The Mn/DOT Bridge Office presents

BRIDGE RATING 101

Class Description:

With the ever increasing load demands on our local bridges, and a heightened awareness on bridge safety, there is no better time to bring our Minnesota County Engineers together to start the education process on "Bridge Load Rating". At the request of the County Engineer's Standards & Bridge Committee, the Mn/DOT Bridge Office has developed a one day basic bridge rating workshop for the County Engineers and their assistants.

What can the County Engineer expect to learn by attending the Bridge Load Rating Workshop? They will learn the what, why, when and how on rating local bridges. Common questions such as why rate, when to rate, and who is qualified to rate will be addressed. Truck trends and effects on bridge rating, and the ideal bridge rating history will be discussed in the workshop.

The workshop will cover rating procedures, discussing the applied loads and the capacity of distressed members. There will be rating example problems and class exercises to help you gain the necessary knowledge to assess whether or not an existing simple span local bridge can tolerate additional dead weight from a proposed overlay, etc... The workshop will discuss important information related to posting, and permitting bridges. Legal loads, special hauling vehicles, and posting rules will also be discussed!

The workshop will allow time for discussing common mistakes when rating and posting bridges, and offer ample time to take questions and comments for discussion and answer, and take requests for possible follow up materials.

The class participant will also go home with worked out rating examples and other valuable reference materials.

Who should attend? The class is designed for the County Engineers and their assistants interested in learning the basics of local bridge rating and posting. Interested City Engineers and their assistants are also welcomed. So leave your calendars open for the date and convenient location listed below. Sign up early, class size is limited to 50 participants. Your attendance is appreciated.

Class Locations and Times:

- Duluth, March 26th, 2008
- Bemidji, March 27th, 2008
- St Cloud, March 4th, 2008
- Marshall, March 5th, 2008
- Owatonna, March 13th, 2008

(All classes will run from 9:00 am to 3:30 pm)
(A big lunch will be included)

2008 Bridge Load Rating Class 101 Agenda

I. Introductions - Dave — 9:00-9:05 (5 min)
   a. Presenters
II. Load Rating Basics - Dave — 9:05-9:40 (35 min)
   a. Rating information
      i. Why rate?
      ii. What code to use?
      iii. When to do a new rating?
      iv. Who is qualified?
      v. When to hire a consultant?
   b. Current rating ages
   c. Ideal bridge rating history
   d. Truck trends
III. General Equations - Brian — 9:40-10:00 (20 min)
   a. Rating Equation
   b. Inventory level
   c. Operating level

Break — 10:00-10:10 (10 min)

IV. Load Rating Procedure - Brian — 10:10-10:40 (30 min)
   a. Applied Loads
      i. Dead Load
      ii. Live Load
   b. Capacity
   c. Equation calculations
   d. Submitting results to MnDOT
V. Incorporating Member Distress - Ed — 10:40-10:55 (15 min)
   a. Loss of Capacity
   b. High stress areas
      i. Moment
      ii. Shear
      iii. Bearing
      iv. Compression
      v. Tension
VI. Posting, SHV's and Permitting - Ed — 10:55-11:40 (45 min)
   a. Legal loads
   b. Posting trucks
   c. Specialized Hauling Vehicles
   d. Posting rules
   e. Overweight permits

Lunch — 11:40-12:40 (60 min)

VII. Load Rating Ex. #1 - Simple span non-composite steel – Brian — 12:40-1:15 (35 min)
   a. With/without overlay
   b. With/without section loss
   c. Posting load
   d. Overweight Permit

VIII. Load Rating Ex. #2 - Simple span nail laminated timber - Dave 1:15-1:45 (30 min)
deck (without distress)
a. Calculate wheel load distribution
b. Calculate section modulus and capacity
c. Calculate dead load flexural stress
d. Calculate live load flexural stress
e. Calculate load ratings and postings

IX. Class Exercise - Simple span timber slab (with distress) - Dave 1:45-2:10 (25 min)
a. Group 1 – Wheel load distribution, section modulus, capacity
b. Group 2 – Dead load flexural stress
c. Group 3 – Live load flexural stress
d. Group 4 – Operating ratings
e. Group 5 – Postings

Break 2:10-2:25 (15 min)

X. Brief worked out example reviews - Dave 2:25-2:40 (15 min)
a. Abutment Timber piling (without decay)
b. Abutment Timber piling (with decay)
c. Abutment timber pile cap with loss of pile
d. Calculating section properties with section loss
e. Additional considerations for different bridge types
f. Other example from manuals

XI. Submittal to MnDOT and Review of process - Ed 2:40-3:10 (30 min)
a. Forms
i. Bridge Load Rating Form
ii. Culvert rating sheet
iii. Physical Inspection Form
b. Website
c. Bridge Management/Pontis
d. Results of Timber Haulers county rating project

XII. Common mistakes/questions - Ed 3:10-3:30 (20 min)

2008
Bridge Load Rating Class 101

Workshop Agenda

I. Introductions
II. Load Rating Basics
III. General Equations
IV. Load Rating Procedure
V. Incorporating Member Distress
VI. Posting, SHV's and Permitting
VII. Load Rating Example #1
   - Simple Span Non-composite Steel

Workshop Agenda

VIII. Load Rating Example #2
   - Simple Span Nail Laminated Timber Deck (without distress)
IX. Class Exercise
   - Simple Span Nail Laminated Timber Deck (with distress)
X. Review of worked out examples
XI. Submittal to MN/DOT and Review of Process
XII. Common mistakes and questions

Bridge Load Rating Class 101 Presenters

- Dave Conkel, P.E.
  State Aid Bridge Engineer

- Brian Homan, P.E.
  Assistant State Aid Bridge Engineer

- Ed Lutgen, P.E.
  North Region Bridge Engineer

Special Thanks to our Class Sponsors

- Minnesota County Engineers Association
  Bridge Committee
  Alan Forsberg, Blue Earth Co.
  Jim Grube, Hennepin Co.,
  & Members

  Standards Committee
  Gary Danielson, Kandiyohi Co.
  Dave Enblom, Cass Co.,
  & Members

Special Thanks to our Class Sponsors

- Special County Engineers
  Class Planning
  Duane Blanck (Retired), Crow-Wing Co.
  David Halbersma, Pipestone Co.

- Mn/DOT
  State Aid Office
  Patti Loken, Mary Ann Hillyer, Julie Skallman, Rick Kjonaas
  & Staff
  Bridge Office
  Dan Dorgan, Gary Peterson, Lowell Johnson,
  Kevin Western, Jim Pierce, Steve Brown
  & Staff

Bridge Load Rating Class 101 Course Goal

- The class was born from legislation, dubbed the "Timber Haulers Bill", which allows special permits with appropriate fees for increased vehicle loading.
- This class is financed with money from the Timber Haulers Bill, and was developed due to an overall general interest of the Minnesota County Engineer in load rating and posting.

Bridge Load Rating Class 101 Course Goal

- This class was created with input from other Minnesota County Engineers and Mn/DOT Bridge Rating Engineers to give us:
  - A basic understanding of local bridge rating and posting.
  - Provide information on the use of load rating equations, and load rating procedures.
  - Provide informational handouts with rating and posting examples for various types of bridges.

Bridge Load Rating Class 101 Course Goal

- Provide information on special hauling vehicles, posting rules and overweight permits.
- Provide information on bridge load rating forms, culvert rating sheets, and other Mn/DOT related materials.
- Note, this class is not intended, however, to make all class participants proficient at rating bridges.

Workshop Agenda

I. Introductions
II. Load Rating Basics
III. General Equations
IV. Load Rating Procedure
V. Incorporating Member Distress
VI. Posting, SHV's and Permitting
VII. Load Rating Example #1
   - Simple Span Non-composite Steel

What is a Bridge?

- Minnesota (Minnesota Rules 8810.8) Definition:
  - Erected over a depression or obstruction such as water, a highway or railway, etc.
  - Has a track or passageway for carrying traffic or other moving loads

What is a Bridge?

- Minnesota (Minnesota Rules 8810.8) Definition:
  - Has an opening measured horizontally along center of roadway of 10 feet or more
  - The opening is measured between undercopings of abutments, spring line of arches, or between extreme ends of openings for multiple boxes

What is Load Rating?

- The process of determining the safe live load carrying capacity of a new or an existing vehicular bridge is called its load rating.
- Note: load rating should not be confused with bridge condition rating which is entered into PONTIS.

What is Load Rating?

- The final load rating will be the rating of the weakest point of the weakest member within the bridge.

What is Load Rating?

- Live Loads are transient loads that remain in place for a relatively short time.
  - Cars
  - Busses
  - Trucks
  - Wind
  - Stream flow pressure
  - Etc
- Trucks usually used as Live Load for load rating of bridges

What is Load Rating?

- Live load carrying capacity is based on:
  - Bridge inspections
    - Description of any structural modifications
    - Identify any condition changes
    - Measurements of any losses
      - Decay
      - Corrosion
    - Field measurements

What is Load Rating?

- Live load carrying capacity is based on:
  - Engineering analysis
    - Applied engineering principles
    - Recognized formulae
    - Computer programs

What is Load Rating?

- Live load carrying capacity is based on:
  - Engineering judgment.
    - May be used to establish if substructure controls rating
    - May be used to establish if certain superstructure components will not control the rating

What is Load Rating?

- Live load carrying capacity is based on:
  - Nondestructive Load tests
    - Maybe used when bridge cannot be accurately modeled by analysis
    - Owner believes a load test will provide a more realistic load capacity

What is Load Rating?

- Load ratings may be subdivided into specific types depending on live load
  - Design load rating: The live load model is the AASHTO design HS loading. The rating is expressed relative to a HS20 truck.
  - Legal load rating: Sometimes called posting rating. The live load model is one or more of the "legal trucks".

What is Load Rating?

- Load ratings may be subdivided into specific types depending on live load
  - Annual Permit load rating: The live load model represents a truck or class of trucks that may operate under an annual overweight permit
  - Single Trip Permit load rating: The live load model is the specific overweight permit truck

What is Load Rating?

- Load rating is expressed as a rating factor (RF) or weight in tonnage for a particular live load.
  - Generally a RF ≥ 1.0 indicates that the bridge has adequate live load carrying capacity
  - Generally a RF < 1.0 indicates that the bridge has inadequate live load carrying capacity
  - A RF is always associated with a particular live load

Why do Ratings?

- Minnesota Administrative Rule 8810.95 Subp.2 Ratings.
  - Bridges must be rated to determine safe load carrying capacity and reported on a structure inventory sheet.
- FHWA National Bridge Inspection Standards (NBIS)
  - The MCE requires as part of every inspection cycle, bridge load ratings to be reviewed and updated

Why do Ratings?

- Bridge load ratings reported to the NBI weigh heavily in the determination of the Sufficiency Rating
- Used in preparing highway needs
- Used in prioritizing projects
- Used in distributing bridge funds to local governments

Why do Ratings?

- To have a consistent summary of load carrying capacities of all state bridges
- Bridge load ratings are used to post bridges and for issuing load permits
- To monitor safety of bridges over time
- To help determine when rehabilitation or replacement is needed

When to do Ratings?

- According to FHWA National Bridge Inspection Standards (NBIS):
  - Ratings should be done when there is any relevant changes in condition from:
    - Maintenance or improvement work
    - Strength of members
    - Dead load
  - Ratings should be reviewed and updated following bridge inspections

When to do Ratings?

- Minnesota Administrative Rule 8810.95 Subp.2 Ratings.
  - Bridge must be rerated when it is determined that a significant change has occurred in the condition of the bridge.
  - Bridge must be rerated when the allowable legal load using the bridge is increased.
  - Changes in the rating of a bridge must be indicated on the structure inventory sheet.

When to do Ratings?

- Increase in Dead Load
  - Bituminous Overlay
  - Gravel
  - Increase weight of railing
  - New deck
  - New beam or girder
- Significant changes in AASHTO specifications, state policies or federal directives.
- A change in law regulating truck weights

When to do Ratings?

- Loss of capacity
  - Corrosion or damage to steel structural elements resulting in section loss
  - Decay or damage to timber structural elements resulting in section loss
  - Spalling or salt related damages in concrete
  - Bridge hit by an oversize load

When to do Ratings?

- Settlement or movement of a pier or abutment
- Repairs or remodeling
- New bridges are rated before opened to traffic

What Code to Use?

- FHWA National Bridge Inspection Standards (NBIS)
  - Adopted the AASHTO Manual for Condition Evaluation of Bridges (MCE) as a NBIS regulation

What Code to Use?

- The AASHTO Manual for Condition Evaluation of Bridges (MCE)
  - MCE 2nd Edition with 2003 Interims
  - 2005 Guide Specifications for MCE and Load and Resistance Factor Rating (LRFR)
  - New MCE soon to be released, 2008?
    - Part A - LRFR
    - Part B - Allowable Stress Rating (ASR) & Load Factor Rating (LFR)

What Code to Use?

- Use AASHTO Bridge Design Specifications for all matters not covered in the MCE
  - 2002 AASHTO Standard Specifications
    - ASR
    - LFR
  - AASHTO Load Resistance Factor Design (LRFD) Specifications
    - LRFR

What Code to Use?

- Four rating methods are available:
  - Allowable Stress Rating (ASR)
  - Load Factor Rating (LFR)
  - Load and Resistance Factor Rating (LRFR)
  - Load Testing

Allowable Stress Rating (ASR)

- Uses actual (service) loads to produce maximum stress in member
- Maximum stress in member not to exceed the allowable stress
- Allowable stress is the limiting stress of the material with an appropriate factor of safety applied

Allowable Stress Rating (ASR)

- Per 2003 MCE 2nd Edition, timber and masonry bridges should be evaluated using ASR.

$$\text{Maximum stress under service load} \leq \text{Allowable Stress} = \frac{\text{Yield Stress}}{\text{Factor of Safety}}$$

Load Factor Rating (LFR)

- Uses separate load factors (or safety factors) applied to the actual loads
- The magnitude of load factors reflect the uncertainty in the load calculation
- The effect of the factored loads are not to exceed the strength of the member

Load Factor Rating (LFR)

- Per 2003 MCE 2nd Edition, steel, reinforced concrete, and prestressed concrete bridges should be evaluated using LFR.

(Strength Provided) × SF ≥ (Actual Loads) × SF

Note: Safety Factors (SF) are called strength reduction factors and load factors. The SF for load are typically 1.3 for dead load and 2.17 for live load. The SF for strength are typically .90 for flexural strength and .85 for shear strength

Load Factor Rating (LFR)

- LFR is the current standard for MnDOT
  - New bridges are designed using LRFD and are rated using LFR
  - Old rating forms not valid for steel and concrete (ASR)

Load & Resistance Factor Rating (LRFR)

- Similar to LFR, but uses statistically based load and resistance factors eliminating judgment and past experience to determine factors
- Load and resistance factors have been calibrated by trial designs to provide a uniform level of safety

Load & Resistance Factor Rating (LRFR)

- LRF method ensures that only 2 out of 10,000 bridges will have factored loads greater than the factored resistance of the bridge

(Strength Provided) × SF ≥ (Actual Loads) × SF

Note: LRFR Safety Factors (SF) for load and strength are statistically based to provide a uniform level of safety

Load Testing

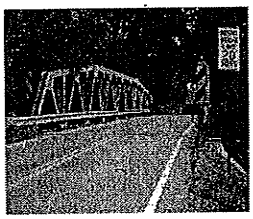
- Is an effective means of evaluating the load rating of a bridge
- Bridge load testing generally consists of:
  - Load evaluation
    - Tests on load and load effects from traffic
  - Diagnostic load testing
    - Test on effects of a known load on bridge
  - Proof load testing (should not be used)
    - Test to determine maximum live load bridge can support

Load Testing

- National Cooperative Highway Research Program (NCHRP) Project 12-28(13) developed a Manual for Bridge Rating Through Load Testing
- Manual covers:
  - General Load Testing Procedures
  - Load Test Equipment and Measurements
  - Diagnostic Load Tests
  - Proof Load Tests

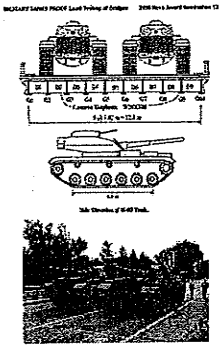
Diagnostic Load Testing

- Some general procedures:
  - Obtain strain responses from known load.
  - Structural behavior determined from strain data
  - Develop computer model
  - Simulate load test on computer model
  - Perform load rating on calibrated model



Proof Load Testing

- Using military tanks for proof load
  - Analytical methods can under estimate the actual strength
  - Carrying capacity can be larger due to unintended composite action, contribution from sidewalks, rail and etc..



FHWA Policy on Rating Methods

- Before October 1, 2010
  - New LRFD bridges and replacement bridges use LFR or LRFR
- After October 1, 2010
  - New LRFD bridges and replacement bridges use LRFR
- Bridges that warrant a rerating
  - Existing rating is LRFR, rerate using LRFR

FHWA Policy on Rating Methods

- Bridges that warrant a rerating (cont.)
  - No existing rating, use LRFR, LFR or ASR (for timber or masonry bridges)
  - Existing rating is LFR or ASR, rerate using LRFR, LFR or ASR (for timber or masonry bridges)
    - Non NHS bridges load rated prior to January 1, 1994, use LRFR, LFR or ASR
    - Non NHS bridges load rated after January 1, 1994, use LRFR, or LFR
  - Posting analysis can use ASR, LFR or LRFR (beyond 2010)

Mn/DOT Policy on Rating Methods

- Currently
  - Design new bridges and replacement bridges using LRFD and rate LFR, but allow ASR for masonry and timber bridges
- 2008 - Mn/DOT to complete studies of LRFR

Mn/DOT Policy on Rating Methods

- 2009 - Mn/DOT and NHI provide LRFR training
- 2010 - Mn/DOT to meet FHWA policy on LRFR
- 2011 + - Rerate all bridges using LRFR

Who is Qualified?

- The person charged with overall responsibility for load-rating bridges shall have:
  - Professional Engineer License
  - 5-years bridge design and inspection experience
  - Knowledge and skills for proper evaluation

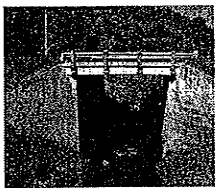
Who is Qualified?

- The PE's signature of approval on the rating form holds the same level of responsibility as the Engineer's signature of approval on the bridge plans and specifications

Some Ratings Are Not Current

More bridges are vulnerable

- Of 14,921 Local Bridges and Culverts:
  - 2,301 have not been rated for 30 years
    - Average operating rating of HS 26 or lower
  - Median age = 65 years
    - Many may need to be re-rated
  - Of 765 that are posted
    - 190 (25%) Have Not Been Rated in 30 Years and Postings May Need to be lowered



Ideal Load Rating History

- Bridge Built
  - Rated at Design level
- Gradual Corrosion
  - Rating gradually lowered as section reduces
- Severe Deterioration
  - Posting at appropriate level
- Replacement
  - Cycle starts over again

Statewide Bridge Rating Program Needs

- Office of Inspector General requires FHWA to better monitor load ratings
- Consultant inspection findings often are recommending new bridge ratings
- Likely to have less posted load limits based only on engineering judgment

Statewide Bridge Rating Program Needs

- New ratings are required for significant changes in condition or loading since last inspection
- Continued pressures from truckers may require statewide permitting, a one stop shop for all load permits in the future
- AASHTO introduced the new Special Hauling Vehicle (SHV) posting vehicle

Statewide Rating Funding Concept

- Use federal HBRRP funds to pay for bridge ratings on the local level
- Possible Priorities:
  - First, bridges with changed conditions
  - Second, bridges with possible SHV limitations

Statewide Rating Funding Concept

- Possible Priorities:
  - Third, new bridge plans (by 2011, all new bridges must be rated under LRFR)
  - Fourth, rerate the remaining bridges, county roads followed by township roads
- If County Engineers support it, the Bridge Office and State Aid could pursue.

Truck Trends

- Trucks are getting heavier and increased allowable limits have created greater:
  - Agency costs for inspecting
  - Agency costs for rating
  - Agency costs for posting

Truck Trends

- Timber Haulers:
  - Allowed a weight exception to 90,000 GVW for combination vehicles with 6 axles
  - For hauling forest products
  - Annual permit of $300
  - Winter weight increase to 98,000 GVW

Truck Trends

- Truck Size and Weight Committee
  - Review of truck weight laws was needed:
    - Truck traffic increasing
  - Recommendations:
    - A request for truck weight increases
      - 7 axle 97,000 GVW combination vehicle

Truck Trends

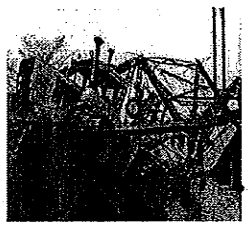
- Truck Size and Weight Committee
  - Recommendations:
    - Allow 4 new vehicle configurations with special permit
      - 6 axle 90,000 GVW combination vehicle
      - 7 axle 97,000 GVW combination vehicle
      - 8 axle 108,000 GVW twin trailer vehicle
      - 7 axle 80,000 GVW single-unit truck

Truck Trends

- Truck Size and Weight Committee
  - Recommendations:
    - Eliminate liability exemptions for farm implements that damage roads and bridges
    - Eliminate requirements for seasonal harvest permits (permit allows 10% load increase)
    - Expand seasonal harvest allowance to include all farm crops

Truck Trends

- Implements of Husbandry:
  - Agricultural equipment exempt from weight restrictions
    - Grain carts (used in the fall)
    - Manure wagons (used in the spring)
    - They have no axle restrictions, can carry same load as 6 axle truck on only 2 axles



Truck Trends

- Increased overweight permits (truck load exceeds legal weight)
  - A practice that is putting additional wear and tear on our infrastructure
  - According to a government study, one 40-ton truck does as much damage to the road as 9,600 cars

Truck Trends

- Increase high stress cycles
  - Higher loads create larger fatigue stresses which can result in fatigue cracking of materials
  - High stress cycles can lead to requirements of posting, maintenance, and/or expensive repairs

Bridge Rating & Related Resources

- Mn/DOT Bridge Office Rating Section:
  - Gary Peterson, 651-366-4507
  - Lowell Johnson, 651-366-4552
- Mn/DOT Bridge Office Website:
  - http://www.dot.state.mn.us/bridge/

Bridge Rating & Related Resources

- Overdimension/Overweight Permit Section
  - http://www.dot.state.mn.us/motorcarrier
- Seasonal Load Limits
  - http://www.mrr.dot.state.mn.us

Minnesota Rules, Table of Chapters

Table of contents for Chapter 8810

8810.8000 DEFINITIONS.

Subpart 1. **Abandonment.** "Abandonment" means the abandoning of an existing bridge.

Subp. 1a. **Agency.** "Agency" means a county, municipality, or township.

Subp. 2. **Bridge.** "Bridge" is defined as a structure including supports erected over a depression or an obstruction such as water, highway, or railway, having a track or passageway for carrying traffic or other moving loads, and having an opening measured horizontally along the center of the roadway of ten feet or more between undercopings of abutments, between spring line of arches, or between extreme ends of openings for multiple boxes. Bridge also includes multiple pipes where the clear distance between openings is less than half of the smaller contiguous opening. Illustrations of measurements are shown as Figures 1 to 4. This definition of a bridge includes, for application of parts 8810.8000 to 8810.8500, only those railroad bridges over or under a public highway or street.

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Subp. 3. **Commissioner.** "Commissioner" means the commissioner of transportation.

Subp. 4. **Construction.** "Construction" means the construction of a road, street, or bridge to replace an existing deficient bridge or a bridge that has been destroyed.

Subp. 5. **Metropolitan council.** "Metropolitan council" means a commission established in accordance with Minnesota Statutes, chapter 473.

Subp. 6. **Reconstruction.** "Reconstruction" means the reconstruction of an existing deficient bridge.

Subp. 7. **Regional development commission.** "Regional development commission" means a commission established by the state planning officer in accordance with Minnesota Statutes, chapter 462.

Subp. 8. **Road systems defined.** "Other roads or streets" means those local roads and streets, not on the state-aid system, under the jurisdiction of the counties, municipalities, and townships.

"State-aid" means the system of roads and streets established and designated by the commissioner under the jurisdiction of a county or urban municipality.

