfrac{(\quad\text{in.})(\quad\text{in.})^3}{12} =$ ________ in.^{4}

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* For those interested in additional detail relating to calculation of moments of inertia, reference is made to Elements of Strength of Materials, p. 346.

SECTION PROPERTIES OF CHANNEL ONLY (Cont.):

$Y_{b1} = 1/3(W_d) = 1/3\ (\quad \text{in.}) = \underline{\qquad\qquad}$ in.

$Y_{b2} = 2/3(W_d) = 2/3\ (\quad \text{in.}) = \underline{\qquad\qquad}$ in.

$Y_{b3} = W_d + 1/2\ (T_f) = (\quad \text{in.}) + (1/2)(\quad \text{in.}) = \underline{\qquad\qquad}$ in.

$$Y_{na} = \frac{(A_1)(Y_{b1}) + (A_2)(Y_{b2}) + (A_3)(Y_{b3})}{A_1 + A_2 + A_3}$$

$$Y_{na} = \frac{(\quad \text{in.}^2)(\quad \text{in.})+(\quad \text{in.}^2)(\quad \text{in.})+(\quad \text{in.}^2)(\quad \text{in.})}{(\quad \text{in.}^2) + (\quad \text{in.}^2) + (\quad \text{in.}^2)}$$

$$= \boxed{\qquad\qquad} \text{ in.}$$

Now, the moment of inertia of the prestressed concrete channel section (I_{na}) equals:

$$I_1 + I_2 + I_3 + A_1(Y_{na} - Y_{b1})^2 + A_2(Y_{na} - Y_{b2})^2 + A_3(Y_{b3} - Y_{na})^2$$

$$I_{na} = (\quad \text{in.}^4)+(\quad \text{in.}^4)+(\quad \text{in.}^4)+(\quad \text{in.}^2)(\quad \text{in.} - \quad \text{in.})^2$$

$$+ (\quad \text{in.}^2)(\quad \text{in.} - \quad \text{in.})^2+(\quad \text{in.}^2)(\quad \text{in.} - \quad \text{in.})^2$$

$$= \boxed{\qquad\qquad} \text{ in.}^4$$

Section modulus at the top of the channel (S_T):

$$S_T = \frac{I_{na}}{(W_d + T_f - Y_{na})}$$

$$S_T = \frac{(\quad \text{in.}^4)}{(\quad \text{in.}) + (\quad \text{in.}) - (\quad \text{in.})} = \boxed{\qquad\qquad} \text{ in.}^3$$

SECTION PROPERTIES OF CHANNEL ONLY (Cont.):

Section modulus at the bottom of the channel (S_B)

$$S_B = \frac{I_{na}}{Y_{na}}$$

$$S_B = \frac{(\qquad \text{in.}^4)}{(\qquad \text{in.})} = \boxed{\qquad\qquad} \text{ in.}^3$$

S_B and S_T should be entered in the appropriate spaces in the table on sheet 12.4-10.

Calculate Initial Prestressing Load (P_i):

$$P_i = \text{(Number of Strands)(Tensioning Load)}$$

$$P_i = (\qquad\qquad)(\qquad\quad \text{lbs.}) = \underline{\qquad\qquad} \text{ lbs.}$$

Calculate Initial Prestress (f_i):

$$f_i = \frac{-P_i}{\text{Channel Area}} = \frac{(-)\qquad\qquad \text{lbs.}}{\text{in.}^2} = \boxed{-\qquad\qquad} \text{ psi*}$$

This value, including the minus sign, should be entered in the appropriate spaces in the table on sheet 12.4-10.

Determine Prestress Moment (P_{ie}) at Midspan:

$$\bar{y}_1 = \frac{2(\text{dim.1} + \text{dim.2} + \text{dim.3} + \text{dim.4})}{\text{(total number of strands)}}$$

$$\bar{y}_1 = \frac{2(\qquad \text{in.} + \qquad \text{in.} + \qquad \text{in.} + \qquad \text{in.})}{(\qquad)}$$

$$\bar{y}_1 = \underline{\qquad\qquad} \text{ in.}$$

NOTE: 1. Dimensions 1,2,3 and 4 are illustrated on the opposite page.
2. If there are less than 4 rows of strands, enter zero for the dimension of unused rows.

---

* The sign convention used in this manual is (-) for compression and (+) for tension.