"Trunk highway" means the system of routes established by

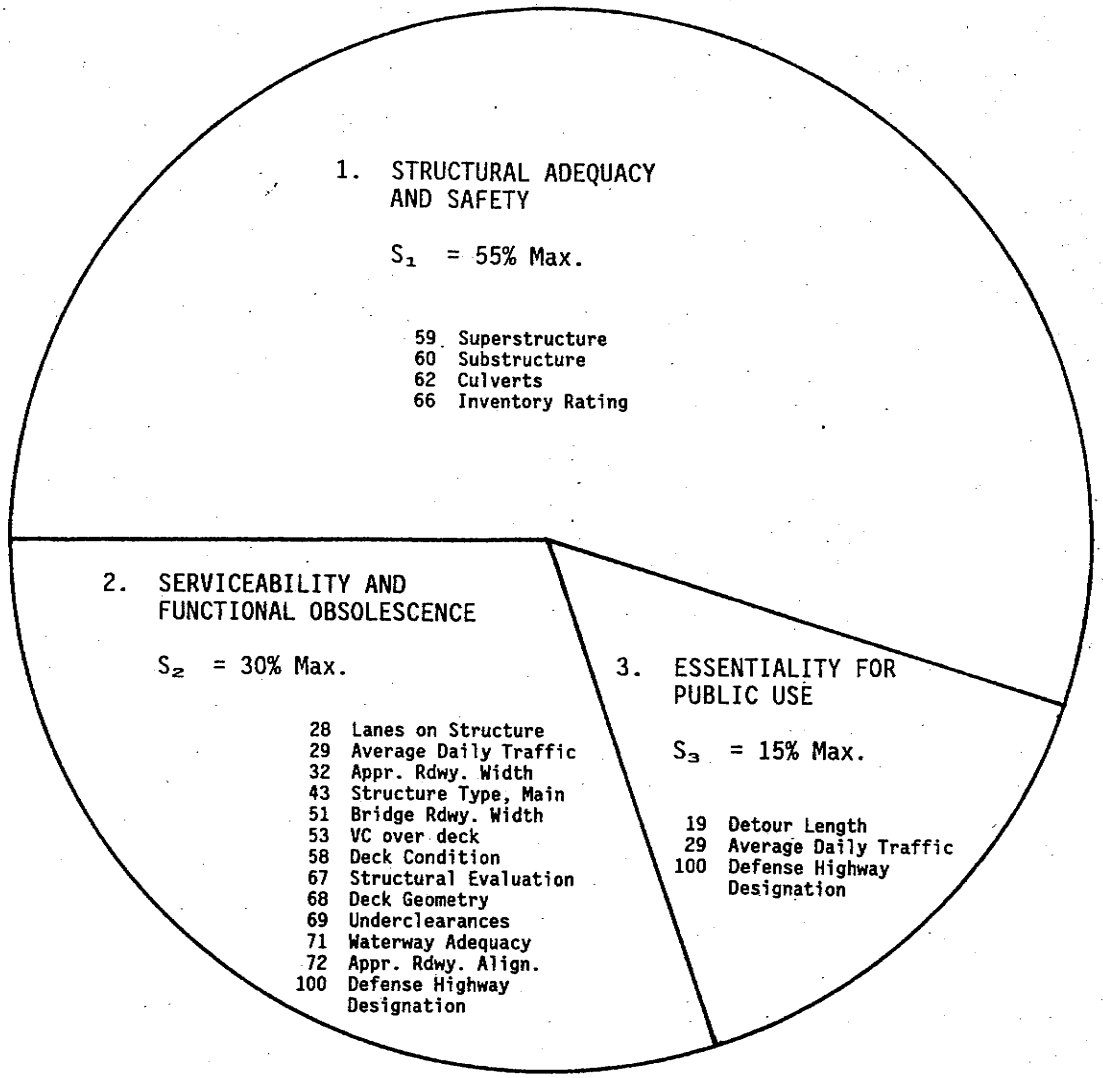


Figure 1. Summary of sufficiency rating factors

Sufficiency Rating Formula

1. Structural Adequacy and Safety (55% maximum)

(a) Only the lowest code of Item 59, 60, or 62 applies.

If Item 59 (Superstructure Rating) or Item 60 (Substructure Rating) is:	≤ 2	then	A = 55%
	= 3		A = 40%
	= 4		A = 25%
	= 5		A = 10%
If Item 59 and 60 = N and Item 62 (Culvert Rating) is:	≤ 2	then	A = 55%
	= 3		A = 40%
	= 4		A = 25%
	= 5		A = 10%

(b) Reduction for Load Capacity

Calculate using the following formulas where IR is the Inventory Rating (MS Loading) in tons or use Figure 2:

MS = METRIC TONS
HS = ENGLISH TONS
MS18 = HS20

$$B = (32.4 - IR)^{1.5} \times 0.3254$$

or

If $(32.4 - IR) \leq 0$, then B = 0

"B" shall not be less than 0% nor greater than 55%.

S1 = 55 – (A + B)

S1 shall not be less than 0% nor greater than 55%.

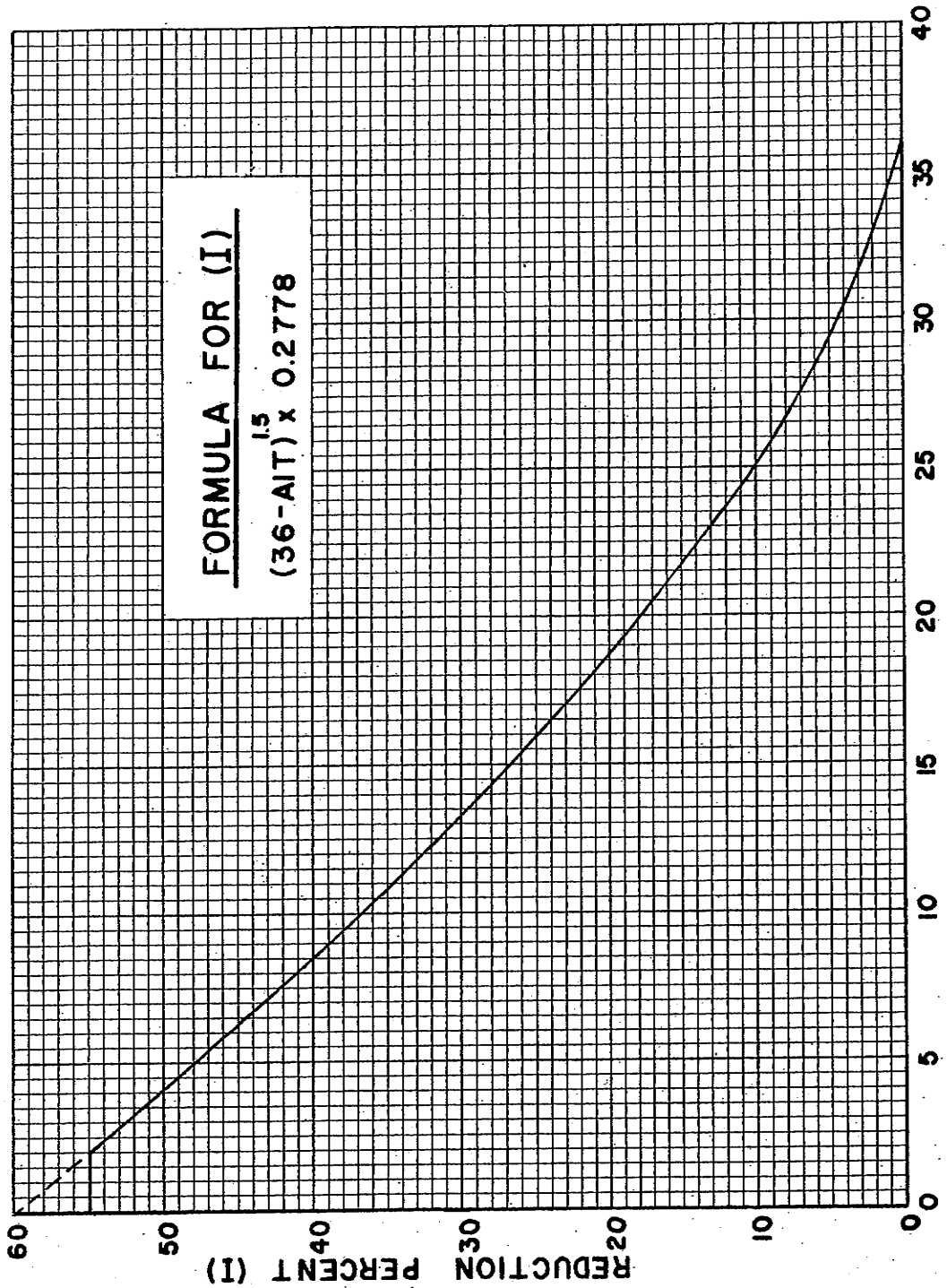


Figure 2. Reduction for adjusted inventory tons

TS&W Report June, 2006

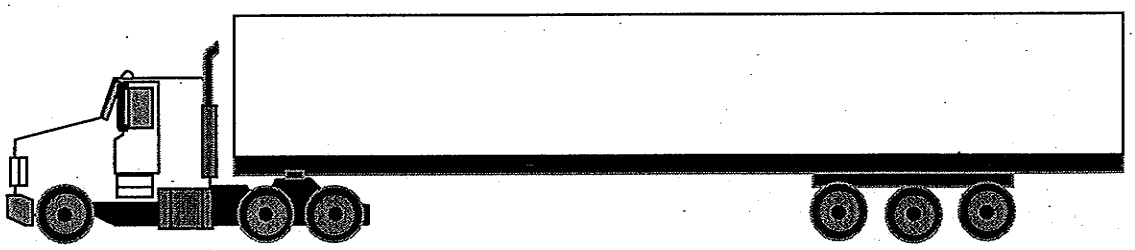
Recommendations

Based on technical analysis of alternative changes to TS&W laws, as well as the input from the outreach process, advisory committee feedback, the Northstar Workshop, and Departmental deliberations, the following TS&W proposals were recommended and advanced by Mn/DOT for legislative consideration consistent with the policy principles adopted for this study. These recommendations were developed through an extensive technical and policy evaluation process. The analysis has followed nationally accepted methodology used by the NAS/TRB TS&W studies and the U.S. DOT/FHWA *Comprehensive Truck Size and Weight Study*. The findings also have been peer reviewed by national and international experts in TS&W policy, infrastructure impacts, and safety analysis.

The final list of recommendations represents a balanced approach that protects highway infrastructure and safety while providing industry productivity improvements that will benefit Minnesota's economy and competitiveness. The vehicle configurations are recommended for operation under special permit with appropriate fees to recover additional administration, enforcement, and infrastructure costs. Special safety requirements also are recommended as deemed appropriate to the specific configuration based on a comprehensive review of each vehicle against accepted international performance standards. Finally, changes to SLR and other TS&W policy issues are recommended to be considered in parallel to potential vehicle configuration weight increases.

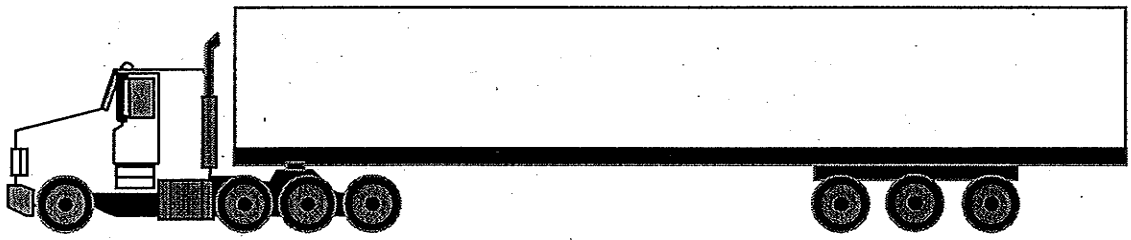
Allow Six-Axle 90,000 Pounds Gross Vehicle Weight on Non-Interstate 10-Ton Network

- Must meet bridge formula, axle, and tire weight limits.
- Maximum trailer length of 53 feet (no change).
- Winter and seasonal increases to 99,000 pounds GVW, but no additional tolerances or exemptions allowed beyond 99,000 pounds.
- Allowed on 10,000-mile 10-Ton Network (non-Interstate); except sealed intermodal containers are allowed on Interstates.
- Similar to timber haulers truck enacted by the Minnesota legislature in 2004 and livestock truck enacted in 2005 (effective 2006).
- Requirements: special permits with fees; axles to be added by certified remanufacturer; and brakes required on every wheel.



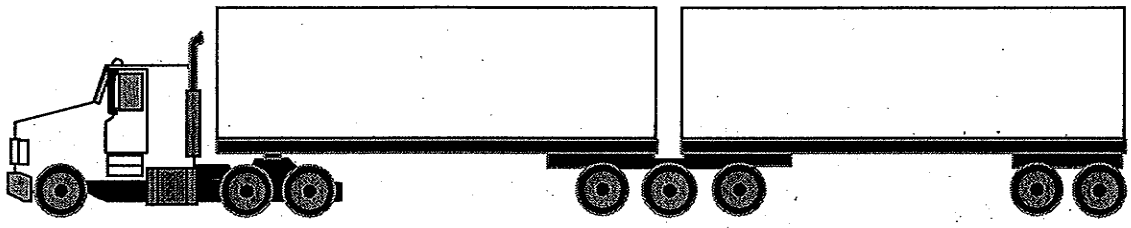
Allow Seven-Axle 97,000 Pounds Gross Vehicle Weight Non-Interstate 10-Ton Network

- Must meet axle, bridge formula, and tire weight limits.
- Maximum trailer length of 53 feet (no change).
- Winter and harvest increases to 99,000 pounds GVW, but no additional tolerances or exemptions allowed beyond 99,000 pounds.
- Allowed on 10,000-mile 10-Ton Network (non-Interstate).
- Requirements: permits with fees; axles to be added by certified remanufacturer; brakes on every wheel; and subject to Federal Motor Carrier Safety Administration (FMCSA) regulations under Title 49, Part 391.



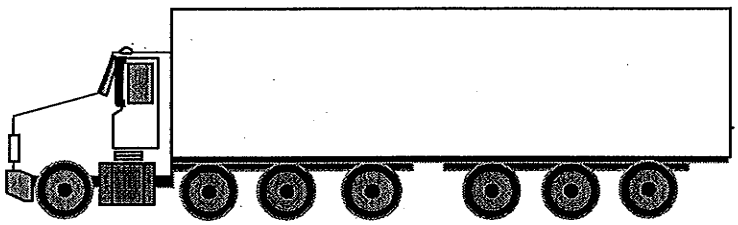
Allow Eight-Axle 108,000-Pound Twin Trailer on Non-Interstate Minnesota Twin Trailer Network and National Truck Network

- Must meet axle, bridge formula, and tire weight limits.
- Maximum trailer length of 28.5 feet each (no change).
- Allowed on pre-approved state trunk highway routes only (Minnesota Twin Trailer Network and National Truck Network) – approximately 6,700 miles.
- No harvest or winter increases nor any other tolerances or exemptions beyond 108,000 pounds.
- Same as paper products truck enacted in 2005 (effective 2006).
- Requirements: permits with fees; B-train coupling; axles to be added by certified remanufacturer; brakes on every wheel; Commercial Driver's License (CDL) endorsement for multitrailer operations; and subject to FMCSA regulations.



Allow 80,000 Pounds Gross Vehicle Weight Single-Unit Truck on 10-Ton Network (Including Interstate)

- Must meet axle, bridge formula, and tire weight limits.
- Vehicle length increase up to 45 feet maximum (from current 40 feet).
- Lift axles must be down with loads.
- Axles in excess of 4 must be steering axles/castering axles
- Requirements: Permits with fees; axles to be added by certified remanufacturer; and brakes on every wheel.



Change Spring Load Restrictions

Based on extensive input and discussion regarding the economic impact of SLR in Minnesota, the following changes are recommended:

- All county roads default to seven tons per axle rather than to the current five tons unless posted otherwise.
- State trunk highways remain at 10 tons per axle unless posted otherwise.
- City streets and township roads continue to default to five tons per axle unless posted otherwise.
- SLR for gravel roads ends two weeks later than paved roads.

Other Legislative Policy Issues to be Considered

Following is a list of other legislative policy issues to be considered parallel to potential weight increases. These issues are closely linked to the above-recommended TS&W legislative changes. Other policy issues for future consideration are included in the appendices.

- Eliminate liability exemptions for farm implements that damage roads or bridges;
- Remove the 73,280-pound GVW limit for five-axle semitrailers on 9-ton roads and allow axle weights and the bridge formula to control up to 80,000 pounds GVW;

18.4.5 Instrumentation Plan

The rating engineer must provide an instrumentation plan, which shows instrumentation recommendations and requirements for the load test. Final approval of the plan, however, is left to the discretion of the testing engineer who must determine whether it is feasible to place instruments at the indicated locations. The testing engineer may relocate, change or eliminate unnecessary or improperly placed instruments. Since the majority of the instrumentation is electronic, it is necessary to route wires from the instrument to the data acquisition system. The wires must be located so they will not interfere with or be damaged by traffic.

18.4.6 Test Procedure

Two test procedures are available for load testing. Proper selection of a procedure is critical for accurate rating.

18.4.6.1 Static Testing

Static testing is conducted by incrementally placing calibrated weights on test vehicles, then locating the vehicles at positions on the structure that create maximum stresses. Load may be applied at more than one point at the same step, especially in the case of continuous bridges, or bridges with various shear or fracture-critical components. Since large structures typically require many load increments, there may be a great deal of time between the first and last load application. Therefore, it may be necessary to consider temperature effects in the analysis of the test data.

Extreme caution must be used when performing static loading, in that sudden distress of the components may occur between load increments. It is the responsibility of the testing engineer to stop the test at the first sign of non-linear behavior in the structure.

18.4.6.2 Dynamic Testing

Dynamic testing is used primarily to determine a more accurate value for the impact factor to use in load rating. It is also used to determine the natural frequency of vibration of the bridge. Dynamic testing is performed by loading a test vehicle with weight and then driving the vehicle over the bridge at a known velocity. In order to obtain a complete envelope of the dynamic effects, runs are made using various speeds, lane locations, and weights.

18.4.7 Analysis of Test Data

During the test, the testing engineer must compare the structural response with the analytical data to ensure that the response remains in the linear-elastic range. After testing, the testing engineer will reduce the data and place it in a standard format so that the rating engineer can assess the differences between the measured and analytical results. Through data analysis, the testing engineer will eliminate environmental (temperature) and electrical effects (noise) so that only pertinent results are presented.

18.4.8 Verification of Analytical Model

It is the responsibility of the rating engineer to:

- determine the adequacy of the analytical model
- identify the potential causes of differences
- make necessary adjustments

Sound reasoning must be used to explain the differences so that future analysis and rating can be performed accurately.

18.4 RATING BY LOAD TESTING

18.4.1 Condition Assessment

Before deciding to use load testing to establish a bridge's load rating, a strength evaluation should be performed as previously discussed. Load testing is usually recommended if a bridge receives an unsatisfactory rating from this evaluation. The recommendation is also conditional on the underlying cause of the low rating. Typical reasons for recommending load testing may be physical damage, questionable or insufficient "as-built" plans, or any physical characteristic which affects performance but cannot be adequately accounted for in the strength evaluation.

A structure should not be load tested solely because it received an insufficient rating for a special permit vehicle. Prior to conducting a load test in these cases, the analysis method used to rate the structure must be examined. If a simplified procedure was used, a refined evaluation must be made. If this evaluation also results in a deficiency, and there are no alternative truck routes, a load test may be deemed necessary.

Before load testing a structure considered insufficient due only to age, a comprehensive on-site inspection must be performed. The inspection must consider the current state of the bridge components, and the basis which makes it currently obsolete. Based on these factors, the engineer then determines if the bridge requires load testing.

Prior to conducting a load test, all possible ramifications must be considered. Any conditions that would cause the load testing to be a hazard to either the public or the individuals conducting the test, must be reconciled prior to the testing. Modifications such as testing at night or limiting the scope of the testing may be required.

18.4.2 Types of Load Tests

There are two types of tests: proof load and diagnostic. Proof load testing is typically recommended when the bridge exhibits signs of distress such as corrosion or damage which cannot be accurately quantified, when it is suspected that the "as-built" bridge is different from existing plans, or for any other reason that hinders determining the bridge capacity. Diagnostic loading is typically used to compare actual bridge response to analytical values, based on presumably accurate information regarding the current conditions.

18.4.2.1 Proof Load Test

During proof load testing, the response of the bridge is monitored while the load is applied incrementally up to a level equal to the target rating (usually the factored live load). Testing is stopped if the measured response becomes non-linear. For example, if the target inventory rating is HS20, the bridge is loaded up to a target load of (2.17)(1+I)(HS20 Loading). The coefficient 2.17 is the live load factor according to the *Standard Specifications*. Theoretically, this method does not rely on bridge analysis; it relies solely on field test data to determine the load capacity of the bridge. The rating established is the lower bound of the actual load capacity.

Because the bridge is loaded to the ultimate live loading which is 2.17 times the standard truck loading, precautionary measures are necessary to minimize the risk of damage to the bridge. In order to ensure safety, at every load increment the measured data is compared with predicted values before proceeding to the following load increment.

Workshop Agenda

I. Introductions
II. Load Rating Basics
III. General Equations
IV. Load Rating Procedure
V. Incorporating Member Distress
VI. Posting, SHV's and Permitting
VII. Load Rating Example #1
   - Simple Span Non-composite Steel

General Equations

- Per the AASHTO Manual for Condition Evaluation of Bridges (MCE), the following general expression should be used in determining the load rating of a structure:

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

General Equations

Reserve Capacity for Live Load

Maximum Design (HS) or Legal Live Load

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

RF = Rating factor for live load capacity
C = Capacity of the member
D = Dead load effect on member
L = Live load effect on member
I = Impact Factor
A_1 = Factor for dead load
A_2 = Factor for live load

General Equations

- RF = Rating Factor for live load capacity
  - The Rating Factor (RF) is useful in understanding the factor of safety on the bridge.
  - The Rating Factor multiplied by the rating vehicle in tons gives the rating of the structure

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

General Equations

- C = Capacity of the member
  - The nominal capacity to be used in the rating equation depends on:
    - Structural materials – Timber, concrete, or steel
    - Rating method used – ASR, LFR, or LRFR
    - Rating Level used – Inventory or Operating

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

General Equations

- C = Capacity of the member
  - The units used for "C" depend on the member and "load effect" being checked.
  - The "load effect" is the effect of the applied load on the member
  - Typical "load effects" are axial force, vertical shear force, bending moment, axial stress, shear stress and bending stresses

Capacity of the members

- Timber (using ASR method)
  - F_B, F_V, F_C = Allowable stresses in the wood
    - Tables provided for all species, sizes, and grades of wood in AASHTO Standard Specs.
  - Timber Adjustment Factors
    - C_D = Load duration factor
    - C_F = Bending Size factor
    - C_M = Wet service factor
    - C_r = Repetitive member factor
    - C_f = Bending form factor
    - C_P = Column Stability factor

Capacity of the members

- Timber (using ASR method)
  - Inventory Level
    - $C = F_x' = F_x * C_D * C_F * C_M * C_r * C_f * C_P$
  - Operating Level
    - $C = 1.33 \times F_x'$ (Reduction from this maximum allowable stress may be warranted based on grade and condition at time of inspection)

Capacity of the members

- Concrete and Steel
  - Inventory and Operating Level (using ASR method)
    - In a FHWA Tech Memo they disallowed ASR procedure
  - Inventory and Operating Level (using LFR method)
    - $C = \phi M_n$ or ϕV_n (Found based on procedures in AASHTO Standard Specifications for Highway Bridges)
    - $\phi = 1.0$ for Steel
    - $\phi = 0.9$ for Concrete flexure
    - $\phi = 0.85$ for Concrete shear

Capacity of the members

- Steel – If F_y is unknown, use the following table for yield strength:

Year of Construction	Yield Point F_y (ksi)
Before 1905	26
1905-1936	30
1937-1963	33
1964-1989	36
1989-present	50

General Equations

- D = Dead load effect on member
  - Dead loads include self weight and any permanent external loads
  - Should be computed in accordance with the existing conditions of the member at the time of analysis

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

General Equations

- L = Live load effect on member
  - Live loads are temporary loads that act on a structure
  - Live loads can be due to
    - Design truck (HS) and lane loading
    - Legal Trucks, Special Hauling Vehicles (SHV's)
    - Permit trucks

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

General Equations

- L = Live load effect on member
  - For simple span bridges, live loads can typically be calculated using moment and shear tables provided
  - Live loads should be distributed to member in accordance with AASHTO Design Specs.