STRESSES ON CHANNEL ONLY:

$e_1 = Y_{na} - \bar{y}_1$

$e_1 = (\quad \text{in.}) - (\quad \text{in.}) = \_\_\_\_\_\_\_\_ \text{ in.}$

$P_i e_1 = (Pi)(e_1)$

$= (\quad \text{lbs.})(\quad \text{in.})$

$= \boxed{\quad\quad} \text{ in-lbs.}$

Determine Prestress Moment ($P_i e_2$) at Ends:

$$\bar{y}_2 = \frac{2(\text{dim.1} + \text{dim.2} + \text{dim.3} + \text{dim.4})}{(\text{Total Number of Strands})}$$

$$\bar{y}_2 = \frac{2(\quad \text{in.} + \quad \text{in.} + \quad \text{in.} + \quad \text{in.})}{(\quad\quad)}$$

$\bar{y}_2 = \_\_\_\_\_\_\_\_ \text{ in.}$

$e_2 = Y_{na} - \bar{y}_2$

$e_2 = (\quad \text{in.}) - (\quad \text{in.}) = \_\_\_\_\_\_\_\_ \text{ in.}$

$P_i e_2 = (Pi)(e_2)$

$= (\quad \text{lbs.})(\quad \text{in.})$

$= \boxed{\quad\quad} \text{ in-lbs.}$

Determine Fractional Loss of Prestress (a):

A^*_s = (Number of Strands)(Area of Strands)*

$= (\quad)(\quad \text{in}^2) = \_\_\_\_\_\_\_\_ \text{ in}^2$

$$a = \frac{45{,}000\ A^*_s}{Pi}$$

$$a = \frac{45{,}000\ (\quad \text{in.}^2)}{(\quad \text{lbs.})} = \_\_\_\_\_\_\_\_$$

*Strand sizes and properties are tabulated for common strand sizes on Plate Iv in Appendix A.

LEGEND FOR SUMMARY OF STRESSES						
Item	Section Modulus (in^3)	Moment (in-lbs.)	At Midspan Top	At Midspan Bottom	At Ends Top	At Ends Bottom
Prestress Force			f_i	f_i	f_i	f_i
Prestress Moment at Midspan	S_T / S_B	P_ie_1	$\frac{P_ie_1}{S_T}$	$-\frac{(P_ie_1)}{S_B}$		
Prestress Moment at Ends	S_T / S_B	P_ie_2			$\frac{P_ie_2}{S_T}$	$-\frac{(P_ie_2)}{S_B}$
Loss of Prestress Force			$-(a)\left(\frac{P_ie_1}{S_T}+f_i\right)$	$-(a)\left(\frac{-P_ie_1}{S_B}+f_i\right)$	$-(a)\left(\frac{P_ie_2}{S_T}+f_i\right)$	$(a)\left(\frac{-P_ie_2}{S_B}+f_i\right)$
Dead Load	S_T / S_B	M_{DL}	$-\frac{M_{DL}}{S_T}$	$\frac{M_{DL}}{S_B}$		
TOTALS			σ_1	σ_2	σ_3	σ_4

SUMMARY OF STRESSES (in psi.)						
Item	Section Modulus (in^3)	Moment (in-lbs.)	At Midspan Top	At Midspan Bottom	At Ends Top	At Ends Bottom
Prestress Force						
Prestress Moment at Midspan						
Prestress Moment at Ends						
Loss of Prestress Force						
Dead Load						
TOTALS *						

* When totaling the columns, pay strict attention to (+) and (-) signs.

COMPUTE SECTION PROPERTIES OF THE COMPOSITE SECTION:

Reduce the thickness of the cast in place slab by 1/4" to account for scaling

$$T_d = \begin{bmatrix} \text{Adjusted} \\ \text{Thickness of Cast} \\ \text{In Place Slab} \end{bmatrix} \frac{\sqrt{\text{Concrete Strength of Slab}}}{\sqrt{\text{Concrete Strength of Channel}}}$$

$$T_d = [(\quad \text{in.} - .25\ \text{in})] \left[\frac{\sqrt{\quad \text{psi.}}}{\sqrt{\quad \text{psi}}}\right] = \_\_\_\_\_\_\ \text{in.}$$

Area of 4(A4) = $(W_c)\ (T_d) = (\quad \text{in})\ (\quad \text{in}) = \_\_\_\_\_\_\ \text{in.}^2$

Moment of Inertia of 4 $(I_4) = \dfrac{(W_c)(T_d)^3}{12} = \dfrac{(\quad \text{in})(\quad \text{in})^3}{(\quad\quad)} = \_\_\_\_\_\_\ \text{in}^4$

$Y_{b4} = (W_d) + (T_f) + 1/2\ (T_d) = (\quad \text{in}) + (\quad \text{in}) + 1/2\ (\quad \text{in}) = \_\_\_\_\_\_\ \text{in}$

$$Y_{na}\ \text{comp} = \frac{(\text{Area of Channel})\ (Y_{na}\ \text{Channel} + (A_4)\ (Y_{b4})}{(\text{Area of Channel}) + (A_4)}$$

$$Y_{na}\ \text{comp} = \frac{(\quad \text{in}^2)\ (\quad \text{in}) + (\quad \text{in}^2)(\quad \text{in})}{(\quad \text{in}^2) + (\quad \text{in}^2)} = \_\_\_\_\_\_\ \text{in.}$$

The moment of Inertia of the composite section equals:

$$I_{na}\ \text{comp} = I_{na} + I_4 + (\text{Area of Channel})(Y_{na}\ \text{comp} - Y_{na})^2 + A_4(Y_{b4} - Y_{na}\ \text{comp})^2$$

$$I_{na}\ \text{comp.}\ (\quad \text{in}^4) + (\quad \text{in}^4) + (\quad \text{in}^2)(\quad \text{in}- \quad \text{in})^2 + (\quad \text{in}^2)\ (\quad \text{in}- \quad \text{in})^2$$

I_{na} comp = $\_\_\_\_\_\_\ \text{in}^4$

Section Modulus at the top of the channel

$S_T\ \text{comp} = I_{na}\ \text{comp} / (W_d + T_f - Y_{na}\ \text{comp})$

$S_T\ \text{comp} = (\quad \text{in}^4)/(\quad \text{in} + \quad \text{in} - \quad \text{in}) = \_\_\_\_\_\_\ \text{in}^3$

Section Modulus at the bottom of the channel

$S_B\ \text{comp} = I_{na}\ \text{comp} / Y_{na}\ \text{comp}$

$S_B\ \text{comp} = (\quad \text{in}^4) / (\quad \text{in}) = \_\_\_\_\_\_\ \text{in}^3$

SECTION 12.4 - MOMENT CAPACITY RATING AT INVENTORY LEVEL (CONT.)

LOADS AND STRESSES ON COMPOSITE SECTION:

Calculate Applied Dead Load Moment (M_{DL2}):

____

M_{DL2} = (1/8) (W_{DL2}) (Span Length)^2 (12)

M_{DL2} = (1/8) (lb/ft) (ft)^2 (12) = ________ in-lbs

At mid-span the corresponding stresses are:

Top of Channel, σ_1 comp = $\dfrac{M_{DL2}}{S_T \text{ comp}} = \dfrac{(\quad\quad \text{in-lbs})}{(\quad\quad \text{in}^3)}$ = ____________ psi

Bottom of Channel, σ_2 comp = $\dfrac{M_{DL2}}{S_B \text{ comp}} = \dfrac{(\quad\quad \text{in-lbs})}{(\quad\quad \text{in}^3)}$ = ____________ psi

Now combining these with the stresses already in the channels (Page 12.4-10) yields . . . .

At midspan:

Top of Channel, σ_1 Total = σ_1 + σ_1 comp

= (psi) + (psi) = __________ psi

Bottom of Channel, σ_2 Total = σ_2 + σ_2 comp

= (psi) + (psi) = ________ psi

At ends:

Top of Channel, Total = σ_3 (page 12.4-10) = ___________ psi

Bottom of Channel, Total = σ_4 (page 12.4-10) = ___________ psi

Determine Allowable Maximum Unit Stresses:

Allowable in compression (σ_c):

σ_c = 0.40 (f'c) = 0.40(psi) = $\boxed{-\quad}$ psi.