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

General Equations

- I = Impact Factor
  - Impact Factor (I) is added to all live loads to account for the speed, vibration, and momentum of vehicular traffic.

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

General Equations

- I = Impact Factor
  - The AASHTO specifications for bridge design define the impact factor as follows:

$$I = \frac{50}{(L + 125)} \leq 0.3$$ (L = Length of span)

  - Per AASHTO 3.8.1.2 – Impact is not considered for Timber members

General Equations

- A_1 = Factor for dead load
  - Allowable Stress Rating Method (ASR)
    - A_1 = 1.0 (Inventory & Operating Level)
  - Load Factor Rating Method (LFR)
    - A_1 = 1.3 (Inventory & Operating Level)

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

General Equations

- A_2 = Factor for live load
  - Allowable Stress Rating Method (ASR)
    - A_2 = 1.0 (Inventory & Operating Level)
  - Load Factor Rating Method (LFR)
    - A_2 = 2.17 (Inventory Level)
    - A_2 = 1.3 (Operating Level)

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

Load Rating Calculations (LFR)

- There are two levels of load ratings: (Established through different values of A_1 and A_2)
  - Inventory
  - Operating

Load Rating Calculations (LFR)

- Inventory
  - Design level of stresses
  - Incorporates existing conditions
  - Results in a live load which can safely utilize structure for an indefinite period of time

(HS-20 Truck)

$$RF = \frac{C - 1.3D}{2.17(L + I)}$$

Load Rating Calculations (LFR)

- Operating
  - Describes maximum permissible live load for bridge
  - Limited load applications
  - Loading at this level may shorten life of bridge
  - Used for Posting and permit level rating

$$RF = \frac{C - 1.3D}{1.3(L + I)}$$

Load Rating Calculations (ASR)

- The two levels of load ratings, Inventory and Operating, are established through different values of "C" or Capacity of member.

$$RF = \frac{C - 1.0D}{1.0(L + I)}$$

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Load Rating Procedure

- Gather Bridge Data
- Collect Information From On-site Inspections
- Determine Applied Loads
- Determine Capacity of Member
- Conduct Load Rating Calculations
- Submit Results to MnDOT

Gather Bridge Data

- Bridge design plans
- Bridge fabrication drawings
- Design calculations
- All repair plans for structure
- Most recent Structure Inventory Report

Collect info from on-site inspections

- Changed conditions
  - Damage
  - Repairs
  - Section loss
  - Traffic

Bridge Inspection

- When design plans or fabrication drawings for a bridge are unavailable or unreadable, or when conditions change from those detailed in the plans, field measurements are required.
- Field measurements should be made only with sufficient precision to serve the purpose for which they are intended.

Bridge Inspection

- The following limits of accuracy are generally ample for field measurements:
  - Timber members - Nearest ¼"
  - Concrete members - Nearest ½"
  - Asphalt surfacing - Nearest ½"
  - Steel rolled sections
    - Necessary accuracy to identify section
    - More precise measurements warranted for determination of section loss
  - Span Lengths - Nearest 0.1 foot

General Equation

Reserve Capacity for Live Load

Maximum Design (HS) or Legal Live Load

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

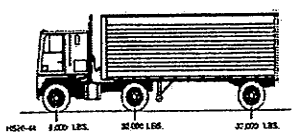
RF = Rating factor for live load capacity
C = Capacity of the member
D = Dead load effect on member
L = Live load effect on member
I = Impact Factor
A_1 = Factor for dead load
A_2 = Factor for live load

Determine the Loads

- Dead Loads
  - Compute according to existing conditions
    - Point loads – pilasters, lighting, diaphragms. etc.
    - Line loads – Beams, stool, etc.
    - Distributed loads – Slab, sidewalk, railings, overlay, gravel, etc.
    - Material unit weight must be at least the value specified in the AASHTO Design Specs.

Determine the Loads

- Live Loads
  - AASHTO HS20 Truck Live Load



Determine the Loads

- Live Loads
  - Standard AASHTO HS lane loads may be used for all span lengths where it will result in greater effects than the standard HS truck (simple span greater than 140')
  - Legal Trucks
    - (Type M3, Type M3S2, Type M3S3 and new Single Unit (SU) trucks
  - Permit Trucks

Determine the Loads

- Live Loads
  - Number of loaded lanes and transverse placement of wheels shall be in accordance with AASHTO Design Specs (Section 3).
  - Roadway widths 18-20 ft, 2 design lanes, each half the width, centered live load
  - Widths less than 18 ft, 1 design lane

Determine the Loads

- Live Loads
  - For vehicle load distribution (Consult Section 3 of AASHTO Standard Specs)
    - Steel Beams
    - Concrete beams
    - Concrete Slabs
    - Longitudinal and Transverse Timber Deck
    - Floor Beams

Impact

- Impact Factor (I) is added to all live loads to account for the speed, vibration, and momentum of vehicular traffic.
- Per AASHTO 3.8.1.2 – Impact not considered for Timber members

Determine Capacity of Member

- Nominal capacity based on Load Factor section of AASHTO Standard Specs 17th ed.
  - Structural steel
  - Reinforced concrete
  - Prestressed concrete

Determine Capacity of Member

- Load Factor methods for timber and masonry are not available – Use ASR
- MUST include the affects of deteriorated or damaged sections

Determine Capacity of Member

- Calculate section properties
  - Incorporate distress
  - Composite properties
  - Non-composite properties

Select Safety Factors

- Select Factors for Rating Method used:
  - Allowable Stress Rating Method (ASR)
  - Load Factor Rating Method (LFR)
  - Load and Resistance Factor Rating Method (LRFR)

Conduct Live Load Analysis

- AASHTO Manual for Condition Evaluation of Bridges Tables (Appendix)
  - Simple spans
    - HS-20 truck
    - Legal trucks
  - Continuous spans need computer program

Conduct Load Rating Calculations

- Inventory Rating *(frequent loads)*
  - Load the bridge can carry for extended periods
  - Design Load (live load for which bridge was originally intended)

Conduct Load Rating Calculations

- Operating Rating *(less frequent loads)*
  - Absolute maximum permissible load.
  - Unlimited vehicles operating at this level may reduce bridge life.

Conduct Load Rating Calculations

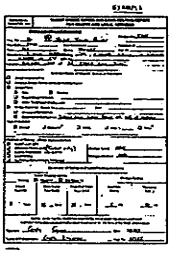
- Typically the superstructure is the only component rated
- If other portions of the bridge system are deteriorated, they should be fully analyzed and considered in the load rating calculations.

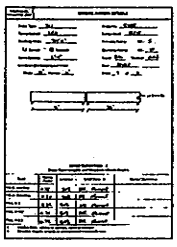
Rating Members

- Typically not checked
  - Splices or connections
  - Fatigue
  - Concrete deck
  - Local failure (bearing, yielding)
  - Shear for slabs
  - Substructure (bearings, piles, movement)
  - Secondary members (diaphragms, wind bracing)

Submit Results to MnDOT

Form RC-CL Mn/DOT Bridge Rating and Load Posting Report



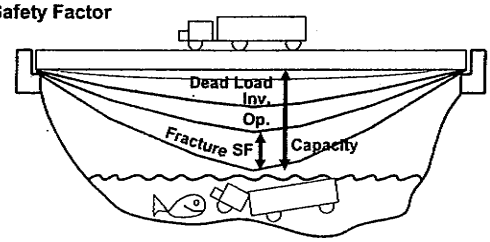


Bridge Management
Attn: Jim Pierce
3485 Hadley Ave N
Oakdale MN 55128

Bridge Loads by Deflection Extra Credit!!

Inventory deflection
Operating deflection
Fracture load
Safety Factor

Capacity = DL+ Fracture LL



2008
Bridge Load Rating Class 101

Workshop Agenda

I. Introductions
II. Load Rating Basics
III. General Equations
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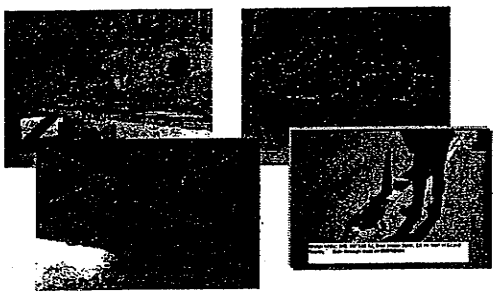
Incorporating Member Distress



It is EXTREMELY important to consider distress!

$$\text{Rating} = \frac{\text{Capacity} - \text{Dead Load}}{\text{Live Load}}$$

What is Causing Distress?

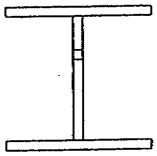
- Bridge hit
- Corrosion or rot
  - How much and can you stop?
  - Critical location??
- Thermal movement
- Serviceability problem
- Scour
- Illegal overload

Condition of Bridge

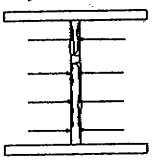
- Bridge ratings should be based on a recent and thorough bridge inspection
- Adequate data should be available to evaluate the affect of damage.
  - Description
  - Location
  - Extents

Inspection Requirements

Incorrect Report
1" diameter section loss in web



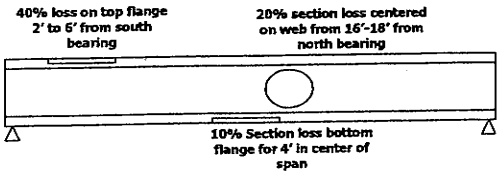
Correct Report
Original t_w=5/16"
Average thickness in 4 places is 0.15"



Inspection Requirements

Incomplete Report
40% Section loss on beams

Accurate Report



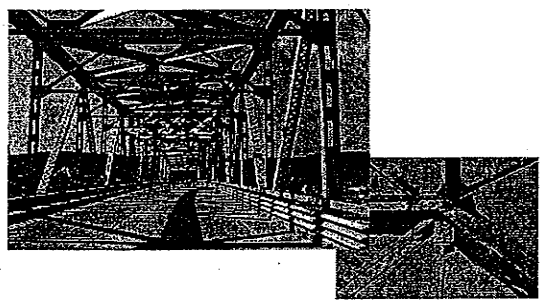
High Stress Areas (Beams)

- Moment
  - Beam flanges or primary tension rebar in center span (continuous at piers for -M)
- Shear
  - Webs near support points
- Bearing
  - At support locations, integrity of flanges and webs

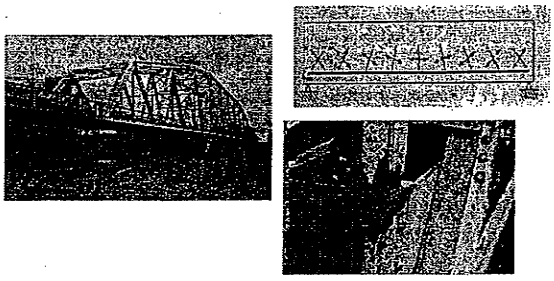
High Stress Areas

- Compression
  - Top flange in simple span
  - Find in truss (lattice bracing)
  - Make sure braced properly (LTB)
- Tension
  - Bottom flange simple span
  - Look at net section (Ag-bolt holes)
  - Find in truss

Trusses



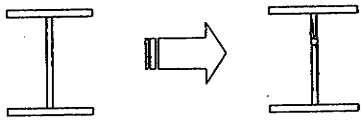
Critical Regions



How to Incorporate Distress

- Change cross-sectional properties to reflect loss of section
  - Concrete sections
    - Reduce area of reinforcement
    - Loss of concrete area in compression
  - Steel or Timber axial member
    - Tension reduce area (net section)
    - Compression reduce area (loss of bracing)
  - Steel or Timber beam
    - Moment of inertia

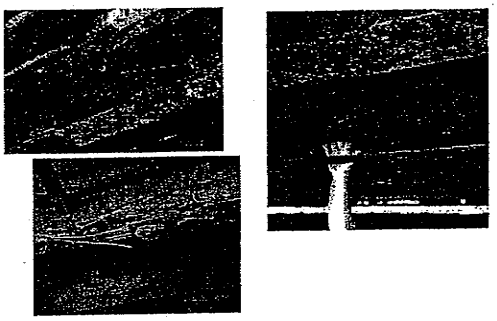
Structural Distress Example



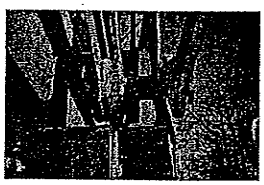
$$A_{rating} = A_{design} - A_{distress}$$

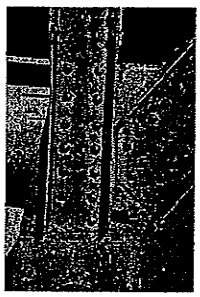
$$I_{rating} = I_{design} - (I_{distress} + A_{distress} d^2)$$

Structural Distress Example

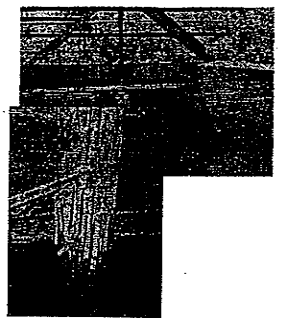


Structural Distress Example

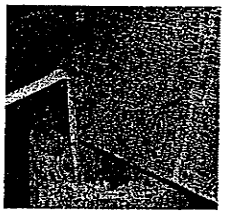




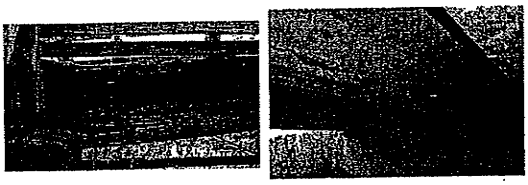
Structural Distress Example



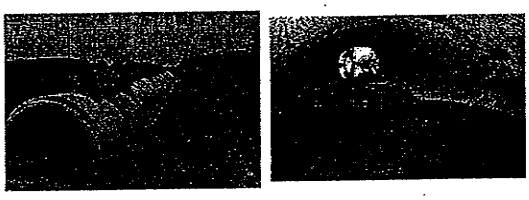
Structural Distress Example



Structural Distress Example



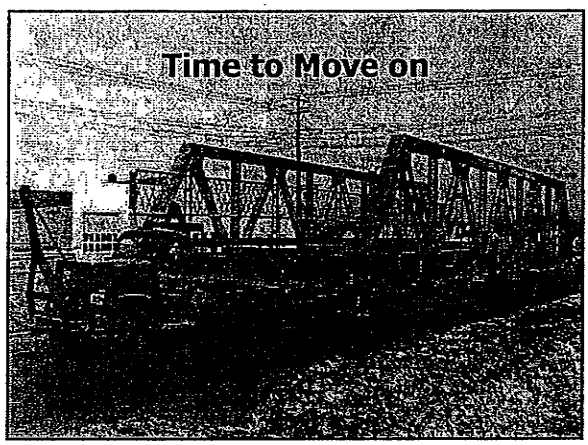
Structural Distress Example



Repairs

Impact of Distress at Critical Location

Rating	Original	2" Overlay	Steel I Beam 20% Section Loss
Inventory	HS11.6	HS10.8	HS5.0
Operating	HS19.4	HS18.1	HS10.0

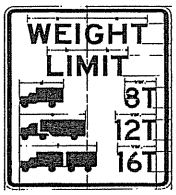


2008
Bridge Load Rating
Class 101

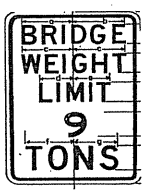
Workshop Agenda

I. Introductions
II. Load Rating Basics
III. General Equations
IV. Load Rating Procedure
V. Incorporating Member Distress
VI. Posting, SHV's and Permitting
VII. Load Rating Example #1
   - Simple Span Non-composite Steel

Bridge Posting



R12-5



R12-1A

Permit Sign

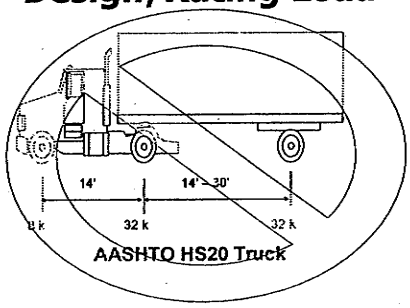
Maximum Load on Bridge

Posting Load Restrictions

- The posted limit is the total weight on the respective truck which will produce max allowable load effect.
- Different than maximum capacity of bridge (Proof Load). The bridge should not fall down or have excessive deflections.
  - Safety Factor 1.3
  - Conservative modelling assumptions
  - Ductility
  - Inelastic behavior

Posting Load

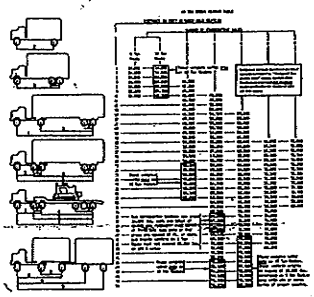
Design/Rating Load



Legal Load (Formula B)

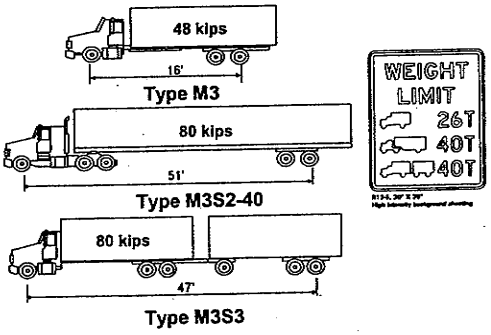
Legal loads per Statute 169.825. Legal loads are defined by chart with length, corresponding weights and number of axles

10% Winter increase



Posting Load

- **Three trucks used to represent all the combinations of legal loads**



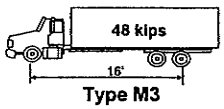
Posting Load Calculation

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

RF = Rating factor for live load capacity
C = Capacity of the member
D = Dead load on member
L = Live load on member
I = Impact Factor
A_1 = Factor for dead load
A_2 = Factor for live load

Use the same equation and values used to conduct the load rating. Substitute the posting trucks for the HS20 live load. Multiply the result by the weight of the truck to get the maximum allowable weight for that particular truck classification.

Simple Posting Example



Type M3

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

$$0.5 = \frac{100 - 50}{100 \text{ (Type 3)}}$$

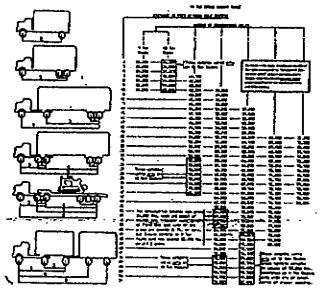
0.5 x 24 = 12 tons

SHV's

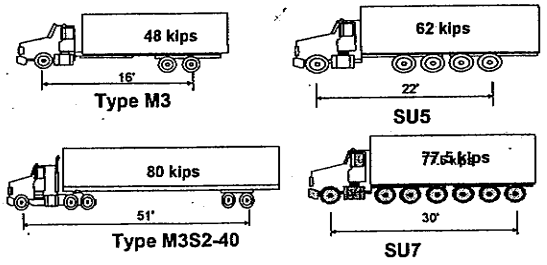
- Specialized Hauling Vehicles are single trucks with typically liftable axles that produce significantly higher stresses than posting trucks.
- NCHRP Project 12-63

Specialized Hauling Vehicles

- Formula B developed 60 years ago.
- Trucking industry innovatively modified their trucks in past 10 years.
- SHV's add weight in chart by just adding axles without adding proportional length.



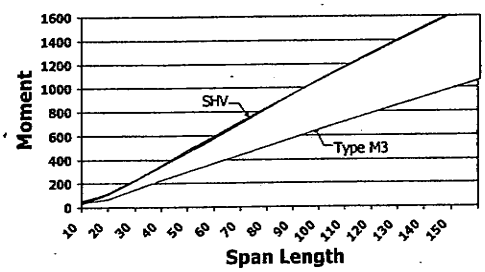
SHV's



SHV's

- These are legal and are becoming more commonplace on all roads.

SHV vs HS20 vs Type 3

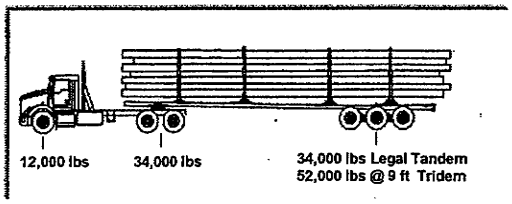


SHV's

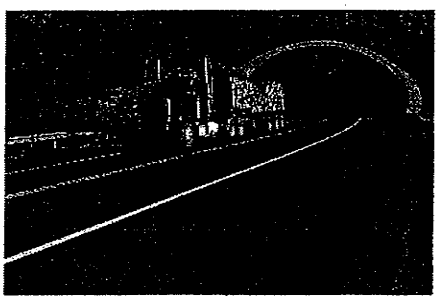
- Currently not posting for these vehicles.
- Will be a requirement in new "Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges" (LRFR)
- MnDOT just started running analysis on TH bridges
- Highly suggested to do SHV analysis on any new ratings and post for these trucks now

Timber Haulers Truck

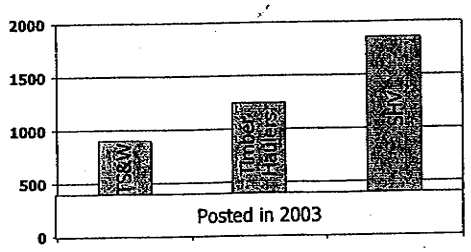
- First haul of raw timber forest products
- Unregulated permitted truck
- County bridges should already be posted



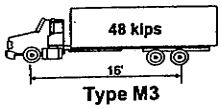
Timber Haulers Truck



Minnesota Postings (Est)



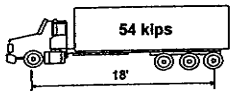
Type M3 Posting (24 tons)



Type M3

- RF<0.125 close bridge (3 tons)
- 0.125<RF<1.1 post at calculated tonnage
- RF>1.1 no posting needed for Type 3 refer to SU4

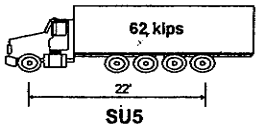
SHV SU4 Posting (27 tons)



SU4

- 0.89<RF<1.1 post at calculated tonnage
- RF>1.1 no posting needed for SU4 refer to SU5

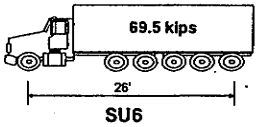
SHV SU5 Posting (31 tons)



SU5

- 0.87<RF<1.1 post at calculated tonnage
- RF>1.1 no posting needed for SU5 refer to SU6

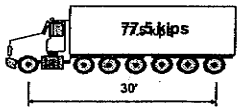
SHV SU6 Posting (34.75 tons)



SU6

- 0.89<RF<1.1 post at calculated tonnage
- RF>1.1 no posting needed for SU6 refer to SU7

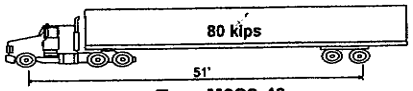
SHV SU7 Posting (38.75 tons)



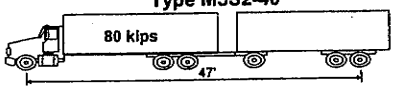
SU7

- 0.89<RF<1.025 post at calculated tonnage
- 1.025<RF<1.13 post at 40 tons
- RF>1.13 no posting needed for SU7 refer to combination truck trailors

Truck M3S2, M3S3 (40 tons)



Type M3S2-40



Type M3S3

- RF<0.35 post using lowest single truck posting with sign R12-1

Truck M3S2, M3S3 (40 tons)

- 0.35<RF<1.00 post at indicated tonnage
- 1.00<RF<1.12 post at 40 tons
- 1.12<RF<1.23 post with the 45 T (Permit Weight Limit) sign unless single truck requires posting then use 40 tons for these two trucks (Timber Haulers truck)
- 1.23<RF no posting required

Posting Tidbits

- Typically round down to even integers
- The M3S2 and M3S3 semis post at same tonnage
- If as an owner you do not have SHV or Timber Haulers truck, you are not required by code to post for those vehicles.
- Do posting analysis for any bridge that has operating rating less than HS28
- Use Type M3, SHV's M3S2 and M3-3 for all 9 and 10 ton routes. If lower axle weight road then ratio down posting truck.
- If posting from engineer judgement the HS operating rating = posting for single truck = 0.625 x posting for semi.