Allowable in tension (σ_t)

for a coastal environment (over salt water)

$\sigma_t = 3\sqrt{f'c} = 3\sqrt{\quad\quad psi}$ = $\boxed{+\quad}$ psi

or, for a non-coastal environment

$\sigma_t = 6\sqrt{f'c} = 6\sqrt{\quad\quad psi}$ = $\boxed{+\quad}$ psi

Determine Available Moment Capacity for Live Load($M_a i$):

Available Moment Capacity for the two conditions shown below should be computed. The controlling condition for available Live Load Moment (M_{ai}) is the lower value of equations 1 or 2. The stresses σ_3 and σ_4 represent the stresses in the ends of the member due to prestressing forces. Stress σ_3 should not be greater than σ_t and σ_4 should not be greater (in absolute value) than σ_c. If either σ_3 or σ_4 exceed the allowable stress, a special note should be placed on the Summary of Ratings Table as follows:

"The stresses introduced at the ends of the members due to prestressing forces exceed the allowable stresses."

σ_1 Total, σ_2 Total, σ_3 and σ_4 are the summary of stresses due to prestressing and all dead loads as calculated on previous pages.

Include (-) and (+) signs in all calculations.

1. [σ_1 Total - σ_c] (S_T comp) (1/12000)

   = [(psi) - (psi)](in^3)(1/12000)

   = $\boxed{\quad\quad}$ ft-kips

2. $[\sigma_t - \sigma_2 \text{ Total}] (S_B \text{ comp}) (1/12000)$

$= [(\quad \text{psi.}) - (\quad \text{psi})](\quad \text{in.}^3)(1/12{,}000)$

$= \boxed{\quad\quad}$ ft.-kips

And, controlling condition $(M_a i) = \boxed{\quad\quad}$ ft-kips.

Determine Impact Factor (I):

$$I = 1 + \left[\frac{50}{(\text{Span Length} + 125')}\right], \text{ but not greater than } 1.30$$

$$I = 1 + \left[\frac{50}{(\quad \text{ft.}) + 125)}\right] = \underline{\qquad\qquad}$$

use $I = \boxed{\quad\quad}$

Determine Distribution Factor (DF):

Number of Design Traffic Lanes $(N_L) = \dfrac{(\text{Roadway Width})}{12}$

rounded off to the next lower number of lanes except for roadway widths of 18 to 24 feet use two design traffic lanes.

$$\frac{(\qquad \text{ft})}{12} = \_\_\_\_\_\_\_\_\_\_. \quad \text{Use } N_L = \boxed{\qquad} \text{ design lanes}$$

N_C = number of channels in span = $\boxed{\qquad}$

$$c = 2.2\frac{(\text{Overall Bridge Width})}{\text{Design Span Length}}$$

$$= 2.2\,\frac{\qquad \text{ft.}}{\qquad \text{ft.}} = \boxed{\qquad}$$

If c is less than or equal to 3.0:

$$DF = \frac{12(N_L) + 9}{N_C\left[5 + \frac{N_L}{10} + \left(3 - \frac{2N_L}{7}\right)\left(1 - \frac{c}{3}\right)^2\right]}$$

$$DF = \frac{12(\qquad) + 9}{(\qquad)\left[5 + \frac{(\qquad)}{10} + \left(3 - \frac{2(\qquad)}{7}\right)\left(1 - \frac{(\qquad)}{3}\right)^2\right]}$$

$DF = \boxed{\qquad}$

If c is greater than 3.0:

$$DF = \frac{12(N_L) + 9}{N_C\left(5 + \frac{N_L}{10}\right)} = \frac{12(\qquad) + 9}{(\qquad)\left(5 + \frac{(\qquad)}{10}\right)}$$

$DF = \boxed{\qquad}$

SECTION 12.4 - MOMENT CAPACITY RATING AT OPERATING LEVEL
(Considering Strength Requirements)

Begin by computing the following parameters:

Wc = nominal channel width = ____________ in.

(From Section 2.5)

$$d = W_d - \bar{Y}_1 + T_f + T_d$$

$$= (\quad \text{in.}) - (\quad \text{in.}) + (\quad \text{in.}) + (\quad \text{in.})$$

$$= \underline{\qquad\qquad} \text{ in.}$$

$$A^*_s = \text{(Number of Strands)(Area of Strands)}^*$$

$$= (\quad)(\quad \text{in.}^2) = \underline{\qquad\qquad} \text{ in.}^2$$

$$p^* = \frac{A^*_s}{(Wc)(d)} = \frac{(\quad \text{in.}^2)}{(\quad \text{in.})(\quad \text{in.})} = \underline{\qquad\qquad}$$

$$f's = \frac{\text{Ultimate Strength Per Strand}}{\text{Area Per Strand}}$$

$$= \frac{(\quad \text{lb.})}{(\quad \text{in}^2)} = \underline{\qquad\qquad} \text{ lb/in.}^2$$

---

* Strand areas and properties are tabulated for common strand sizes on Plate IV in Appendix A.

$$f^*_{su} = (f's)\left[1 - \frac{(0.5(p^*)(f's)}{f'c}\right]$$

$$= (\qquad\qquad \text{psi.})\left[1 - \frac{(0.5)(\qquad\qquad)(\qquad\qquad \text{psi})}{(\qquad\qquad \text{psi})}\right]$$

= __________ psi.

CHECK MAXIMUM STEEL PERCENTAGE:

$P^* \; f^*_{su}/f'c \leq .30$

$(\qquad\qquad)\;(\qquad\qquad \text{psi})/(\qquad\qquad \text{psi}) =$

Compute Ultimate Moment Capacity (Mu):

IF STEEL PERCENTAGE $\leq$ .30

$$Mu = A^*_s f^*_{su} d\left[1 - \frac{(0.6)(p^*)(f^*_{su})}{f'c}\right]$$

$$Mu = (\qquad \text{in}^2)(\qquad\qquad \text{psi})(\qquad \text{in})\left[1 - \frac{(0.6)(\qquad\qquad)(\qquad\qquad \text{psi})}{(\qquad\qquad \text{psi})}\right]$$

Mu = __________ in-lbs ÷ 12,000 = $\boxed{\qquad\qquad}$ ft.-K

IF STEEL PERECENTAGE >.30:

$Mu = .25\ f'cW_cd^2$

$Mu = .25\ (.\qquad \text{psi})(\qquad \text{in})(\qquad \text{in})^2\ (1/12000) =$ ______ ft. k.

SECTION 12.4 - SHEAR CAPACITY RATINGS FOR OUTER QUARTERS OF CHANNEL

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = \frac{(\Sigma W_{DL})\ (\text{Span Length})}{4(1000)} = \frac{(\qquad \text{lb/ft.})(\qquad \text{ft.})}{4000}$$

$$V_{DL} = \boxed{\qquad\qquad} \text{ kips}$$

Compute Impact Factor (I):

$$I = 1 + \frac{50}{(3/4)(\text{Span Length}) + 125}$$, but not greater than 1.30

$$I = 1 + \frac{50}{(3/4)(\qquad \text{ft}) + 125} = \underline{\qquad\qquad}$$

Use $I = \boxed{\qquad\qquad}$

Compute "d":

$$d = W_d + T_f + T_d - [\bar{y}\ \text{midspan} + 1/2\ (\bar{y}\ \text{ends} - \bar{y}\ \text{midspan})]$$

$$d = (\qquad \text{in}) + (\qquad \text{in.}) + (\qquad \text{in.}) - [(\qquad \text{in}) + (1/2)(\qquad \text{in} - \qquad \text{in})]$$

$$d = \boxed{\qquad\qquad} \text{ in.}$$

Compute "j":

$$j = 1 - \frac{0.6p^*f^*_{su}}{f'c}$$

$$p^* = \frac{A^*s}{(W_c)(\bar{d})}$$

$$p^* = \frac{(\qquad in^2)}{(\qquad in)(\qquad in)}$$

$$p^* = \underline{\hspace{6cm}}$$

$$f^*_{su} = f's(1- \frac{0.5\ p^*f's}{f'c})$$

$$f^*_{su} = (\qquad psi.)(1 - \frac{0.5(\qquad)(\qquad psi.)}{(\qquad psi.)})$$

$$f^*_{su} = \underline{\hspace{5cm}}\ psi.$$

$$so,\ j = 1 - \frac{0.6(\qquad)(\qquad psi.)}{(\qquad psi.)} = \boxed{\qquad\qquad}$$

SECTION 12.4 - SHEAR CAPACITY RATINGS FOR OUTER QUARTERS OF CHANNEL (CONT.)