Approximate Weights of Various Trucks

- School bus – 35 k
- Articulating city Bus – over legal – 70 k
- Fire Truck – wide variety – 62 k (3 axles)
- Small Tractor – 12 k (95 hp)
- Large Tractor – 41 k (350 hp)
- Concrete Truck – up to 77 k
- Single Truck full of gravel – up to 77 k

Permitting

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

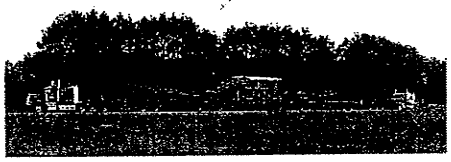
RF = Rating factor for live load capacity
C = Capacity of the member
D = Dead load on member
L = Live load on member
I = Impact Factor
A_1 = Factor for dead load
A_2 = Factor for live load

Use the same equation and values used to conduct the load rating. Substitute the permit truck for the live load. If the RF > 1 then the truck can pass without restriction.

Permitting

- If RF<1.0 for permit truck
  - Can ask truck to be the only vehicle on bridge. Can use the single lane distribution factor, Beam Spacing/7 rather than Beam Spacing/5.5
  - Can ask truck to limit speed to 5mph and not use brake. This reduces impact to 5% in calculations.
  - Apply both restrictions
  - JUST BECAUSE THE TRUCK WENT OVER THE BRIDGE LAST WEEK AND NOTHING HAPPENED DOES NOT MEAN AS AN ENGINEER YOU CAN IGNORE CALCULATIONS. HIGH STRESS LOADINGS SIGNIFICANTLY REDUCE THE LIFE OF STRUCTURE AND REDUCE THE FUTURE SAFETY FACTORS.

Permitting



Robbins Motor Transportation (445,000 lbs, 225')

Mn/DOT Structure Inventory Report

Bridge ID: 5292 **CSAH 34 over STREAM** Date: 02/27/2008

+ GENERAL +

Agency Br. No.
District 7 **Maint. Area**
County 40 - LE SUEUR
City
Township TYRONE
Desc. Loc. 2.0 MI SW OF JCT TH 169
Sect., Twp., Range 07 - 112N - 25W
Latitude 44d 31m 31.83s
Longitude 93d 52m 56.01s
Custodian COUNTY
Owner COUNTY
Inspection By LESUEUR COUNTY
BMU Agreement No.
Year Built 1933
Year Fed Rehab
Year Remodeled
Temp
Plan Avail. NO PLAN

+ STRUCTURE +

Service On HIGHWAY
Service Under STREAM
Main Span Type STEEL BM SPAN
Main Span Detail
Appr. Span Type
Appr. Span Detail
Skew
Culvert Type
Barrel Length
Number of Spans
MAIN: 1 APPR: 0 TOTAL: 1
Main Span Length 27.0 ft
Structure Length 30.2 ft
Deck Width 23.9 ft
Deck Material TIMBER
Wear Surf Type GRAVEL
Wear Surf Install Year
Wear Course/Fill Depth 0.25 ft
Deck Membrane NONE
Deck Rebars N/A
Deck Rebars Install Year
Structure Area 722 sq ft
Roadway Area 689 sq ft
Sidewalk Width - L/R
Curb Height - L/R
Rail Codes - L/R 32 32

+ ROADWAY +

Bridge Match ID (TIS) 1
Roadway O/U Key 1-ON
Route Sys/Nbr CSAH 34
Roadway Name or Description
CSAH 34
Roadway Function MAINLINE
Roadway Type 2 WAY TRAF
Control Section (TH Only)
Ref. Point (TH Only)
Date Opened to Traffic
Detour Length 4 mi.
Lanes 2 Lanes ON Bridge
ADT (YEAR) 200 (2003)
HCADT
Functional Class. RURAL LOCAL

+ RDWY DIMENSIONS +

If Divided	NB-EB	SB-WB
Roadway Width	22.9 ft	
Vertical Clearance		
Max. Vert. Clear.		
Horizontal Clear.		
Lateral Clr. - Lt/Rt		
Appr. Surface Width	34.0 ft	
Roadway Width	22.9 ft	
Median Width		

+ MISC. BRIDGE DATA +

Structure Flared NO
Parallel Structure NONE
Field Conn. ID
Cantilever ID
Foundations
Abut. TIMBER - PILE BENT
Pier N/A

+ PAINT +

Year Painted 1959 **Pct. Unsound**
Painted Area
Primer Type LEAD
Finish Type PHENOLIC RESIN ALUM.

+ BRIDGE SIGNS +

Posted Load VEHICLE & SEMI
Traffic NOT REQUIRED
Horizontal OBJECT MARKERS
Vertical NOT APPLICABLE

+ INSPECTION +

Deficient Status S.D.
Sufficiency Rating 39.6
Last Inspection Date 10-22-2007
Inspection Frequency 12
Inspector Name PONTIS
Structure P-LOAD POSTED

+ CONDITION CODES +

Deck	4
Superstructure	4
Substructure	4
Channel	6
Culvert	N

+ APPRAISAL RATINGS +

Structure Evaluation	4
Deck Geometry	4
Underclearances	N
Waterway Adequacy	8
Approach Alignment	5

+ SAFETY FEATURES +

Bridge Railing 0-SUBSTANDARD
Appr. Guardrail UNKN
GR Transition UNKN
GR Termini UNKN

+ IN DEPTH INSP. +

Frac. Critical
Underwater
Pinned Asbly.
Spec. Feat.

+ WATERWAY +

Drainage Area
Waterway Opening 216 sq ft
Navigation Control NO PRMT REQD
Pier Protection UNKN
Nav. Vert./Horz. Clr.
Nav. Vert. Lift Bridge Clear.
MN Scour Code I-LOW RISK
Scour Evaluation Year 1991

+ CAPACITY RATINGS +

Design Load UNKN
Operating Rating HS 17.5
Inventory Rating HS 12.1
Posting VEH: 23 SEMI: 36 DBL:
Rating Date 10-01-1981
Mn/DOT Permit Codes
A: N B: N C: N

Bk. 5292

BR. 5292

BRIDGE
WEIGHT
23 TONS
PER VEHICLE
36 TONS
PER TRUCK AND
FULL TRAILER OR
SEMI-TRAILER
COMBINATION

AGE AND CONDITION OF STRUCTURES BY ROUTE SYSTEM 2007

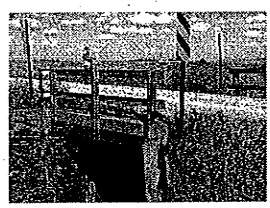
CONDITION OF STRUCTURES 10 FT AND OVER									
ROUTE SYSTEM	BRIDGES	CULVERTS	TOTAL STRUCTURES	RATED > 30 YRS	AVG OP RT	POSTED	POSTED & > 30 YRS	AVG OP RT	
COUNTY	3,596	4,210							
TOWNSHIP	1,985	3,991							
CITY	747	392							
TOTAL	6,328	8,593	14,921	2,301	26.9	765	190	15.5	

Workshop Agenda

I. Introductions
II. Load Rating Basics
III. General Equations
IV. Load Rating Procedure
V. Incorporating Member Distress
VI. Posting, SHV's and Permitting
VII. Load Rating Example #1
   - Simple Span Non-composite Steel

Load Rating Example #1

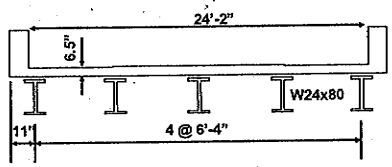
- Simple Span Non-composite Steel
  (Bridge Type 301)



Procedure

1. Calculate Member Capacity
2. Calculate Dead Load
   - Incorporate section loss
3. Calculate Live Load
   - HS20 / Posting / Permit trucks
4. Rating Equation
5. Check Posting if applicable
6. Check Permit Truck if applicable

Load Rating Example #1
Br 5905



- Non-composite
- 36'-0" simple span
- F_y = 33 ksi

Load Rating Example #1

- Calculate W24x80 moment capacity
  - I_x = 2230 in^4
  - $S_x = \frac{I_x}{D/2} = \frac{2230\ in^4}{24\ in/2} = 186\ in^3$
  - $M_y = S_x \times F_y = 186\ in^3 \times 33\ ksi$
    $= 6138\ k - in = 512\ k - ft$

Load Rating Example #1

- Calculate Dead Load Moment
  - Beam - 80 lb/ft
  - Haunches, diaphragms and Misc - say 8 lb/ft
  - 2 - Railings - 320 lb/ft
  - Deck 6.5" with 6'- 4" tributary area

Load Rating Example #1

- **Calculate Dead Load Moment**
  - **M = $wL^2/8$ (uniform load simple span max M)**

$$M_{DL} = \frac{\left(80 + 8 + \frac{320 \times 2}{5} + \left(\frac{6.5}{12} \times 6.33 \times 150\right)\right) \times 36^2}{8}$$

$$M_{DL} = 118350 \text{ lb-ft} = 118.4 \text{ k-ft}$$

Load Rating Example #1

- **Calculate Live Loads**
  - **Distribution Factor = Beam spacing/11**
  - **Assume Impact = 30%**

$$DF = 6.333/11 = 0.576$$

$$HS20 = 379 \text{ k-ft}$$ **From AASHTO chart for span length**

$$M_{LL} = HS20 \times DF \times \text{Impact}$$

$$M_{LL} = 379 \times 0.576 \times 1.3 = 283.7 \text{ k-ft}$$

Load Rating Example #1

- **Calculate Inventory Rating**

$$\text{HS Rating} = \frac{\text{Capacity} - 1.3 \times \text{Dead Load}}{2.17 \times \text{Live Load}_{\text{HS20+Impact}}} \times 20$$

$$\text{HS Rating} = \frac{(512 \text{ k-ft}) - (1.3 \times 118.4 \text{ k-ft})}{2.17 \times 283.7 \text{ k-ft}} \times 20$$

$$\text{HS Rating} = \text{HS11.6}$$

Load Rating Example #1

- **Calculate Operating Rating**

$$\text{HS Rating} = \frac{\text{Capacity} - 1.3 \times \text{Dead Load}}{1.3 \times \text{Live Load}_{\text{HS20+Impact}}} \times 20$$

$$\text{HS Rating} = \frac{(512\,\text{k-ft}) - (1.3 \times 118.4\,\text{k-ft})}{1.3 \times 283.7\,\text{k-ft}} \times 20$$

$$\text{HS Rating} = \text{HS19.4}$$

Load Rating Example #1

- **Scenario 1**
  - **Assume 2" deck overlay with ½" milled off**
    Calculate Dead Load Moment

$$M_{\text{Overlay}} = \frac{\left(80 + 8 + \frac{320 \times 2}{5} + \left(\frac{(6.5 + 2 - 0.5)}{12} \times 6.33 \times 150\right)\right) \times 36^2}{8}$$

$$M_{DL} = 137587\,\text{lb-ft} = 137.6\,\text{k-ft}$$

Load Rating Example #1

- **Calculate Updated Rating with overlay**

$$\text{Inv Rating} = \frac{(512\,\text{k-ft}) - (1.3 \times 137.6\,\text{k-ft})}{2.17 \times 283.7\,\text{k-ft}} \times 20$$

$$\text{Inv Rating} = \text{HS10.8}$$

$$\text{Op Rating} = \frac{(512\,\text{k-ft}) - (1.3 \times 137.6\,\text{k-ft})}{1.3 \times 283.7\,\text{k-ft}} \times 20$$

$$\text{Op Rating} = \text{HS18.1}$$

Load Rating Example #1

- Scenario 2
  - Assume 50% section loss to bottom flange at center span
  - $I_x = \sum(I_x + (A \times d^2))$– Parallel Axis Theorem
  - $I_x = 1670\ in^4$
  - $S_x = 123\ in^3$
  - $M_y = S_x \times F_y = 123 \times 33 = 4060\ k\text{-}in = 338\ k\text{-}ft$

Load Rating Example #1

- Calculate Updated Rating with section loss

$$\text{Inv Rating} = \frac{(338\,k\text{-}ft) - (1.3 \times 118.4\,k\text{-}ft)}{2.17 \times 283.7\,k\text{-}ft} \times 20$$

$$\text{Inv Rating} = HS5.0$$

$$\text{Op Rating} = \frac{(338\,k\text{-}ft) - (1.3 \times 118.4\,k\text{-}ft)}{1.3 \times 283.7\,k\text{-}ft} \times 20$$

$$\text{Op Rating} = HS10.0$$

Rating Summary

Rating	Original	2" Overlay	Section Loss
Inventory	HS11.6	HS10.8	HS5.0
Operating	HS19.4	HS18.1	HS10.0

Load Rating Example #1 (Cont.)

- Check for posting on section loss example
  - Check Type M3, M3S2 and M3S3 at operating level

$$\text{Rating Factor} = \frac{\text{Capacity} - 1.3 \times \text{Dead Load}}{1.3 \times \text{Live Load}_{\text{Posting Truck}} + \text{Impact}}$$

  - To get posting weight take truck weight times rating factor

Load Rating Example #1

- Find Posting Truck moments from chart
  - Type M3 (24 tons) = 410 k - ft
  - Type M3S2 (40 tons) = 379.4 k - ft
  - Type M3S3 (40 tons) = 388.6 k - ft

Truck	RF	Tons	Sign
Type M3	0.60	14.4	14
Type M3S2	0.65	25.9	24
Type M3S3	0.63	25.2	24

Load Rating Example #1 (Cont.)

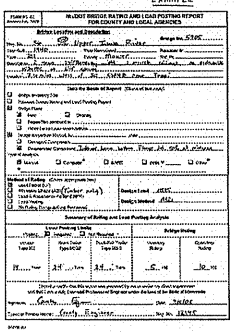
- Other things to check by engineer
  - Plastic moment capacity, Z
  - Shear capacity (near supports)
  - Serviceability AASHTO 10.57.1
    - 0.8Fy noncomposite
    - 0.95Fy composite
  - Bearing Stiffeners
  - Exterior Beam
  - Compression Flange Bracing

Load Rating Example #1

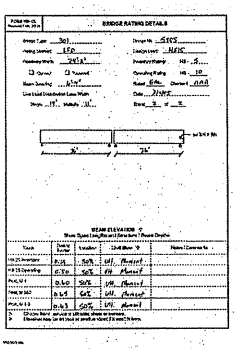
- Not usually checked
  - Splices, diaphragms or connections
  - Fatigue
  - Concrete Deck
  - Local failure
  - Shear for slabs
  - Substructure (bearings, piles, movement)

REPORT FORM

EXAMPLE



REPORT FORM



Workshop Agenda

VIII. Load Rating Example #2
- Simple Span Nail Laminated Timber Deck (without distress)

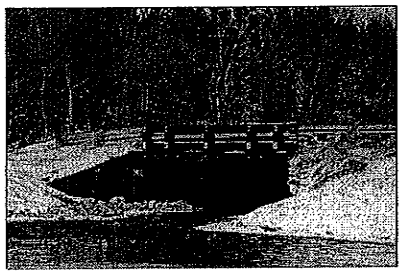
IX. Class Exercise
- Simple Span Nail Laminated Timber Deck (with distress)

X. Review of worked out examples

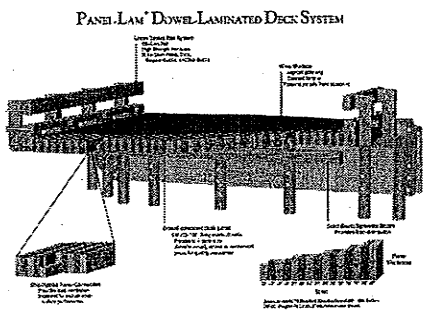
XI. Submittal to MN/DOT and Review of Process

XII. Common mistakes and questions

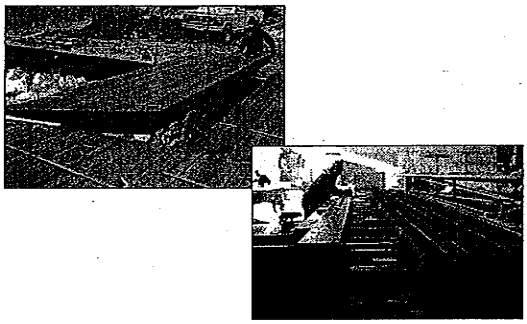
Simple Span Nail Laminated Timber Deck Example (without distress)



Nail Laminated Timber Deck



Nail Laminated Timber Deck

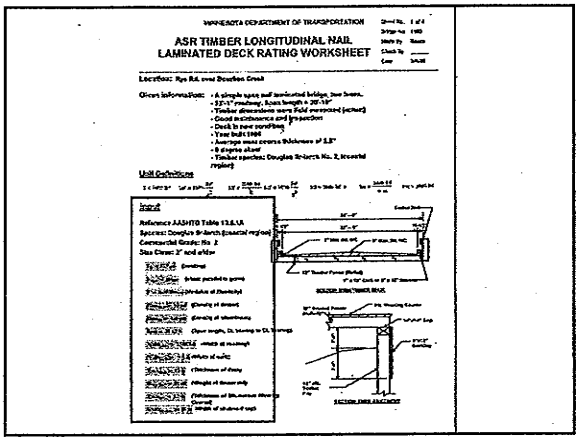


Simple Span Nail Laminated Timber Deck Example (without distress)

- A simple span nail laminated bridge, two lanes.
- 32'-1" roadway, Span length = 20'-10"
- Timber dimensions were field measured (actual)
- Good maintenance and inspection

Simple Span Nail Laminated Timber Deck Example (without distress)

- Deck in new condition
- Year built 1966
- Average wear course thickness of 3.5"
- 0 degree skew
- Timber species: Douglas fir-larch No. 2



MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER LONGITUDINAL NAIL LAMINATED DECK RATING WORKSHEET

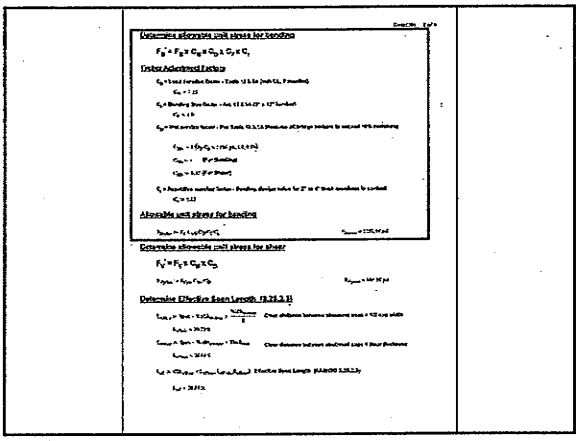
Location: Xyz Rd. over Bourbon Creek

Given Information:
- A simple span nail laminated bridge, four beam.
- 23'-5" roadway, Span length = 25'-10"
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Deck in new condition
- Year built 1966
- Average wear course thickness of 2.5"
- 0 degree skew
- Timber species: Douglas Fir-larch No. 2, Assorted region)

Unit Definitions

Input

Reference AASHTO Table 4.3.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: No. 2
Size Class: 2" and wider



Determine allowable unit stress for bending

$F'_b = F_b \times C_M \times C_D \times C_t \times C_L$

Timber Adjustment Factors

C_D = Load duration factor - Table 4.3.2a (with CL, F bending)

C_D = 1.25

C_t = Beating time factor - Art. 4.3.4.4 (F' > 10" bending)

C_t = 1.0

C_M = Wet service factor - Per Table 4.3.3.1 Moisture of timber is assumed <19% maintaining

$F_{bx} = F_b C_D \times 1.0 \times 1.0 \times 1.0 \times 1.0$

C_{Mbx} = 1.0 for Bending

C_{Mv} = 0.67 for Shear

C_L = Beam stability factor - Bending design value for 2" to 4" thick members is exempt

C_L = 1.0

Allowable unit stress for bending

$F_{bx} = F_b \times C_D \times C_t \times C_L$ F_{bx} = 1125.00 psi

Determine allowable unit stress for shear

$F'_v = F_v \times C_M \times C_D$

$F_{vx} = F_{vx} \times C_M \times C_D$ F_{vx} = 180.00 psi

Determine Effective Beam Length (3.3.3.3)

$l_{ex} = 1.92 l_u + \frac{h_x C_{Mbx}}{\pi}$ Clear distance between abutment seat = 12 inch width

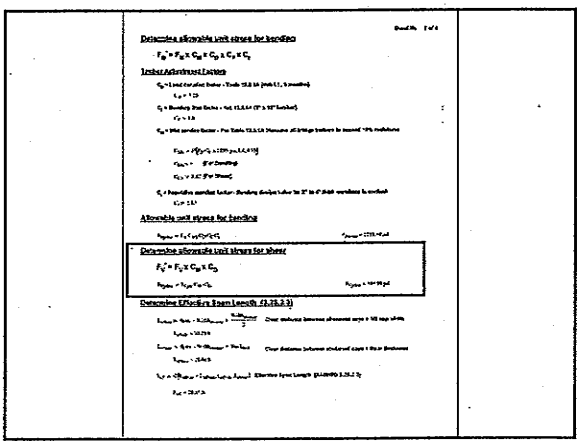
l_{ex} = 24.750

$l_{ex} = l_u + h_x C_{Mbx} + 3$ in l_{ex} Clear distance between abutment seats = Beam Enclosure

l_{ex} = 24.0 ft

$l_{ex} \leq 1.92 l_{ex} + 1.5 l_{ex} + 1.5 l_{ex} + 3$ in Effective Span Length (4.3.3.3.3)

l_{ex} = 24.0 ft



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C_D = 1.25

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C_{Mv} = 0.67 for Shear

C_L = Beam stability factor - Bending design value for 2" to 4" thick members is exempt

C_L = 1.0

Allowable unit stress for bending

$F_{bx} = F_b \times C_D \times C_t \times C_L$ F_{bx} = 1125.00 psi

Determine allowable unit stress for shear

$F'_v = F_v \times C_M \times C_D$

$F_{vx} = F_{vx} \times C_M \times C_D$ F_{vx} = 180.00 psi

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l_{ex} = 24.750

$l_{ex} = l_u + h_x C_{Mbx} + 3$ in l_{ex} Clear distance between abutment seats = Beam Enclosure

l_{ex} = 24.0 ft

$l_{ex} \leq 1.92 l_{ex} + 1.5 l_{ex} + 1.5 l_{ex} + 3$ in Effective Span Length (4.3.3.3.3)

l_{ex} = 24.0 ft

Determine allowable unit stress for bending

$F'_b = F_b \times C_{fd} \times C_{fp} \times C_p \times C_c$

Ticket Adherence Factor

C_{fd} = Load duration factor - Table 12.5.3 (with LL, 1 member)

$C_{fd} = 1.0$

C_{fp} = Bending key factor - see 12.5.3.4(3)" C f-C" symbol

$C_{fp} = 1.0$

C_{fp} = Flat service factor - For Table 12.5.3, if member 12.5 edge factors to exceed 10% reinforced

$C_{fp} = 40 b_f / d = 1.00 \times 12 / 12 = 1.0$

$C_{fp} = 1$ - For Bending

$C_{fp} = 1.17$ - For Shear

C_c = Repetitive member factor - Bending design value for 3" to 4" thick members in contact

$C_c = 1.0$

Allowable unit stress for bending

$F'_{bend} = F_b \times C_{fd} \times C_{fp} \times C_c$ $F'_{bend} = 1125.0$ psi

Determine allowable unit stress for shear

$F'_v = F_v \times C_{fd} \times C_p$

$F'_{shear} = F_{shear} \times C_{fd} \times C_p$ $F'_{shear} = 250.0$ psi

Determine Effective Span Length (3.24.2.3)

$L_{eff,1} = \text{Span} - 12 in/ft_{eff,1} + \frac{w_{eff,1}}{3}$ Clear distance between shortened ends + 1/2 top width

$L_{eff,1} = 29.25$ ft

$L_{eff,2} = \text{Span} - 12 in/ft_{eff,2} + 2 in/ft_{eff,2}$ Clear distance between shortened edges + Deck thickness

$L_{eff,2} = 30.0$ ft

$L_{eff} = 2.0 \times L_{eff,1} + L_{eff,2} + L_{eff,3}$ Effective Span Length (AASHTO 3.24.2.3)

$L_{eff} = 29.0$ ft

Concrete Live Load Distribution Width per AASHTO 3.24.3.2

Normal to direction of the span, wheel load shall be distributed as follows.
Panel Span 30" max.

$P_{wheel} = 16$ kips (HS20 wheel load)

$TCA = \frac{P_{wheel}}{12 psi}$ $TCA = 1333.3$ in^2 Tire Contact Area (3.30)

$L_1 = \sqrt{TCA}$ $L_1 = 36.5$ in (Width of tire contact transverse or span for HS20-44 truck) $L_1 = 3.0$ ft

$S_w = L_1 + [2 \times (w_{eff,1} + D)]$ (Wheel load distribution width perpendicular to deck span for 1 wheel) $S_w = 6.0$ ft

Dead Loads

DL of Stable Deck $DL_{deck} = D_{deck} \times w_{deck} \times \text{Unit Wt}_{conc}$ $DL_{deck} = 150.0$ plf

DL of Rail Assume dead load of railing system is uniformly distributed across the deck width

$DL_{rail} = \frac{\text{Rail Wt} \times 2 \text{ lbs}}{\text{Deck Width} \times (12 in/ft)}$ $DL_{rail} = 25$ plf

DL of Wearing Course $DL_{WC} = D_{WC} \times w_{deck} \times \text{Unit Wt}_{WC}$ $DL_{WC} = 150.0$ plf

Total Dead Load of superstructure $DL_{ss} = DL_{deck} + DL_{rail} + DL_{WC}$ $DL_{ss} = 325.0$ plf

Concrete Dead Load Moment

$M_{DL} = \frac{DL \times L_{eff}^2}{8}$ $M_{DL} = 34.0$ k-ft

Concrete Live Load Moment

HS20 Truck Moment per AASHTO App. A (Note that Live load moments in App. A are based on one lane without impact)

For Span = 28 feet $M_{LL} = 161$ k-ft

For Span = 31 feet $M_{LL} = 183$ k-ft

Therefore for Span = 29.25 feet $M_{LL} = 161 + \left[\frac{29.5 - 28}{31 - 28}\right] \times (M_{LL,31} - M_{LL,28})$

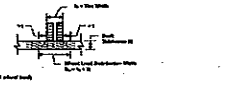
$M_{LL} = 171.5$ k-ft

$M_{LL} = 171.5$ k-ft

$M_{LL,Wheel} = \frac{M_{LL}}{2}$ (Moment per wheel) $M_{LL,Wheel} = 85.8$ k-ft

Concrete Live Load Distribution Width per AASHTO 3.24.3.2

Normal to direction of the span, wheel load shall be distributed as follows.
Panel Span 30" max.



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Dead Loads

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DL of Wearing Course $DL_{WC} = D_{WC} \times w_{deck} \times \text{Unit Wt}_{WC}$ $DL_{WC} = 150.0$ plf

Total Dead Load of superstructure $DL_{ss} = DL_{deck} + DL_{rail} + DL_{WC}$ $DL_{ss} = 325.0$ plf

Concrete Dead Load Moment

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Concrete Live Load Moment

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$M_{LL} = 171.5$ k-ft

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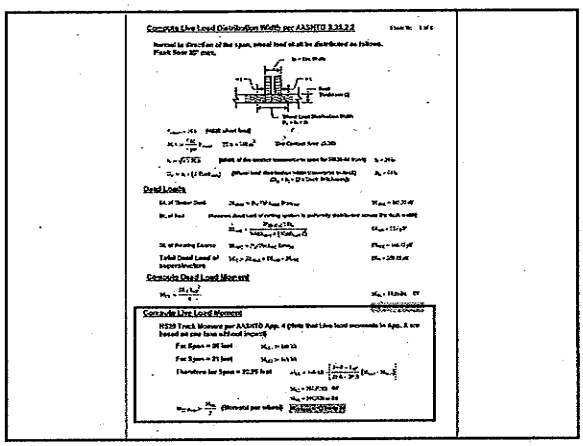


Compute Live Load Distribution Width per AASHTO 3.25.2.2

Normal to direction of the span, wheel load shall be distributed as follows.
Flank Span 30" max.

Dead Loads

Compute Dead Load Moment

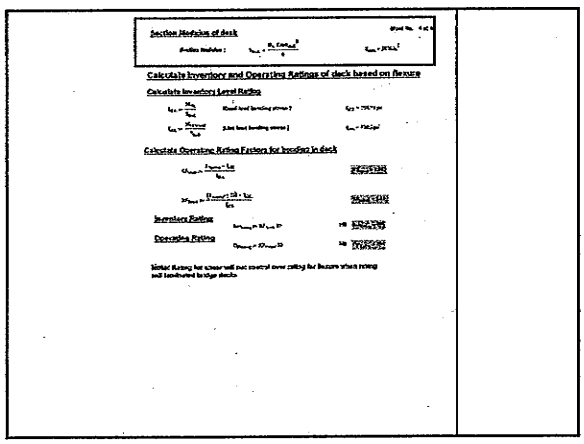
Compute Live Load Moment

HS20 Truck Moment per AASHTO App. 4 (Note that Live load moments in App. A are based on one lane without impact)

For Span = 30 feet

For Span = 21 feet

Therefore for Span = 20.25 feet



Section Modulus of deck

Calculate Inventory and Operating Ratings of deck based on flexure

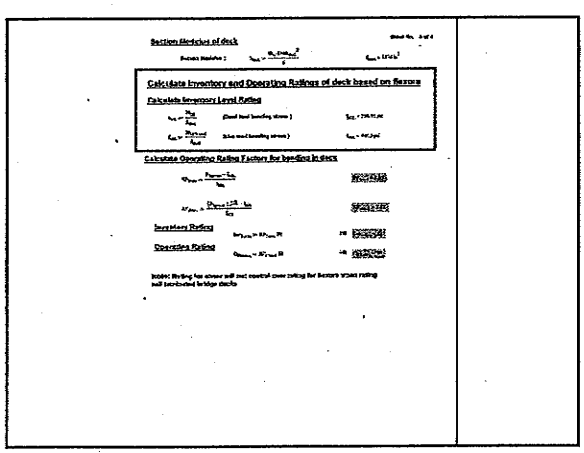
Calculate Inventory Level Rating

Calculate Operating Rating Factors for bending in deck

Inventory Rating

Operating Rating

Note: Rating for shear will not control over rating for flexure when rating will bombarded bridge decks



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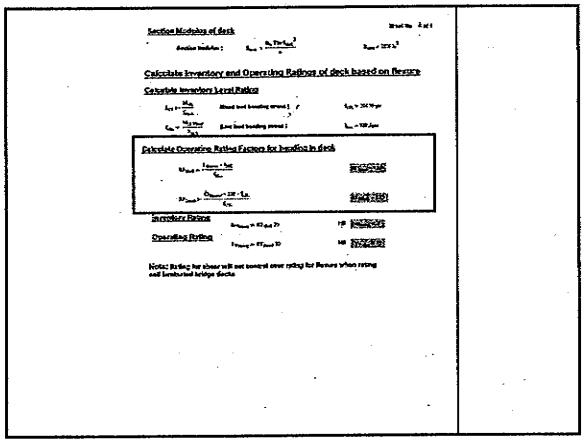


Section Modulus of deck

Calculate Inventory and Operating Ratings of deck based on flexure

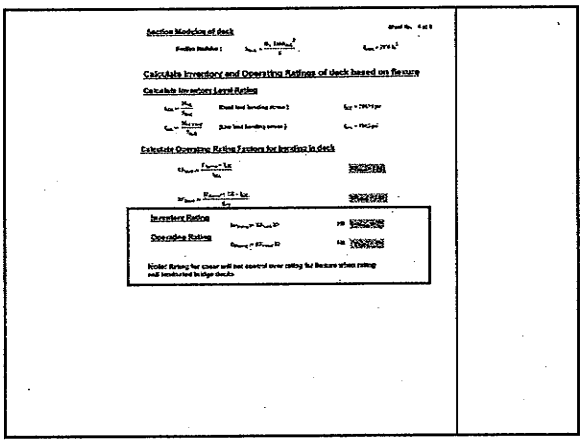
Calculate Inventory Level Ratios

Calculate Operating Rating Factors for bending in deck

Inventory Rating

Operating Rating

Note: Rating for shear will not control over rating for flexure when rating nail laminated bridge decks



Section Modulus of deck

Calculate Inventory and Operating Ratings of deck based on flexure

Calculate Inventory Level Ratios

Calculate Operating Rating Factors for bending in deck

Inventory Rating

Operating Rating

Note: Rating for shear will not control over rating for flexure when rating nail laminated bridge decks

Simple Span Nail Laminated Timber Deck Example #2 (with distress)

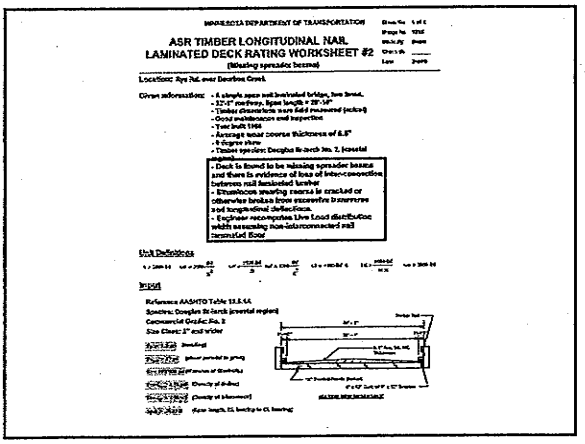
- A simple span nail laminated bridge, two lanes.
- 32'-1" roadway, Span length = 20'-10"
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built 1966

Simple Span Nail Laminated Timber Deck Example #2 (with distress)

- Average wear course thickness of 6.5"
- 0 degree skew
- Timber species: Douglas fir-larch No. 2
- Deck is found to be missing spreader beams and there is evidence of loss of inter-connection between nail laminated lumber

Simple Span Nail Laminated Timber Deck Example #2 (with distress)

- Bituminous wearing course is cracked or otherwise broken from excessive transverse and longitudinal deflections.
- Engineer must recompute Live Load distribution width assuming non-interconnected nail laminated floor



MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER LONGITUDINAL NAIL LAMINATED DECK RATING WORKSHEET #2

(Missing spreader beams)

Location: Rye Rd. over Dearborn Creek

Given information:

- A simple span nail-laminated bridge, two lanes,
- 32'-0" roadway, Span length = 29'-10"
- Timber characteristic were field measured (values)
- Good maintenance and inspection
- Year built 1966
- Average wear course thickness of 6.5"
- 0 degree skew
- Timber species: Douglas fir-larch No. 2 (coastal

Deck is found to be missing spreader beams and there is evidence of loss of interconnection between nail laminated lumber

- Bituminous wearing course is cracked or otherwise broken from excessive transverse and longitudinal deflections.
- Engineer recomputes Live Load distribution width assuming non-interconnected nail laminated floor

Unit Definitions

Notes

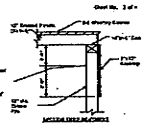
Reference AASHTO Table 15.5.4A

Species: Douglas fir-larch (coastal region)

Commercial Grade: No. 2

Size Class: 2" and wider

Input (cont)



Determine Allowable soil stress for bearing

$F_b = F_g \times C_{sp} \times C_D \times C_h \times C_v$

Finker-Adolphont Factors

C_{sp} = Load duration factor - Table 13.5.1A Pub LL, 3 months)

$C_{sp} = 1.15$

C_D = Shallow Depth factor - eq 13.5.1-1 $d = 11'$ (united)

$C_{sp} = 1$

C_h = Pier section factor (Per Table 13.5.1A (concrete of bridge defined as coastal CPS substructure)

$C_{sp} = \sqrt{B_f / C_f}$ (12'/ft per 14.5 ft²)

$C_{sp} = 1$ (For Bending)

$C_{sp} = 0.8$ (For Shear)

C_v = Repetitive movement factor - Bending design value for 3" to 6" (min) movement is passed

$C_v = 1$

Allowable soil stress for bearing

$F_{bearing} = F_g C_{sp} \times C_D \times C_v$ $F_{bearing} = 1.15$ ksf

Determine Allowable soil stress for shear

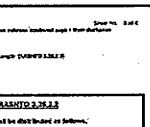
$F_s = F_g \times C_{sp} \times C_D$

$F_{shear} = F_{bearing} C_{sp}$ $F_{shear} = 0.8$ ksf

Determine Effective Span Length (3.24.2.3)

$L_{eff,A} = S_{pan} - W_{pier,top} + \frac{W_{pier,bot}}{3}$ Clear distance between abutment caps + 1/3 pier width

$L_{eff,A} = 28.5'$ +



Sheet No. 3 of 6

$L_{eff,B} = S_{pan} - W_{pier,top} + W_{pier,bot}$ Clear distance between abutment caps + Pier diameter

$L_{eff,B} = 29.5'$

$L_{eff} = \sqrt{L_{eff,A} \times L_{eff,B}}$ Effective Span Length (AASHTO 3.24.2.3)

$L_{eff} = 29.0'$

Concrete Live Load Distribution Width per AASHTO 3.24.3.2

Normal to direction of the span, wheel load shall be distributed as follows, Width from 30" min.

$P_{wheel} = 16$ (MSDF wheel load)

$S_{eff} = \frac{d+11}{12}$ $W_{eff} = 24'$ Slab Centerline max (L/20)

$b = \frac{P_{wheel}}{S_{eff} \times W_{eff}}$ (Width of the assumed transverse to span for MS18-44 truck) $b_1 = 20.2$ in

$b_{eff} = b_1 + (2 \times t_{slab})$ (Where load distribution when transverse to deck (assuming non-continuous) full two-wheel load $P_{wheel} = 16$ k Deck Centerline) $b_{eff} = 32$ in

Dead Loads

DL of Prefab Deck $DL_{deck} = P_{slab} \times t_{slab} \times W_{eff}$ $DL_{deck} = 0.177$ klf

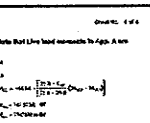
DL of Rail (Assume dead load of railing system is uniformly distributed across the deck width) $P_{rail} = \frac{W_{rail} \times W_{eff}}{(W_{deck} \times W_{deck})}$ $P_{rail} = 0.15$ klf

DL of Wearing Course $DL_{WC} = W_{WC} \times t_{WC} \times W_{eff}$ $DL_{WC} = 0.144$ klf

Total Dead Load of superstructure $DL_T = W_{deck} + W_{rail} + W_{WC}$ $DL_T = 0.471$ klf

Concrete Dead Load Moment

$M_{DL} = \frac{W_{DL} \times L_{eff}^2}{8}$ $M_{DL} = 46.156$ k-ft



Concrete Live Load Moment

Sheet No. 4 of 6

HS20 Truck Moment per AASHTO App. A (Note that Live load moments in App. A are based on one lane without impact)

For Span = 28 feet $M_{LL} = 36$ k

For Span = 34 feet $M_{LL} = 36$ k

Therefore for Span = 28.25 feet $M_{LL} = 44.94 \times \left[\frac{(28.5 - 8)}{(34 - 28)}(36_{34'} - 36_{28'}) + 36_{28'}\right]$

$M_{LL} = 50.8$ k-ft 49'

$M_{LL} = 1.74$ k-ft/in-ft

$M_{LL,per\ wheel} = \frac{M_{LL}}{2}$ (Moment per wheel) $M_{LL,per\ wheel} = 25.4$ k-ft

Section Modulus of Deck

Section Modulus: $S_{deck} = \frac{b_1 \times t_{slab}^2}{6}$ $S_{deck} = 24$ in³

Calculate Inventory and Operating Ratings of deck based on Stress

Calculate Inventory Level Rating

$f_{DL} = \frac{M_{DL}}{S_{deck}}$ (Dead load bending stress) $f_{DL} = 23.1$ ksi

$f_{LL} = \frac{M_{LL,per\ wheel}}{S_{deck}}$ (Live load bending stress) $f_{LL} = 12.7$ ksi/lane

Calculate Operating Rating Factor for loadings in deck

$W_{max,inv} = \frac{F_{allow} - f_{DL}}{f_{LL}}$ $W_{max,inv} = 1.58$ lanes

$W_{max,op} = \frac{W_{max,inv} \times 1.3}{1.3}$ $W_{max,op} = 1.58$ lanes

Inventory Rating	$Inv_{Rating} = W_{max,inv} \times 36$	56 k
Operating Rating	$Op_{Rating} = W_{max,op} \times 36$	56 k

Notes: Rating for shear will not control over rating for flexure when rating mid-sectioned bridge decks

- Since the Operating Rating is less than HS25, you need to consider posting.

2008 BRIDGE RATING CLASS 101

Timber Basics

Sawn Lumber Beams:

- Typically 4" to 8" wide, 12" to 18" deep with spans up to 25'; beams spaced 2' to 6' apart.

Glue Laminated Beams:

- Manufactured from 1-1/2" or 1-3/8" thick lumber laminations with waterproof adhesives.
- Beam widths from 3" to 14-1/4" with depths limited only by transportability. Spans range from 20' to 80'.

Longitudinal Deck Superstructures:

- Glulam Panels
    - Glue laminated longitudinally
    - Panels are 6-3/4" to 14-1/4" deep and 42" to 54" wide
- Nail or Spike laminated Panels
    - Uses 2" to 4" wide sawn lumber, 8"-16" deep, nailed or spiked together
- Clear spans for longitudinal decks up to 36'

Spreader Beams:

- Stiffener beams placed across the deck width to distribute live loads among the individual panels.
- Spreader beams are placed at midspan and at intermediate spacings not to exceed 10 feet.

Timber Substructures:

- Abutments:
    - Pile abutments (See Photo 1 below); superstructure is supported on timber piles (20 to 32 tons capacity) with average pile length from 30 to 65 feet.
    - The superstructure is connected to the piles by a continuous cap attached to the piles and superstructure.

- Piers:
    - Bents used for multiple-span crossings. Constructs of timber piles with bracing to provide stability and resist lateral loads. The superstructure is connected to the piles by a continuous cap attached to the piles and superstructure (See Photo 2 below).

Physical Properties:

- Density – 50 pcf used for design.
- Durability - Keep dry, with moisture content below approximately 20%
- Wood fully saturated with water does not decay
- Decay needs air, water, and temperature
- Chemicals used for preservative treatment have little or no effect on strength

Mechanical Properties:

- Directional Characteristics:
  - Compressive strength parallel to grain may be 5 to 10 times greater than that perpendicular to the grain
  - Modulus of Elasticity (E) parallel to the grain is 10 to 25 times that perpendicular to the grain

- Resistance to Shear Loads:
  - Shear strength parallel to grain has a relative low value. Shear strength across the grain is never considered, since its value is high and failure will occur first in shear parallel to grain

- Resistance to Repeated Loads:
  - Fatigue strength of wood is often several times that of most metals. Repeated loads are generally of little concern. Fatigue is not normally a consideration in timber bridge design.

- Resistance to Rapidly Applied or Long-Continued loads:
  - The load that will cause failure in a member that must carry loads continuously for long periods of time is much less than the load that would cause failure in a few minutes
  - i.e. A member under continuous action of bending stress for 10 years will carry only about 60% of the load under which the same member would fail in a few minutes

Effects:

- Moisture - Higher properties at the lower moisture contents.
- Temperature - Properties increase with cooler temps, decrease with hotter temps. Temp effects are recoverable in short durations. Temps greater than 150 deg F for extended periods will result in permanent reduction in strength.
- Decay - Delay effects can be several times greater than usual observations. 50%-70% strength loss with corresponding weight loss of 3%.
- Ultraviolet - Degradation is very slow – ¼" per century – no serious effects on properties

Adjustments to Tabulated Design Values (standard conditions):

- When actual conditions vary from standard conditions, tabulated values must be adjusted by the appropriate factors.

"Factors":
- C_M – Moisture content (wet use factor)
- C_D – Duration of load
- C_F – Size factor
- C_f – Form factor
- C_L – Lateral stability of beams
- C_P – Lateral stability of columns

Applicable Factors for Strength Properties:

- **Bending F_b:** C_D, C_M, C_F, C_f, C_L
- **Compression F_c:** C_D, C_M, C_P
- **Horizontal shear F_v:** C_D, C_M
- **Modulus of Elasticity E:** C_M

NOTE: Design values are for 10 year load and dry service conditions

(C_D) – When the load is applied for periods other than 10 yrs, tabulated stresses are multiplied by C_D in AASHTO Table 13.5.5A. For vehicle live load, $C_D = 1.15$

(C_M) - Use when moisture content is expected to exceed 19%, per AASHTO Art. 13.5.5.1.1, C_m values given in footnotes of Table 13.5.1A.

(C_F) - For rectangular sawn lumber, bending members 5 inches or thicker and greater than 12" in depth, reference AASHTO 13.6.4.2.2
$C_F = (12/d)^{1/9}$
d = member depth in inches

(C_f) - Form factor, tabulated bending stresses are based on members with square and rectangular cross sections. For round sections, stresses must be modified by the form factor C_f per AASHTO 13.6.4.5.
$C_f = 1.18$

(C_L) - When members are not adequately braced, tabulated bending stress shall be modified by C_L. When there is no danger of lateral buckling, $C_L = 1.0$.
- Other cases use AASHTO 13.6.4.4.3

(C_P) - When a compression member is supported throughout its length to prevent displacement, $C_P = 1.0$ per AASHTO 13.7.3.3.2

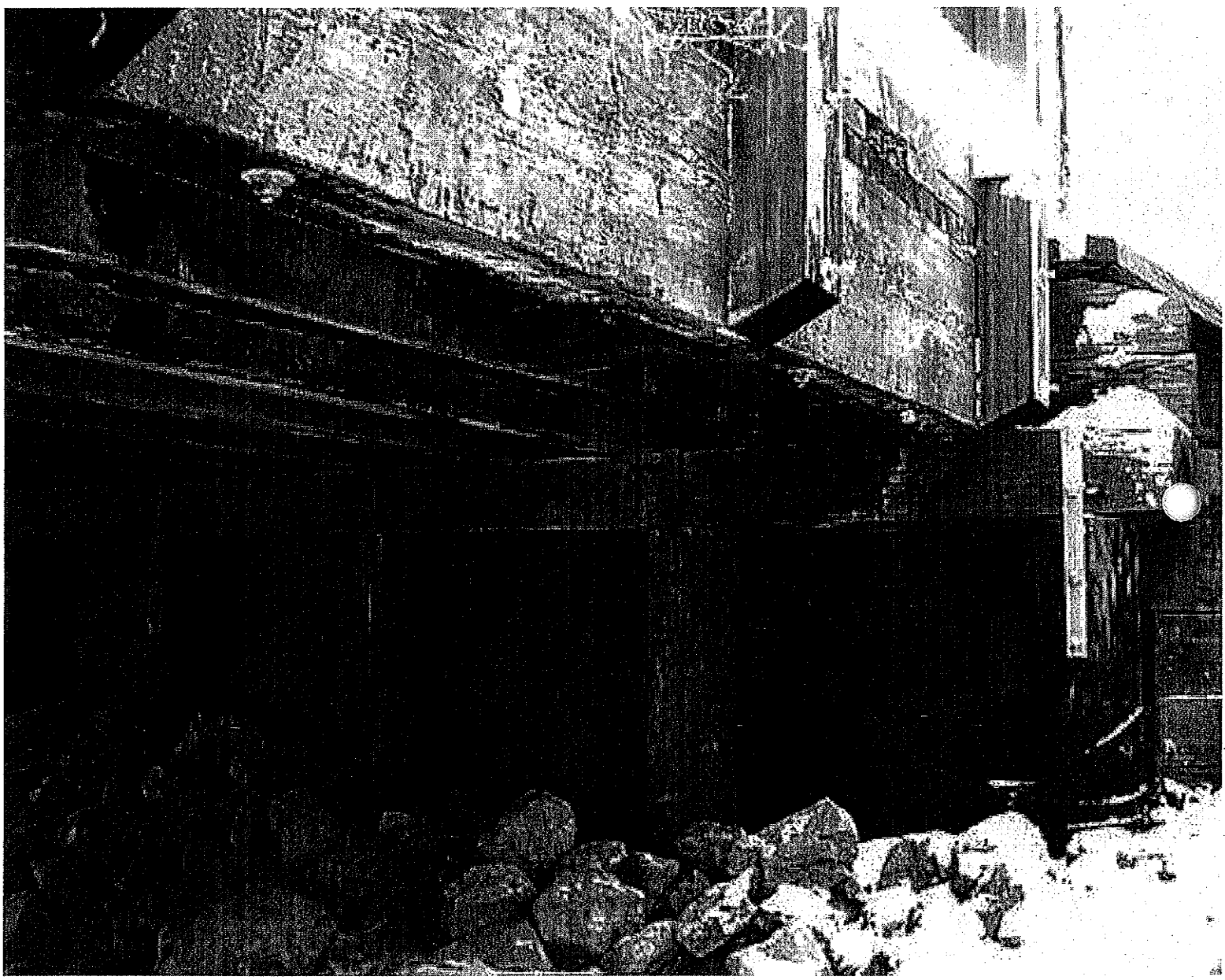


Photo 1

Photo 2

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2008 BRIDGE RATING CLASS 101

DESIGN OF TIMBER SUPERSTRUCTURE BASICS

LOADS

Dead Loads (permanent wt)

- Steel - 490 lb/ft
- Earth - 120 lb/ft
- Compacted gravel - 140 lb/ft
- Timber - 50 lb/ft
- Bit. - 150 lb/ft
- Conc. - 150 lb/ft

- Dead loads are commonly assumed to be uniformly distributed along the length of a structural member (beam, deck panel, etc).