Compute Shear Carried by Concrete (Vc):

$Vc = 0.06\ f'c\ jd\ (w_t + w_b)$

but not greater than $180\ jd(w_t + w_b)$

Vc = 0.06 (psi.)(   )(in)(in +   in)

Vc = ________ lbs ÷ 1000 = ________ kips

Vc = 180 (   )(in)(in +   in)

Vc = ________ lbs ÷ 1000 = ________ kips

Use Vc = $\boxed{}$ kips

Compute Shear Carried by Stirrups (Vs):

$$Vs = \frac{2(Av)(fy)(j)(d)}{(\text{Stirrup Spacing})(1000)}$$

$$Vs = \frac{2(\quad in^2)(\quad psi.)(\quad)(\quad in)}{(\quad in)(1000)}$$

Vs = $\boxed{}$ kips

Compute Ultimate Shear Capacity of Channel (Vu):

Vu = Vc + Vs = (kips) + (kips)

Vu = $\boxed{}$ kips

SECTION 12.4 - SHEAR CAPACITY RATINGS FOR MIDDLE THIRD OF CHANNEL

Calculate Applied Dead Load Shear (V_{DL}):

$$V_{DL} = \frac{(\Sigma W_{DL})\ (\text{Span Length})}{6(1000)} = \frac{(\qquad \text{lb/ft.})(\qquad \text{ft.})}{6000}$$

$$V_{DL} = \boxed{\qquad} \text{ kips}$$

Compute Impact Factor (I):

$$I = 1 + \frac{50}{(2/3)(\text{Span Length}) + 125}\text{, but not greater than } 1.30$$

$$I = 1 + \frac{50}{(2/3)(\qquad \text{ft}) + 125} = \underline{\qquad\qquad}$$

$$\text{Use } I = \boxed{\qquad}$$

Compute "d":

$$d = W_d + T_f + T_d - [\bar{y}\text{ midspan} + 1/3\ (\bar{y}\text{ ends} - \bar{y}\text{ midspan})]$$

$$d = (\qquad \text{in}) + (\qquad \text{in.}) + (\qquad \text{in.}) - [(\qquad \text{in}) + (1/3)(\qquad \text{in} - \qquad \text{in})]$$

$$d = \boxed{\qquad} \text{ in.}$$

Compute "j":

$$j = 1 - \frac{0.6p^*f^*_{su}}{f'c}$$

$$p^* = \frac{A^*s}{(W_c)(d)}$$

$$p^* = \frac{(\quad\quad in^2)}{(\quad\quad in)(\quad\quad in)}$$

$$p^* = \underline{\hspace{4cm}}$$

$$f^*_{su} = f's[1- \frac{0.5\ p^*f's}{f'c}]$$

$$f^*_{su} = (\quad\quad psi.)[1 - \frac{0.5(\quad\quad)(\quad\quad psi.)}{(\quad\quad psi.)}]$$

$$f^*_{su} = \underline{\hspace{4cm}}\ psi.$$

$$so,\ j = 1 - \frac{0.6(\quad\quad)(\quad\quad psi.)}{(\quad\quad psi.)} = \boxed{\hspace{2cm}}$$

Compute Shear Carried by Concrete (Vc):

$Vc = 0.06\ f'c\ jd\ (w_t + w_b)$

but not greater than $180\ jd(w_t + w_b)$

Vc = 0.06 (psi.)(   )(in)(in +   in)

Vc = ________ lbs ÷ 1000 = ________ kips

Vc = 180 (   )(in)(in +   in)

Vc = ________ lbs ÷ 1000 = ________ kips

Use Vc = $\boxed{}$ kips

Compute Shear Carried by Stirrups (Vs):

$$Vs = \frac{2(Av)(fy)(j)(d)}{(\text{Stirrup Spacing})(1000)}$$

$$Vs = \frac{2(\quad in^2)(\quad psi.)(\quad)(\quad in)}{(\quad in)(1000)}$$

Vs = $\boxed{}$ kips

Compute Ultimate Shear Capacity of Channel (Vu):

Vu = Vc + Vs = (kips) + (kips)

Vu = $\boxed{}$ kips

Loading Classification	TYPE OF LOADING	RATING LEVEL											
Florida Legal Loading	SU2	Inventory											
		Operating											
	SU3	Inventory											
		Operating											
	SU4	Inventory											
		Operating											
	C3	Inventory											
		Operating											
	C4	Inventory											
		Operating											
	C5	Inventory											
		Operating											
Design Loading	H	Inventory											
		Operating											
	HS	Inventory											
		Operating											

Note: All ratings are in kips except exterior stringers with sidewalk loading in which the rating is in (lb./ft.^{2}). Design Loading for sidewalks is 85 lb./ft^2.

SUMMARY OF RATINGS

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