Live Loads (vehicular)

Terms:
- Gross vehicle wt (GVW) – is the maximum total weight of vehicle
- Axle load – is the total weight transferred through one axle
- Axle spacing – is center-to-center distance between axles
- Wheel load – is one half the axle load
- Wheel line – is series of wheel loads measured along the vehicle length, one-half GVW
- Track width – is the center-to-center distance between wheel lines

Standard vehicle loads

- AASHTO specifications provide 2 systems of standard vehicle loads: H loads and HS loads. Each system consists of individual truck loads or lane loads.

H Truck loads:

H 15-44
H 20-44

- Represents a two axle truck designated by the letter H followed by the first number indicating the GVW in tons.

- The second number "44" indicates the year the load was adopted by AASHTO (1944). The weight of an H truck assumes $2/10^{th}$ weight to front axle, $8/10^{th}$ to rear axle. Axles are spaced at 14' with track width of 6'.

HS Truck loads:

HS – 15-44
HS – 20-44

- Represent a two axle truck with a one-axle semi-trailer. The configuration and weight of the HS truck is identical to the corresponding H load. The additional semi-trailer axle is equal in weight to the rear truck axle and is spaced at a variable distance of 14 to 30'.

Traffic Lanes:

- Vehicle live loads are applied in design traffic lanes that are 12' wide. The number of traffic lanes depends on the width of the bridge roadway. Fractional parts of design lanes are not permitted.

- Each traffic lane is loaded with one standard truck or one standard lane load, regardless of the bridge length or number of spans. The standard loads occupy a 10' width within the lane. Traffic lanes and vehicle loads in lanes are positioned laterally on the bridge to produce the maximum stress. Traffic lanes cannot overlap.

- The number of traffic lanes to be loaded should be in conformance with AASHTO and the following:

- Rdwy 18' to 20', should have two design lanes
- Rdwy less than 18' should carry one traffic lane

Reduction in Load Intensity

- When live load stress is based on a number of traffic lanes simultaneously loaded, the following percentages of live loads may be used in view of the improbability:

- One or two lanes – 100%
- Three lanes – 90%
- Four lanes or more – 75%

- AASHTO also specifies an alternate military loading. This hypothetical loading consists of two 24,000 lb axles spaced 4' apart. There is no lane load for military loading.

Lane Loads:

- Adopted by AASHTO to provide a simpler method of calculating moments and shears to simulate a train of trucks on the bridge.

- AASHTO specs include a lane load for HS20-44. The uniform load is 640 lbs per linear foot of load lane with concentrated loads for shear and moment.

Modification to Standard Loads

- There may be instances when the standard vehicle loads do not accurately represent the design loading required for a bridge.

- For example, in 1989 live loading for State Aid bridges was revised to HS25 from the previous HS20 loading. Since 1944, when HS20 was established, truck weights have continued to increase along with allowable legal loads in response to trucking industry requests. In fact, the legal load in Minnesota allows trucks which exceed the HS20 design load by up to 10%.

- FYI: Cost imports of revising to HS25 resulted in a weighted average cost increase of 4.0% for bridges.

Distribution of Live Loads

- The fraction of live load transferred to a single member should be per AASHTO design specs. See AASHTO table 3.23.1 Distribution of Wheel Loads.

- Ref. AASHTO art. 3.25.2 Live Load Distribution for plank and nail laminated longitudinal flooring.

Impact (Dynamic / Vibratory Effects)

- Impact should be added to the live load for rating per AASHTO 3.8.

- Note: Because timber's ability to absorb shock and loads of short duration, AASHTO does not require an impact factor for timber bridges (AASHTO standard specs only).

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MINNESOTA DEPARTMENT OF TRANSPORTATION

Sheet No. 1 of 4
Bridge No. 1895
Made By Beam
Check By _____
Date 3/4/08

ASR TIMBER LONGITUDINAL NAIL LAMINATED DECK RATING WORKSHEET

Location: Rye Rd. over Bourbon Creek

Given information:
- A simple span nail laminated bridge, two lanes.
- 32'-1" roadway, Span length = 20'-10"
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Deck in new condition
- Year built 1966
- Average wear course thickness of 3.5"
- 0 degree skew
- Timber species: Douglas fir-larch No. 2, (coastal region)

Unit Definitions

$k \equiv 1000 \cdot lbf$ $ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $klf \equiv \frac{1000 \cdot lbf}{ft}$ $kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $kft \equiv 1000 \cdot lbf \cdot ft$ $ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $ton \equiv 2000 \cdot lbf$

Input

Reference AASHTO Table 13.5.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: No. 2
Size Class: 2" and wider

$F_B := 875 \cdot psi$ (bending)

$F_{Vpar} := 95 \cdot psi$ (shear parallel to grain)

$E := 1600000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$Span := 20.83 \cdot ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 32.083 \cdot ft$ (Width of roadway)

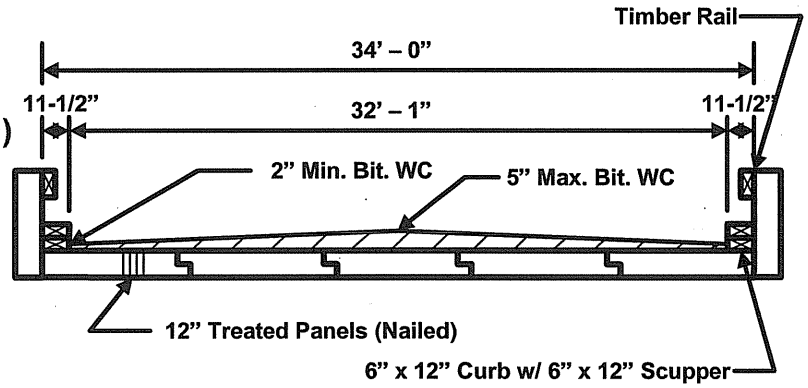
$Width_{rail} := 11.5 \cdot in$ (Width of curb)

$Thick_{deck} := 12 \cdot in$ (Thickness of deck)

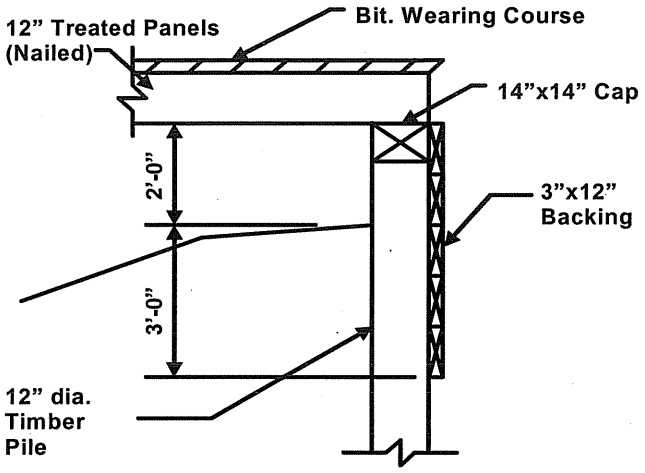
$W_{TimRail} := 70 \cdot plf$ (Weight of timber rail)

$Thick_{WC} := 3.5 \cdot in$ (Thickness of Bituminous Wearing Course)

$Width_{abutcap} := 14 \cdot in$ (Width of abutment cap)



SECTION THRU TIMBER DECK



SECTION THRU ABUTMENT

Determine allowable unit stress for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_r$$

Timber Adjustment Factors

C_D = Load duration factor - Table 13.5.5A (Veh LL, 2 months)

$C_D := 1.15$

C_F = Bending Size factor - Art. 13.5.1A (4" x 14" lumber)

$C_F := 1.0$

C_M = Wet service factor - Per Table 13.5.1A (Assume all bridge timbers to exceed 19% moisture)

$C_{Mb} := \text{if}(F_B \cdot C_F \le 1150 \cdot \text{psi}, 1.0, 0.85)$

$C_{Mb} = 1$ **(For Bending)**

$C_{Mv} := 0.97$ **(For Shear)**

C_r = Repetitive member factor - Bending design value for 2" to 4" thick members in contact

$C_r := 1.15$

Allowable unit stress for bending

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F \cdot C_r$ $\quad F_{Bprime} = 1157.19\,\text{psi}$

Determine allowable unit stress for shear

$$F_V' = F_V \times C_M \times C_D$$

$F_{Vprime} := F_{Vpar} \cdot C_{Mv} \cdot C_D$ $\quad F_{Vprime} = 105.97\,\text{psi}$

Determine Effective Span Length (3.25.2.3)

$L_{effmin} := \text{Span} - \text{Width}_{abutcap} + \frac{\text{Width}_{abutcap}}{2}$ **Clear distance between abutment caps + 1/2 cap width**

$L_{effmin} = 20.25\,\text{ft}$

$L_{effmax} := \text{Span} - \text{Width}_{abutcap} + \text{Thick}_{deck}$ **Clear distance between abutment caps + floor thickness**

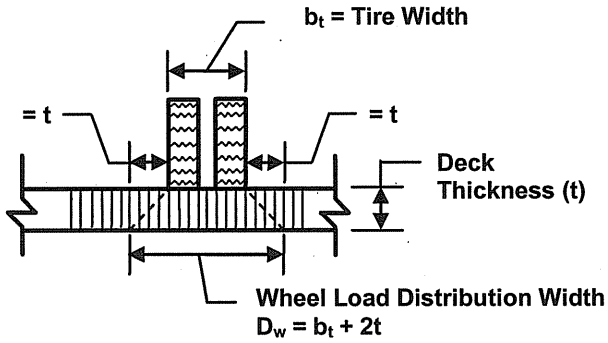
$L_{effmax} = 20.66\,\text{ft}$

$L_{eff} := \text{if}(L_{effmin} < L_{effmax}, L_{effmin}, L_{effmax})$ **Effective Span Length (AASHTO 3.25.2.3)**

$L_{eff} = 20.25 \cdot \text{ft}$

Compute Live Load Distribution Width per AASHTO 3.25.2.2

Normal to direction of the span, wheel load shall be distributed as follows.
Plank floor 20" max.



$P_{wheel} := 16000 \cdot lbf$ **(HS20 wheel load)**

$TCA := \frac{0.01}{1 \cdot psi} \cdot P_{wheel}$ $TCA = 160 \cdot in^2$ **Tire Contact Area (3.30)**

$b_t := \sqrt{2.5 \cdot TCA}$ **(Width of tire contact transverse to span for HS20-44 truck)** $b_t = 20\,in$

$D_w := b_t + (2 \cdot Thick_{deck})$ **(Wheel load distribution width transverse to deck)** $D_w = 44\,in$
($D_w = b_t$ + [2 x Deck Thickness])

Dead Loads

DL of Timber Deck $DL_{deck} := D_w \cdot Thick_{deck} \cdot Dens_{timb}$ $DL_{deck} = 183.33\,plf$

DL of Rail **(Assume dead load of railing system is uniformly distributed across the deck width)**

$DL_{rail} := \frac{W_{TimRail} \cdot 2 \cdot D_w}{Width_{rdwy} + (Width_{rail} \cdot 2)}$ $DL_{rail} = 15.1\,plf$

DL of Wearing Course $DL_{WC} := D_w \cdot Thick_{WC} \cdot Dens_{bit}$ $DL_{WC} = 160.42\,plf$

Total Dead Load of superstructure $DL_T := DL_{deck} + DL_{rail} + DL_{WC}$ $DL_T = 358.85\,plf$

Compute Dead Load Moment

$M_{DL} := \frac{DL_T \cdot L_{eff}^2}{8}$ $M_{DL} = 18.39\,ft\,k$ **or**

$M_{DL} = 220652.62\,in \cdot lbf$

Compute Live Load Moment

HS20 Truck Moment per AASHTO App. A (Note that Live load moments in App. A are based on one lane without impact)

For Span = 20 feet $M_{LL1} := 160 \cdot kft$

For Span = 21 feet $M_{LL2} := 168 \cdot kft$

Therefore for Span = 20.25 feet $M_{LL} := 168 \cdot kft - \left[\frac{21 \cdot ft - L_{eff}}{21 \cdot ft - 20 \cdot ft} \cdot (M_{LL2} - M_{LL1})\right]$

$M_{LL} = 161.97\,kft$ **or**

$M_{LL} = 1943680\,in \cdot lbf$

$M_{LLWheel} := \frac{M_{LL}}{2}$ **(Moment per wheel)** $M_{LLWheel} = 971840\,in \cdot lbf$

Section Modulus of deck

Section Modulus : $S_{deck} := \frac{D_w \cdot Thick_{deck}^2}{6}$ $S_{deck} = 1056\,in^3$

Calculate Inventory and Operating Ratings of deck based on flexure

Calculate Dead and Live Load bending stresses

$f_{bDL} := \frac{M_{DL}}{S_{deck}}$ **(Dead load bending stress)** $f_{bDL} = 208.95\,psi$

$f_{bLL} := \frac{M_{LLWheel}}{S_{deck}}$ **(Live load bending stress)** $f_{bLL} = 920.3\,psi$

Calculate Inventory and Operating Rating Factors for bending in deck

$RF_{Ideck} := \frac{F_{Bprime} - f_{bDL}}{f_{bLL}}$ $RF_{Ideck} = 1.03$

$RF_{Odeck} := \frac{(F_{Bprime} \cdot 1.33) - f_{bDL}}{f_{bLL}}$ $RF_{Odeck} = 1.45$

Inventory Rating

$Inv_{Rating} := RF_{Ideck} \cdot 20$ **HS** $Inv_{Rating} = 21$

Operating Rating

$Op_{Rating} := RF_{Odeck} \cdot 20$ **HS** $Op_{Rating} = 29$

Note: Rating for shear will not control over rating for flexure when rating nail laminated bridge decks

MINNESOTA DEPARTMENT OF TRANSPORTATION

Sheet No. 1 of 4
Bridge No. 1895
Made By Beam
Check By _____
Date 3/4/08

ASR TIMBER LONGITUDINAL NAIL LAMINATED DECK RATING WORKSHEET #2

(Missing spreader beams)

Location: Rye Rd. over Bourbon Creek

Given information:

- A simple span nail laminated bridge, two lanes.
- 32'-1" roadway, Span length = 20'-10"
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built 1966
- Average wear course thickness of 6.5"
- 0 degree skew
- Timber species: Douglas fir-larch No. 2, (coastal region)
- Deck is found to be missing spreader beams and there is evidence of loss of inter-connection between nail laminated lumber
- Bituminous wearing course is cracked or otherwise broken from excessive transverse and longitudinal deflections.
- Engineer recomputes Live Load distribution width assuming non-interconnected nail laminated floor

Unit Definitions

$k \equiv 1000 \cdot lbf$ $ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $klf \equiv \frac{1000 \cdot lbf}{ft}$ $kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $kft \equiv 1000 \cdot lbf \cdot ft$ $ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $ton \equiv 2000 \cdot lbf$

Input

Reference AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)

Commercial Grade: No. 2

Size Class: 2" and wider

$F_B := 875 \cdot psi$ (bending)

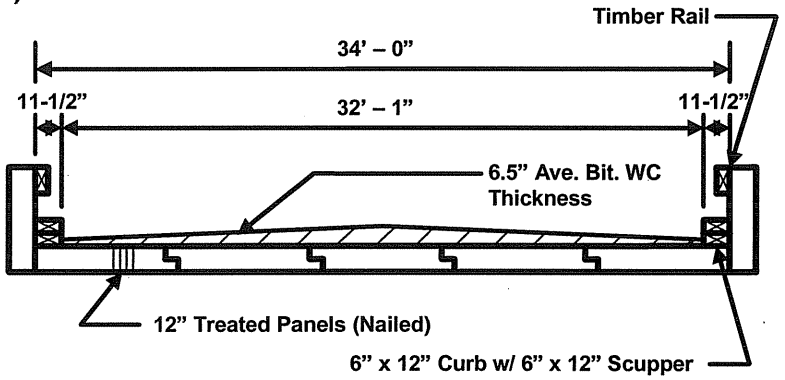
$F_{Vpar} := 95 \cdot psi$ (shear parallel to grain)

$E := 1600000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$Span := 20.83 \cdot ft$ (Span length, CL bearing to CL bearing)



SECTION THRU TIMBER DECK

Input (cont)

$Width_{rdwy} := 32.083 \cdot ft$ (Width of roadway)

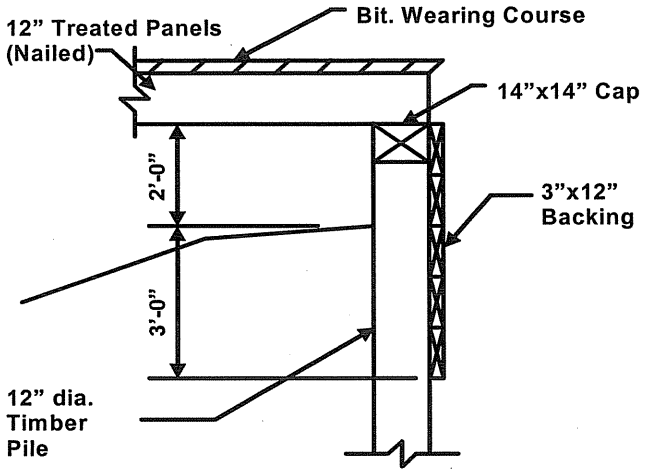
$Width_{rail} := 11.5 \cdot in$ (Width of curb)

$Thick_{deck} := 12 \cdot in$ (Thickness of deck)

$W_{TimRail} := 70 \cdot plf$ (Weight of timber rail)

$Thick_{WC} := 6.5 \cdot in$ (Thickness of Bituminous Wearing Course)

$Width_{abutcap} := 14 \cdot in$ (Width of abutment cap)



SECTION THRU ABUTMENT

Determine allowable unit stress for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_r$$

Timber Adjustment Factors

C_D = Load duration factor - Table 13.5.5A (Veh LL, 2 months)

$C_D := 1.15$

C_F = Bending Size factor - Art. 13.5.1A (4" x 14" lumber)

$C_F := 1.0$

C_M = Wet service factor - Per Table 13.5.1A (Assume all bridge timbers to exceed 19% moisture)

$C_{Mb} := \text{if}\left(F_B \cdot C_F \le 1150 \cdot psi, 1.0, 0.85\right)$

$C_{Mb} = 1$ **(For Bending)**

$C_{Mv} := 0.97$ **(For Shear)**

C_r = Repetitive member factor - Bending design value for 2" to 4" thick members in contact

$C_r := 1.15$

Allowable unit stress for bending

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F \cdot C_r$ $\quad F_{Bprime} = 1157.19\,psi$

Determine allowable unit stress for shear

$$F_V' = F_V \times C_M \times C_D$$

$F_{Vprime} := F_{Vpar} \cdot C_{Mv} \cdot C_D$ $\quad F_{Vprime} = 105.97\,psi$

Determine Effective Span Length (3.25.2.3)

$L_{effmin} := Span - Width_{abutcap} + \frac{Width_{abutcap}}{2}$ **Clear distance between abutment caps + 1/2 cap width**

$L_{effmin} = 20.25\,ft$

$L_{effmax} := Span - Width_{abutcap} + Thick_{deck}$ **Clear distance between abutment caps + floor thickness**

$L_{effmax} = 20.66\,ft$

$L_{eff} := if\left(L_{effmin} < L_{effmax}, L_{effmin}, L_{effmax}\right)$ **Effective Span Length (AASHTO 3.25.2.3)**

$L_{eff} = 20.25 \cdot ft$

Compute Live Load Distribution Width per AASHTO 3.25.2.2

Normal to direction of the span, wheel load shall be distributed as follows. Plank floor 20" max.

$P_{wheel} := 16000 \cdot lbf$ **(HS20 wheel load)**

$TCA := \frac{0.01}{1 \cdot psi} \cdot P_{wheel}$ $TCA = 160 \cdot in^2$ **Tire Contact Area (3.30)**

$b_t := \sqrt{2.5 \cdot TCA}$ **(Width of tire contact transverse to span for HS20-44 truck)** $b_t = 20\,in$

$D_w := b_t + \left(Thick_{deck}\right)$ **(Wheel load distribution width transverse to deck assuming non-interconnected nail laminated floor) ($D_w = b_t$ + Deck Thickness)** $D_w = 32\,in$

Dead Loads

DL of Timber Deck $DL_{deck} := D_w \cdot Thick_{deck} \cdot Dens_{timb}$ $DL_{deck} = 133.33\,plf$

DL of Rail **(Assume dead load of railing system is uniformly distributed across the deck width)**

$DL_{rail} := \frac{W_{TimRail} \cdot 2 \cdot D_w}{Width_{rdwy} + \left(Width_{rail} \cdot 2\right)}$ $DL_{rail} = 10.98\,plf$

DL of Wearing Course $DL_{WC} := D_w \cdot Thick_{WC} \cdot Dens_{bit}$ $DL_{WC} = 216.67\,plf$

Total Dead Load of superstructure $DL_T := DL_{deck} + DL_{rail} + DL_{WC}$ $DL_T = 360.98\,plf$

Compute Dead Load Moment

$M_{DL} := \frac{DL_T \cdot L_{eff}^2}{8}$ $M_{DL} = 18.5\,ft\,k$ **or**

$M_{DL} = 221963.76\,in \cdot lbf$

Compute Live Load Moment

HS20 Truck Moment per AASHTO App. A (Note that Live load moments in App. A are based on one lane without impact)

For Span = 20 feet $M_{LL1} := 160 \cdot kft$

For Span = 21 feet $M_{LL2} := 168 \cdot kft$

Therefore for Span = 20.25 feet $M_{LL} := 168 \cdot kft - \left[\frac{21 \cdot ft - L_{eff}}{21 \cdot ft - 20 \cdot ft} \cdot (M_{LL2} - M_{LL1})\right]$

$M_{LL} = 161.97\,kft$ **or**

$M_{LL} = 1943680\,in \cdot lbf$

$M_{LLWheel} := \frac{M_{LL}}{2}$ **(Moment per wheel)** $M_{LLWheel} = 971840\,in \cdot lbf$

Section Modulus of deck

Section Modulus : $S_{deck} := \frac{D_w \cdot Thick_{deck}^{\,2}}{6}$ $S_{deck} = 768\,in^3$

Calculate Inventory and Operating Ratings of deck based on flexure

Calculate Dead and Live Load bending stresses

$f_{bDL} := \frac{M_{DL}}{S_{deck}}$ **(Dead load bending stress)** $f_{bDL} = 289.02\,psi$

$f_{bLL} := \frac{M_{LLWheel}}{S_{deck}}$ **(Live load bending stress)** $f_{bLL} = 1265.42\,psi$

Calculate Inventory and Operating Rating Factors for bending in deck

$RF_{Ideck} := \frac{F_{Bprime} - f_{bDL}}{f_{bLL}}$ $RF_{Ideck} = 0.69$

$RF_{Odeck} := \frac{(F_{Bprime} \cdot 1.33) - f_{bDL}}{f_{bLL}}$ $RF_{Odeck} = 0.99$

Inventory Rating

$Inv_{Rating} := RF_{Ideck} \cdot 20$ **HS** $Inv_{Rating} = 14$

Operating Rating

$Op_{Rating} := RF_{Odeck} \cdot 20$ **HS** $Op_{Rating} = 20$

Note: Rating for shear will not control over rating for flexure when rating nail laminated bridge decks

- Since the Operating Rating is very close to HS20, you may need to consider posting

Workshop Agenda

VIII. Load Rating Example #2
- Simple Span Nail Laminated Timber Deck (without distress)

IX. Class Exercise
- Simple Span Nail Laminated Timber Deck (with distress)

X. Review of worked out examples

XI. Submittal to MN/DOT and Review of Process

XII. Common mistakes and questions

Class Exercise

- Timber Longitudinal Nail Laminated Deck Bridge
- Given Information:
  - Simple span nail laminated bridge, two lanes.
  - 32'-1" roadway, Span length = 20'-10"
  - Timber dimensions were field measured (actual)
  - Good maintenance and inspection
  - Year built 1966

Class Exercise

- Given Information (cont.):
  - County Engineer confirmed from project files and field verification that the bridge deck actually has an average wearing course thickness of 10".

Class Exercise

- Given Information (cont.):
  - The bituminous wearing course appears to be showing fresh transverse and longitudinal cracks based on periodic inspections.

Class Exercise

- Given Information (cont.):
  - Preliminary rating studies showed that bridge posting was necessary. The County engineer decided to utilize participants of this class to confirm the posting loads for this bridge.

Class Exercise

- Given Information (cont.):
  - Per the request of the County engineer, please break into 5 separate groups with each group being assigned an important rating/posting task. This County Engineer appreciates your help!

Class Exercise

- Group 1
  - Calculate the wheel load distribution width and Section Modulus for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks. Determine operating stress.
- Group 2
  - Calculate the dead load moment and flexural stress for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks.

Class Exercise

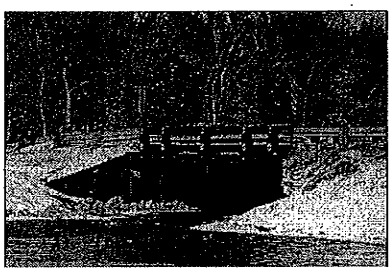
- Group 3
  - Calculate the live load moment and flexural stress for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks.
- Group 4
  - Determine Operating Ratings for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks.

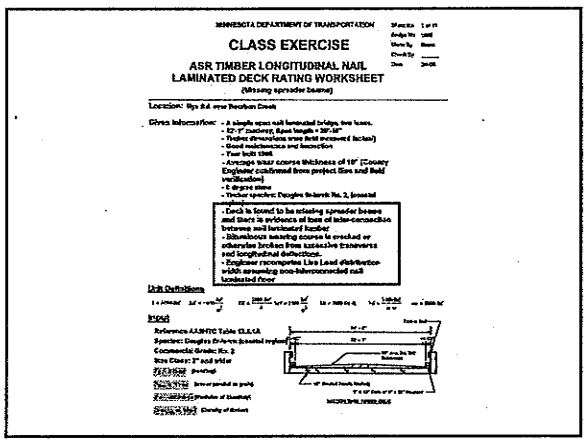
Class Exercise

- Group 5
  - Determine posting requirements for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks. Complete Form RC-CL "Load Posting Report" and R12-5 sign.

Class Exercise Solution

Simple Span Nail Laminated Timber Deck (with distress)





MINNESOTA DEPARTMENT OF TRANSPORTATION

CLASS EXERCISE

ASR TIMBER LONGITUDINAL NAIL LAMINATED DECK RATING WORKSHEET

(Missing spreader beams)

Location: Hyd #4 over Beaches Creek

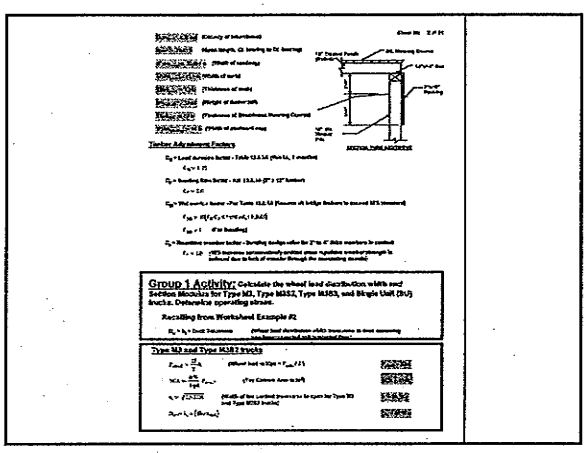
Given Information:
- A simple span nail laminated bridge, two lanes.
- 52-1" nailed, fixed length = 30'-10"
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built 1966
- Average wear course thickness of 10" (County Engineer confirmed from project files and field verification)
- 0 degree skew
- Timber species: Douglas fir-larch No. 2, (assumed species)

- Deck is found to be missing spreader beams and there is evidence of loss of inter-connection between nail laminated layers
- Bituminous wearing course is cracked or otherwise broken from excessive transverse and longitudinal deflections.
- Engineer recognizes Live Load distribution with assuming non-interconnected nail laminated floor

Unit Definitions

Input

Reference AASHTO Table 13.5.1A
Species: Douglas fir-larch (assumed region)
Commercial Grade: No. 2
Size Class: 2" and wider
[illegible]
[illegible]
[illegible]
[illegible]



[illegible]

Timber Adjustment Factors

C_F = Load duration factor - Table 13.5.1A (Note 1, 2 months)

$C_F = 1.15$

C_M = Bending Size factor - Eq 13.5.1A-2 (2" < 12" bottom)

$C_M = 1.0$

C_{fu} = Flat use factor - Per Table 13.5.1A (assume all bridge decks to exceed 10% incised)

$C_{fu} = 1.0$ (Flat bending)

C_i = Incising factor - Bending design value for 2" to 4" thick members is unaffected

$C_i = 1.0$ (NDS incising correspondingly applied when equation member strength is reduced due to lack of member through the measuring month)

Group 1 Activity: Calculate the wheel load distribution width and Section Modulus for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks. Determine operating stress.

Recalling from Worksheet Example #2

D_L = b_L = Deck Thickness (Wheel load distribution width transverse to deck assuming non-interconnected nail laminated deck)

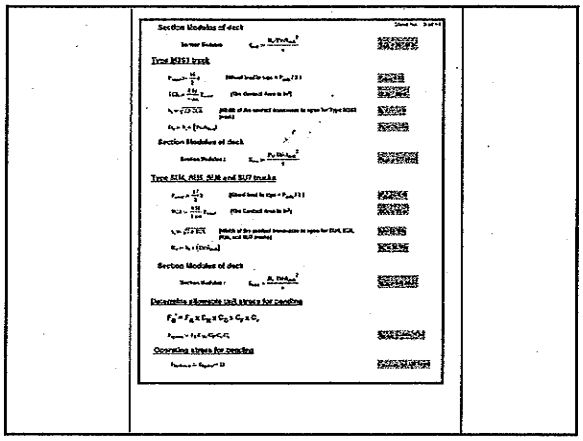
Type M3 and Type M3S2 trucks

$P_{wheel} = \frac{P}{2}$ (Wheel load in kips = $P_{axle}/2$)

$WA = \frac{wb}{Pw}$ P_{wheel} (Tire Contact Area in in²)

$w = \sqrt{WA/b}$ (Width of tire contact area to be used for Type M3 and Type M3S2 trucks)

$D_{wheel} = b + (2w/3)$



Section Modulus of deck

Interior Section $S_{int} = \frac{b_e \cdot (2t_s + t_{ac})^2}{6}$

Type M161 truck

$P_{wheel} = \frac{W_1}{2}$ (Wheel load for type = P_{wheel} / 1)

$W_1 = \frac{A_{tire}}{t_{tire}} P_{wheel}$ (Tire Contact Area in in²)

$b_e = \sqrt{12 \cdot S_{int}}$ (Width of the overlaid transverse to span for Type M161 trucks)

$S_{int} = b_e \cdot (2t_s + t_{ac})$

Section Modulus of deck

Section Modulus : $S_{int} = \frac{b_e \cdot (2t_s + t_{ac})^2}{6}$

Type M164, M135, M144 and SU7 trucks

$P_{wheel} = \frac{W_1}{2}$ (Wheel load in type = P_{wheel} / 1)

$W_1 = \frac{A_{tire}}{t_{tire}} P_{wheel}$ (Tire Contact Area in in²)

$b_e = \sqrt{12 \cdot S_{int}}$ (Width of the overlaid transverse to span for M164, M135, M144, and SU7 trucks)

$S_{int} = b_e \cdot (2t_s + t_{ac})$

Section Modulus of deck

Section Modulus : $S_{int} = \frac{b_e \cdot (2t_s + t_{ac})^2}{6}$

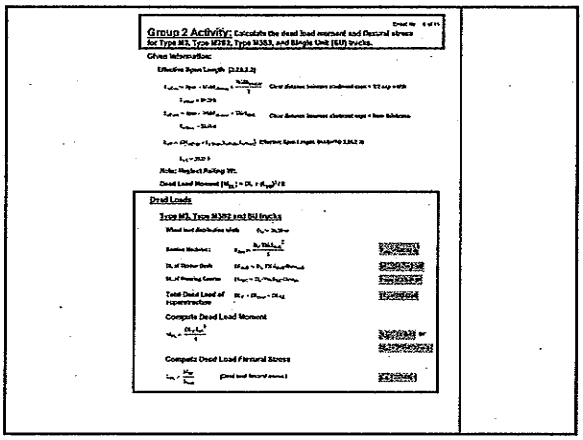
Determine allowable unit stress for decking

$F_d' = F_d \times C_{FE} \times C_D \times C_F \times C_r$

$F_{decking} = 1.5 \times 1.0 \times 0.75 \times 1.0$ = 1.125 ksi

Governing stress for decking

$f_{decking} = f_{Type M}$ = 0.9 ksi



Group 2 Activity: Calculate the dead load moment and flexural stress for Type M3, Type M3S3, Type M3S3, and Single Unit (SU) trucks.

Given Information:

Effective Span Length: [2.2.3.2.2]

$t_{slab} = 8$ in

$t_{wearing} = 2.5$ in

$L_{eff} = 20.0$ ft

Note: Neglect Railing Wt.

Dead Load Moment (M_{DL}) = DL + $(M_{DL})^2$/8

Dead Loads

Type M3, Type M3S2 and SU truck

Wheel load distribution width $S_w = 24.0$ in

Section Modulus: $S_{int} = \frac{b_e \cdot (2t_s + t_{ac})^2}{6}$

DL of Stringer Deck $DL_{deck} = b_e \cdot 2t_s \cdot \gamma_{conc}$

DL of Wearing Course $DL_{wc} = b_e \cdot t_{ac} \cdot \gamma_{wc}$

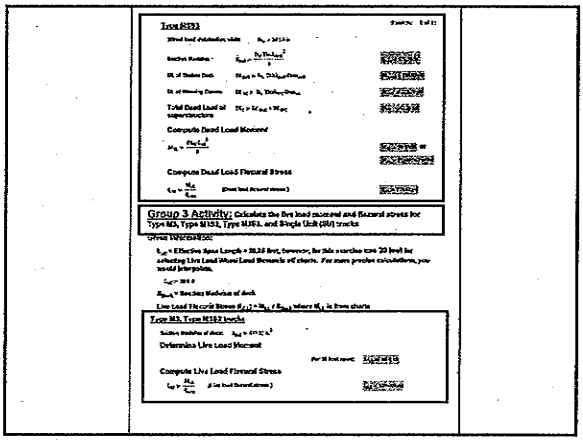
Total Dead Load of superstructure $DL_T = DL_{deck} + DL_{wc}$

Compute Dead Load Moment

$M_{DL} = \frac{DL_T \cdot L_{eff}^2}{8}$

Compute Dead Load Flexural Stress

$f_{DL} = \frac{M_{DL}}{S_{int}}$ (Dead load flexural stress)



Type M153

Wheel load distribution width $S_w = 24.0$ in

Section Modulus: $S_{int} = \frac{b_e \cdot (2t_s + t_{ac})^2}{6}$

DL of Stringer Deck $DL_{deck} = b_e \cdot 2t_s \cdot \gamma_{conc}$

DL of Wearing Course $DL_{wc} = b_e \cdot t_{ac} \cdot \gamma_{wc}$

Total Dead Load of superstructure $DL_T = DL_{deck} + DL_{wc}$

Compute Dead Load Moment

$M_{DL} = \frac{DL_T \cdot L_{eff}^2}{8}$

Compute Dead Load Flexural Stress

$f_{DL} = \frac{M_{DL}}{S_{int}}$ (Dead load flexural stress)

Group 3 Activity: Calculate the live load moment and flexural stress for Type M3, Type M153, Type M3S3, and Single Unit (SU) trucks.

Given Information:

L_{eff} = Effective Span Length = 20.00 feet, however, for this exercise use 20 feet for selecting Live Load Moment Moment's of charts. For more precise calculations, you would interpolate.

$t_{slab} = 20.0$ in

S_{int} = Section Modulus of deck

Live Load Flexural Stress $f_{LL} = M_{LL} / S_{int}$ where M_{LL} is from charts

Type M3, Type M152 trucks

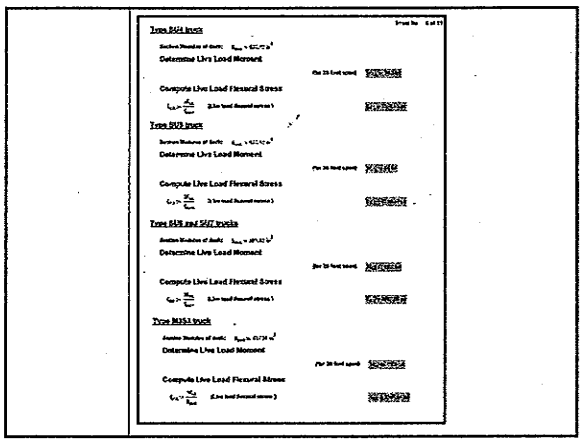
Section Modulus of deck: $S_{int} = 672.0$ in³

Determine Live Load Moment

M_{LL} = 38.0 foot-kips/ft

Compute Live Load Flexural Stress

$f_{LL} = \frac{M_{LL}}{S_{int}}$ (Live load flexural stress)



Type SU4 truck

Determine Live Load Moment

Compute Live Load Flexural Stress

Type SU5 truck

Determine Live Load Moment

Compute Live Load Flexural Stress

Type SU6 and SU7 trucks

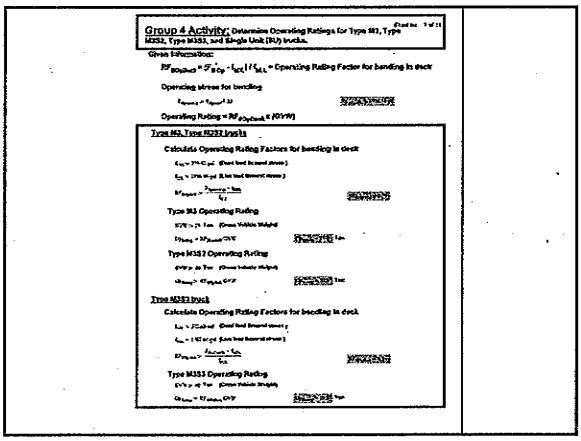
Determine Live Load Moment

Compute Live Load Flexural Stress

Type M343 truck

Determine Live Load Moment

Compute Live Load Flexural Stress



Group 4 Activity: Determine Operating Ratings for Type 3S1, Type M332, Type M333, and Single Unit (SU) trucks.

Given Information:

$RF_{Operating} = \sigma_{allow} - f_{DC} / f_{LL}$ = Operating Rating Factor for bending in deck

Operating stress for bending

Operating Rating = $RF_{Operating} \times (GVW)$

Type M3, Type M333 trucks

Calculate Operating Rating Factors for bending in deck

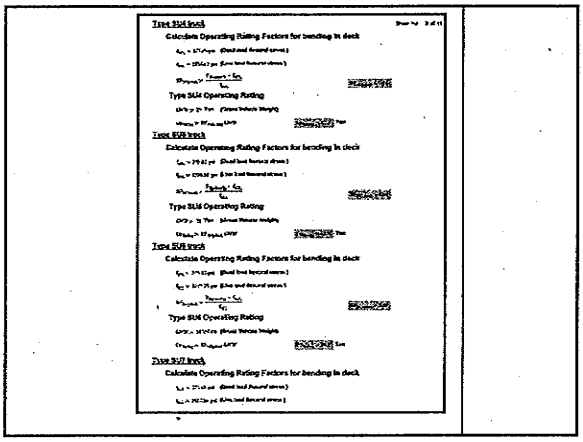
Type 3S3 Operating Rating

Type M332 Operating Rating

Type M343 truck

Calculate Operating Rating Factors for bending in deck

Type M333 Operating Rating



Type SU4 truck

Calculate Operating Rating Factors for bending in deck

Type SU4 Operating Rating

Type SU5 truck

Calculate Operating Rating Factors for bending in deck

Type SU5 Operating Rating

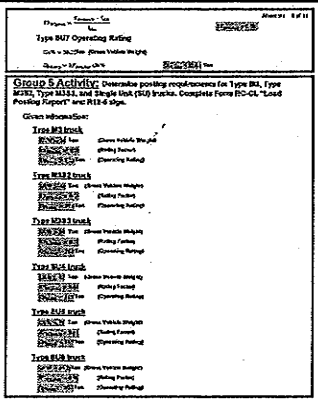
Type SU6 truck

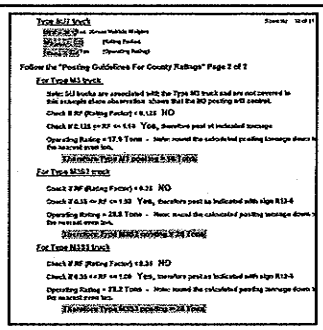
Calculate Operating Rating Factors for bending in deck

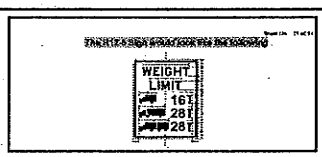
Type SU6 Operating Rating

Type SU7 truck

Calculate Operating Rating Factors for bending in deck


Type BUT Operating Rating
Group 5 Activity: Determine posting requirements for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks. Complete Form RC-CL "Load Posting Report" and R12-6 sign.
Given information:
Type M3 truck
Type M3S2 truck
Type M3S3 truck
Type SU5 truck
Type SU6 truck
Type SU7 truck

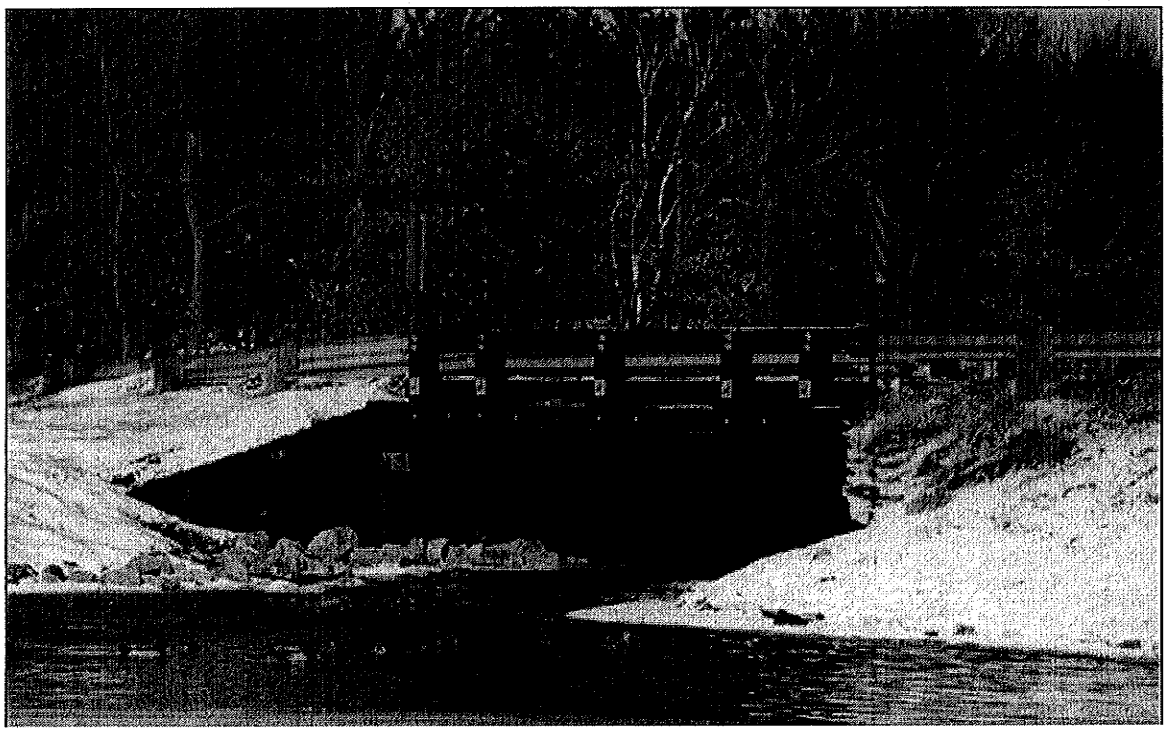

Type SU7 truck
Follow the "Posting Guidelines For County Ratings" Page 2 of 2
For Type M3 truck
For Type M3S2 truck
For Type M3S3 truck


WEIGHT
LIMIT
16T
28T
28T

2008 BRIDGE RATING CLASS 101

CLASS EXERCISE

SIMPLE SPAN NAIL LAMINATED TIMBER DECK WITH DISTRESS



MINNESOTA DEPARTMENT OF TRANSPORTATION

Sheet No. 1 of 11
Bridge No. 1895
Made By Beam
Check By _____
Date 3/4/08

CLASS EXERCISE

ASR TIMBER LONGITUDINAL NAIL LAMINATED DECK RATING WORKSHEET

(Missing spreader beams)

Location: Rye Rd. over Bourbon Creek

Given information:

- A simple span nail laminated bridge, two lanes.
- 32'-1" roadway, Span length = 20'-10"
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built 1966
- Average wear course thickness of 10" (County Engineer confirmed from project files and field verification)
- 0 degree skew
- Timber species: Douglas fir-larch No. 2, (coastal region)
- Deck is found to be missing spreader beams and there is evidence of loss of inter-connection between nail laminated lumber
- Bituminous wearing course is cracked or otherwise broken from excessive transverse and longitudinal deflections.
- Engineer recomputes Live Load distribution width assuming non-interconnected nail laminated floor

Unit Definitions

$k \equiv 1000 \cdot lbf$ $ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $klf \equiv \frac{1000 \cdot lbf}{ft}$ $kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $kft \equiv 1000 \cdot lbf \cdot ft$ $ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $ton \equiv 2000 \cdot lbf$

Input

Reference AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)

Commercial Grade: No. 2

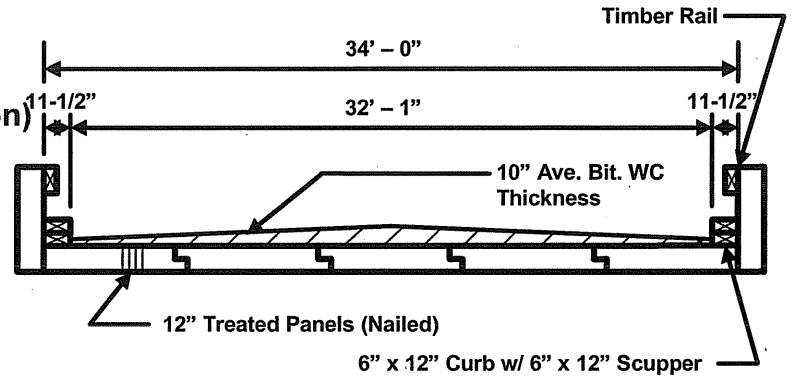
Size Class: 2" and wider

$F_B := 875 \cdot psi$ **(bending)**

$F_{Vpar} := 95 \cdot psi$ **(shear parallel to grain)**

$E := 1600000 \cdot psi$ **(Modulus of Elasticity)**

$Dens_{timb} := 50 \cdot pcf$ **(Density of timber)**



SECTION THRU TIMBER DECK

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$Span := 20.83 \cdot ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 32.083 \cdot ft$ (Width of roadway)

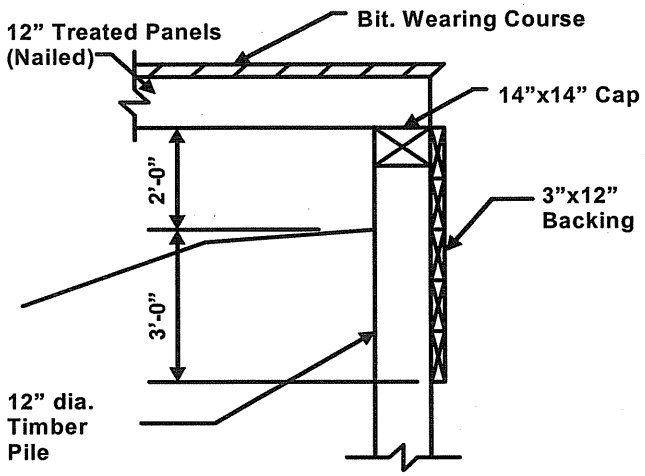
$Width_{rail} := 11.5 \cdot in$ (Width of curb)

$Thick_{deck} := 12 \cdot in$ (Thickness of deck)

$W_{TimRail} := 70 \cdot plf$ (Weight of timber rail)

$Thick_{WC} := 10 \cdot in$ (Thickness of Bituminous Wearing Course)

$Width_{abutcap} := 14 \cdot in$ (Width of abutment cap)



SECTION THRU ABUTMENT

Timber Adjustment Factors

C_D = Load duration factor - Table 13.5.5A (Veh LL, 2 months)

$C_D := 1.15$

C_F = Bending Size factor - Art. 13.5.1A (4" x 14" lumber)

$C_F := 1.0$

C_M = Wet service factor - Per Table 13.5.1A (Assume all bridge timbers to exceed 19% moisture)

$C_{Mb} := if(F_B \cdot C_F \le 1150 \cdot psi, 1.0, 0.85)$

$C_{Mb} = 1$ (For Bending)

C_r = Repetitive member factor - Bending design value for 2" to 4" thick members in contact

$C_r := 1.0$ (15% increase conservatively omitted since repetitive member strength is reduced due to lack of transfer through the connecting dowels)

Group 1 Activity: Calculate the wheel load distribution width and Section Modulus for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks. Determine operating stress.

Recalling from Worksheet Example #2

D_w = b_t + Deck Thickness (Wheel load distribution width transverse to deck assuming non-interconnected nail laminated floor)

Type M3 and Type M3S2 trucks

$P_{wheel} := \frac{17000}{2} \cdot lbf$ (Wheel load in pounds = P_{axle} / 2) $P_{wheel} = 8500\,lbf$

$TCA := \frac{0.01}{1 \cdot psi} \cdot P_{wheel}$ (Tire Contact Area in in^2) $TCA = 85 \cdot in^2$

$b_t := \sqrt{2.5 \cdot TCA}$ (Width of tire contact transverse to span for Type M3 and Type M3S2 trucks) $b_t = 14.58\,in$

$D_w := b_t + (Thick_{deck})$ $D_w = 26.58\,in$

Section Modulus of deck

Section Modulus : $S_{deck} := \frac{D_w \cdot Thick_{deck}^2}{6}$ $S_{deck} = 637.86\,in^3$

Type M3S3 truck

$P_{wheel} := \frac{16000}{2} \cdot lbf$ **(Wheel load in pounds = P_{axle} / 2)** $P_{wheel} = 8000\,lbf$

$TCA := \frac{0.01}{1 \cdot psi} \cdot P_{wheel}$ **(Tire Contact Area in in^2)** $TCA = 80 \cdot in^2$

$b_t := \sqrt{2.5 \cdot TCA}$ **(Width of tire contact transverse to span for Type M3S3 truck)** $b_t = 14.14\,in$

$D_w := b_t + (Thick_{deck})$ $D_w = 26.14\,in$

Section Modulus of deck

Section Modulus : $S_{deck} := \frac{D_w \cdot Thick_{deck}^2}{6}$ $S_{deck} = 627.41\,in^3$

Type SU4, SU5, SU6 and SU7 trucks

$P_{wheel} := \frac{17000}{2} \cdot lbf$ **(Wheel load in pounds = P_{axle} / 2)** $P_{wheel} = 8500\,lbf$

$TCA := \frac{0.01}{1 \cdot psi} \cdot P_{wheel}$ **(Tire Contact Area in in^2)** $TCA = 85 \cdot in^2$

$b_t := \sqrt{2.5 \cdot TCA}$ **(Width of tire contact transverse to span for SU4, SU5, SU6, and SU7 trucks)** $b_t = 14.58\,in$

$D_w := b_t + (Thick_{deck})$ $D_w = 26.58\,in$

Section Modulus of deck

Section Modulus : $S_{deck} := \frac{D_w \cdot Thick_{deck}^2}{6}$ $S_{deck} = 637.86\,in^3$

Determine allowable unit stress for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_r$$

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F \cdot C_r$ $F_{Bprime} = 1006.25\,psi$

Operating stress for bending

$F_{BprimeOp} := F_{Bprime} \cdot 1.33$ $F_{BprimeOp} = 1338.31\,psi$

Group 2 Activity: Calculate the dead load moment and flexural stress for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks.

Given information:

Effective Span Length (3.25.2.3)

$L_{effmin} := Span - Width_{abutcap} + \frac{Width_{abutcap}}{2}$ **Clear distance between abutment caps + 1/2 cap width**

$L_{effmin} = 20.25\,ft$

$L_{effmax} := Span - Width_{abutcap} + Thick_{deck}$ **Clear distance between abutment caps + floor thickness**

$L_{effmax} = 20.66\,ft$

$L_{eff} := if(L_{effmin} < L_{effmax}, L_{effmin}, L_{effmax})$ **Effective Span Length (AASHTO 3.25.2.3)**

$L_{eff} = 20.25 \cdot ft$

Note: Neglect Railing Wt.

Dead Load Moment (M_{DL}) = DL x $(L_{eff})^2 / 8$

Dead Loads

Type M3, Type M3S2 and SU trucks

Wheel load distribution width $D_w := 26.58 \cdot in$

Section Modulus : $S_{deck} := \frac{D_w \cdot Thick_{deck}^2}{6}$ $S_{deck} = 637.92\,in^3$

DL of Timber Deck $DL_{deck} := D_w \cdot Thick_{deck} \cdot Dens_{timb}$ $DL_{deck} = 110.75\,plf$

DL of Wearing Course $DL_{WC} := D_w \cdot Thick_{WC} \cdot Dens_{bit}$ $DL_{WC} = 276.88\,plf$

Total Dead Load of superstructure $DL_T := DL_{deck} + DL_{WC}$ $DL_T = 387.62\,plf$

Compute Dead Load Moment

$M_{DL} := \frac{DL_T \cdot L_{eff}^2}{8}$ $M_{DL} = 19.86\,ft\,k$ **or** $M_{DL} = 238347.23\,in \cdot lbf$

Compute Dead Load Flexural Stress

$f_{bDL} := \frac{M_{DL}}{S_{deck}}$ **(Dead load flexural stress)** $f_{bDL} = 373.63\,psi$

Type M3S3

Wheel load distribution width $D_w := 26.14 \cdot in$

Section Modulus : $S_{deck} := \frac{D_w \cdot Thick_{deck}^2}{6}$ $S_{deck} = 627.36\,in^3$

DL of Timber Deck $DL_{deck} := D_w \cdot Thick_{deck} \cdot Dens_{timb}$ $DL_{deck} = 108.92\,plf$

DL of Wearing Course $DL_{WC} := D_w \cdot Thick_{WC} \cdot Dens_{bit}$ $DL_{WC} = 272.29\,plf$

Total Dead Load of superstructure $DL_T := DL_{deck} + DL_{WC}$ $DL_T = 381.21\,plf$

Compute Dead Load Moment

$M_{DL} := \frac{DL_T \cdot L_{eff}^2}{8}$ $M_{DL} = 19.53\,ft\,k$ **or** $M_{DL} = 234401.67\,in \cdot lbf$

Compute Dead Load Flexural Stress

$f_{bDL} := \frac{M_{DL}}{S_{deck}}$ **(Dead load flexural stress)** $f_{bDL} = 373.63\,psi$

Group 3 Activity: Calculate the live load moment and flexural stress for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks.

Given information:

L_{eff} = Effective Span Length = 20.25 feet, however, for this exercise use 20 feet for selecting Live Load Wheel Load Moments off charts. For more precise calculations, you would interpolate.

$L_{eff} := 20.0 \cdot ft$

S_{Deck} = Section Modulus of deck

Live Load Flexural Stress (f_{bLL}) = M_{LL} / S_{Deck} where M_{LL} is from charts

Type M3, Type M3S2 trucks

Section Modulus of deck: $S_{deck} := 637.92 \cdot in^3$

Determine Live Load Moment

(for 20 foot span) $M_{LL} := 68.9 \cdot kft$

Compute Live Load Flexural Stress

$f_{bLL} := \frac{M_{LL}}{S_{deck}}$ **(Live load flexural stress)** $f_{bLL} = 1296.09\,psi$

Type SU4 truck

Section Modulus of deck: $S_{deck} := 637.92 \cdot in^3$

Determine Live Load Moment

(for 20 foot span) $M_{LL} := 80.2 \cdot kft$

Compute Live Load Flexural Stress

$f_{bLL} := \frac{M_{LL}}{S_{deck}}$ **(Live load flexural stress)** $f_{bLL} = 1508.65\,psi$

Type SU5 truck

Section Modulus of deck: $S_{deck} := 637.92 \cdot in^3$

Determine Live Load Moment

(for 20 foot span) $M_{LL} := 84 \cdot kft$

Compute Live Load Flexural Stress

$f_{bLL} := \frac{M_{LL}}{S_{deck}}$ **(Live load flexural stress)** $f_{bLL} = 1580.14\,psi$

Type SU6 and SU7 trucks

Section Modulus of deck: $S_{deck} := 637.92 \cdot in^3$

Determine Live Load Moment

(for 20 foot span) $M_{LL} := 88.1 \cdot kft$

Compute Live Load Flexural Stress

$f_{bLL} := \frac{M_{LL}}{S_{deck}}$ **(Live load flexural stress)** $f_{bLL} = 1657.26\,psi$

Type M3S3 truck

Section Modulus of deck: $S_{deck} := 627.36 \cdot in^3$

Determine Live Load Moment

(for 20 foot span) $M_{LL} := 71.5 \cdot kft$

Compute Live Load Flexural Stress

$f_{bLL} := \frac{M_{LL}}{S_{deck}}$ **(Live load flexural stress)** $f_{bLL} = 1367.64\,psi$

Group 4 Activity: Determine Operating Ratings for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks.

Given information:

$RF_{BOpDeck} = (F'_{B\,Op} - f_{bDL}) / f_{bLL}$ = Operating Rating Factor for bending in deck

Operating stress for bending

$F_{BprimeOp} := F_{Bprime} \cdot 1.33$ $F_{BprimeOp} = 1338.31\ psi$

Operating Rating = $RF_{BOpDeck}$ x (GVW)

Type M3, Type M3S2 trucks

Calculate Operating Rating Factors for bending in deck

$f_{bDL} := 373.63 \cdot psi$ **(Dead load flexural stress)**

$f_{bLL} := 1296.09 \cdot psi$ **(Live load flexural stress)**

$$RF_{BOpdeck} := \frac{F_{BprimeOp} - f_{bDL}}{f_{bLL}}$$ $RF_{BOpdeck} = 0.74$

Type M3 Operating Rating

$GVW := 24$ **Ton** **(Gross Vehicle Weight)**

$Op_{Rating} := RF_{BOpdeck} \cdot GVW$ **HS** $Op_{Rating} = 17.9$ **Ton**

Type M3S2 Operating Rating

$GVW := 40$ **Ton** **(Gross Vehicle Weight)**

$Op_{Rating} := RF_{BOpdeck} \cdot GVW$ **HS** $Op_{Rating} = 29.8$ **Ton**

Type M3S3 truck

Calculate Operating Rating Factors for bending in deck

$f_{bDL} := 373.63 \cdot psi$ **(Dead load flexural stress)**

$f_{bLL} := 1367.64 \cdot psi$ **(Live load flexural stress)**

$$RF_{BOpdeck} := \frac{F_{BprimeOp} - f_{bDL}}{f_{bLL}}$$ $RF_{BOpdeck} = 0.71$

Type M3S3 Operating Rating

$GVW := 40$ **Ton** **(Gross Vehicle Weight)**

$Op_{Rating} := RF_{BOpdeck} \cdot GVW$ **HS** $Op_{Rating} = 28.2$ **Ton**

Type SU4 truck

Calculate Operating Rating Factors for bending in deck

$f_{bDL} := 373.63 \cdot psi$ **(Dead load flexural stress)**

$f_{bLL} := 1508.65 \cdot psi$ **(Live load flexural stress)**

$$RF_{BOpdeck} := \frac{F_{BprimeOp} - f_{bDL}}{f_{bLL}}$$

$RF_{BOpdeck} = 0.64$

Type SU4 Operating Rating

$GVW := 27$ **Ton** **(Gross Vehicle Weight)**

$Op_{Rating} := RF_{BOpdeck} \cdot GVW$ **HS** $Op_{Rating} = 17.3$ **Ton**

Type SU5 truck

Calculate Operating Rating Factors for bending in deck

$f_{bDL} := 373.63 \cdot psi$ **(Dead load flexural stress)**

$f_{bLL} := 1580.14 \cdot psi$ **(Live load flexural stress)**

$$RF_{BOpdeck} := \frac{F_{BprimeOp} - f_{bDL}}{f_{bLL}}$$

$RF_{BOpdeck} = 0.61$

Type SU5 Operating Rating

$GVW := 31$ **Ton** **(Gross Vehicle Weight)**

$Op_{Rating} := RF_{BOpdeck} \cdot GVW$ **HS** $Op_{Rating} = 18.9$ **Ton**

Type SU6 truck

Calculate Operating Rating Factors for bending in deck

$f_{bDL} := 373.63 \cdot psi$ **(Dead load flexural stress)**

$f_{bLL} := 1657.26 \cdot psi$ **(Live load flexural stress)**

$$RF_{BOpdeck} := \frac{F_{BprimeOp} - f_{bDL}}{f_{bLL}}$$

$RF_{BOpdeck} = 0.58$

Type SU6 Operating Rating

$GVW := 34.75$ **Ton** **(Gross Vehicle Weight)**

$Op_{Rating} := RF_{BOpdeck} \cdot GVW$ **HS** $Op_{Rating} = 20.2$ **Ton**

Type SU7 truck

Calculate Operating Rating Factors for bending in deck

$f_{bDL} := 373.63 \cdot psi$ **(Dead load flexural stress)**

$f_{bLL} := 1657.26 \cdot psi$ **(Live load flexural stress)**

$$RF_{BOpdeck} := \frac{F_{BprimeOp} - f_{bDL}}{f_{bLL}}$$

$RF_{BOpdeck} = 0.58$

Type SU7 Operating Rating

$GVW := 38.75\,Ton$ **(Gross Vehicle Weight)**

$Op_{Rating} := RF_{BOpdeck} \cdot GVW$ **HS** $Op_{Rating} = 22.6$ **Ton**

Group 5 Activity: Determine posting requirements for Type M3, Type M3S2, Type M3S3, and Single Unit (SU) trucks. Complete Form RC-CL "Load Posting Report" and R12-5 sign.

Given information:

Type M3 truck

$GVW := 24$ **Ton** **(Gross Vehicle Weight)**

$RF_{BOpdeck} := 0.74$ **(Rating Factor)**

$Op_{Rating} := 17.9\,Ton$ **(Operating Rating)**

Type M3S2 truck

$GVW := 40$ **Ton** **(Gross Vehicle Weight)**

$RF_{BOpdeck} := 0.74$ **(Rating Factor)**

$Op_{Rating} := 29.8\,Ton$ **(Operating Rating)**

Type M3S3 truck

$GVW := 40$ **Ton** **(Gross Vehicle Weight)**

$RF_{BOpdeck} := 0.71$ **(Rating Factor)**

$Op_{Rating} := 28.2\,Ton$ **(Operating Rating)**

Type SU4 truck

$GVW := 27$ **Ton** **(Gross Vehicle Weight)**

$RF_{BOpdeck} := 0.64$ **(Rating Factor)**

$Op_{Rating} := 17.3\,Ton$ **(Operating Rating)**

Type SU5 truck

$GVW := 31$ **Ton** **(Gross Vehicle Weight)**

$RF_{BOpdeck} := 0.61$ **(Rating Factor)**

$Op_{Rating} := 18.9\,Ton$ **(Operating Rating)**

Type SU6 truck

$GVW := 34.75\,Ton$ **(Gross Vehicle Weight)**

$RF_{BOpdeck} := 0.58$ **(Rating Factor)**

$Op_{Rating} := 20.2\,Ton$ **(Operating Rating)**

Type SU7 truck

$GVW := 38.75$ Ton (Gross Vehicle Weight)

$RF_{BOpdeck} := 0.58$ (Rating Factor)

$Op_{Rating} := 22.6$ Ton (Operating Rating)

Follow the "Posting Guidelines For County Ratings" Page 2 of 2

For Type M3 truck

Note: SU trucks are associated with the Type M3 truck and are not covered in this example since observation shows that the M3 posting will control.

Check if RF (Rating Factor) < 0.125 NO

Check if 0.125 <= RF <= 1.10 Yes, therefore post at indicated tonnage

Operating Rating = 17.9 Tons - Note: round the calculated posting tonnage down to the nearest even ton.

Therefore Type M3 posting = 16 Tons

For Type M3S2 truck

Check if RF (Rating Factor) < 0.35 NO

Check if 0.35 <= RF <= 1.00 Yes, therefore post as indicated with sign R12-5

Operating Rating = 29.8 Tons - Note: round the calculated posting tonnage down to the nearest even ton.

Therefore Type M3S2 posting = 28 Tons

For Type M3S3 truck

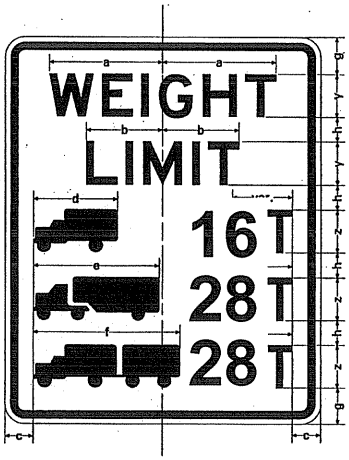
Check if RF (Rating Factor) < 0.35 NO

Check if 0.35 <= RF <= 1.00 Yes, therefore post as indicated with sign R12-5

Operating Rating = 28.2 Tons - Note: round the calculated posting tonnage down to the nearest even ton.

Therefore Type M3S3 posting = 28 Tons

The R12-5 Sign would look like the following



CLASS EXERCISE

FORM RC–CL Revised Mar. 2005	Mn/DOT BRIDGE RATING AND LOAD POSTING REPORT FOR COUNTY AND LOCAL AGENCIES

Bridge Location and Description

Bridge No. 1795

Hwy. No. RYE RD. over/under WHISKY CREEK

Year Built 1966 Year Remodeled — Replaces Br. —

Type TIMBER PANEL County DISTRICT Ref. Pt.

Description 1-SPAN TIMBER NAIL-LAMINATED DECK ON TIMBER ABUTMENTS

Location

Data for Basis of Report (Check all that apply)

- [x] Bridge Inventory File
- [ ] Previous Bridge Rating and Load Posting Report
- [x] Bridge Plans
  - [ ] New
  - [ ] Overlay
  - [ ] Repair/Reconstruction
  - [ ] Other Dead Load Modifications
- [x] Bridge Inspection Report, by UNDESIGNED, date 9/15/06
  - [x] Damaged Component REDUCED WHEEL DISTRIBUTION WIDTH USED DUE TO REDUCED LOAD TRANSFER BY DOWELS AND LACK OF SPREADER BEAM AT MIDDLE OF SPAN.
  - [ ] Deteriorated Component

Type of Analysis:

- [x] Manual
- [ ] Computer*
- [ ] BARS
- [ ] Virtis, V.
- [ ] Other*

*

Method of Rating (Check appropriate box)

- [ ] Load Factor (LF)
- [ ] Allowable Stress (AS)
- [ ] Load & Resistance Factor (LRFR)
- [ ] Load Testing
- [ ] No Rating Computations Performed

Design Load

Design Method

Summary of Rating and Load Posting Analysis

Load Posting Limits — Posting: [x] Required [ ] Not Required			Bridge Rating	
Vehicle Type M3	Semi-Trailer Type M3S2	Truck-Full Trailer Type M3-3	Inventory Rating	Operating Rating
16 Tons	28 Tons	28 Tons	HS	HS

SIGN HERE

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

Signature: Date: 10/16/06

Typed or Printed Name: JIM BEAM LIC. ~~Reg~~ No.

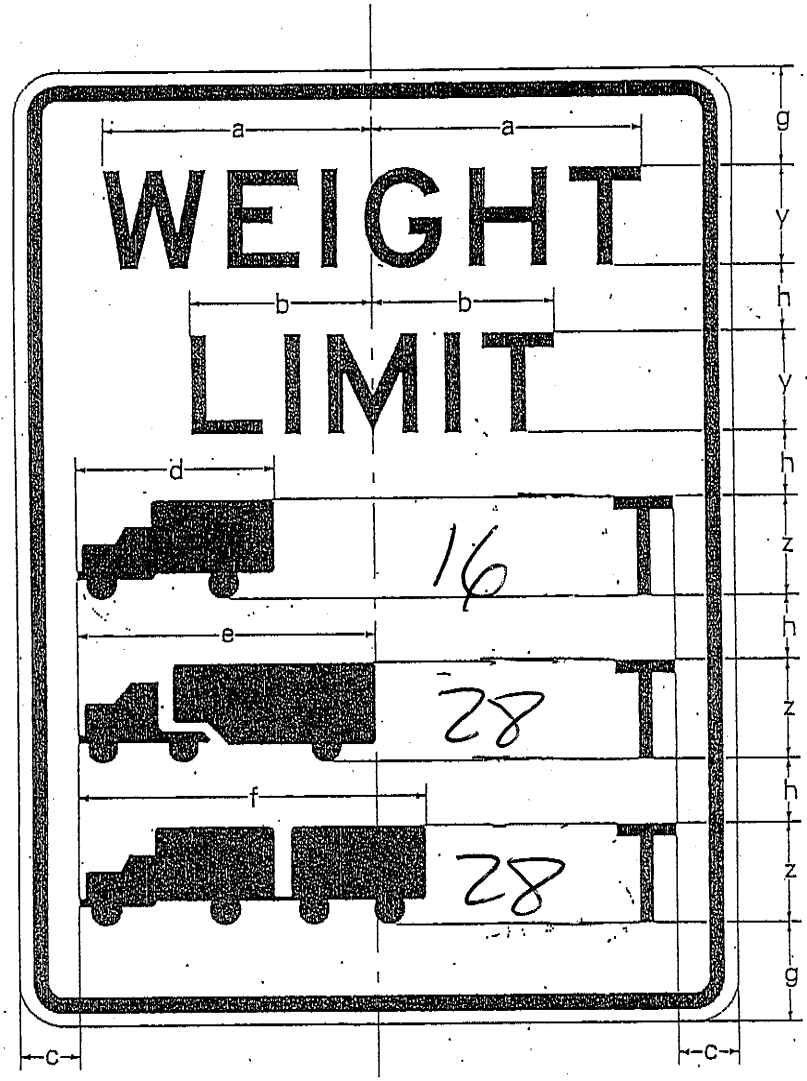
APPROVED 6/11/92

DATE OF REV. 1/1/05

WEIGHT LIMIT

SIGN NUMBER R12-5

SIZE / DIMENSION	24 X 36	30 X 42	36 X 48
RADIUS	1.5	1.88	2.25
MARGIN	.38	.5	.63
BORDER	.63	.75	.88
a	9.5	12.2	13.5
b	6.4	8.2	9.2
c	1.6	1.8	3
d	7	9	10
e	10.5	13.5	15
f	12.2	15.7	17.5
g	3.25	3.75	5
h	3	3	3.25
i			
j			
k			
l			
m			
n			
o			
p			
q			
r			
s			
t			
u			
v			
w			
x			
y	3.5E	4.5E	5E
z	3.5D	4.5D	5D



NOTES: 1. All dimensions are in inches.
2. Color - Black legend and border on white reflectorized background.
3. Appropriate numerals to be specified.

Class Exercise

Workshop Agenda

VIII. Load Rating Example #2
- Simple Span Nail Laminated Timber Deck (without distress)

IX. Class Exercise
- Simple Span Nail Laminated Timber Deck (with distress)

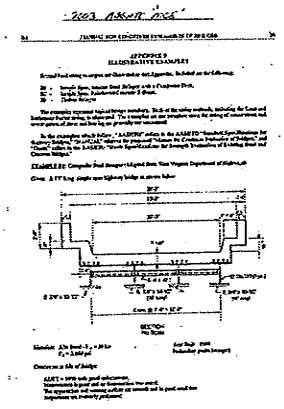
X. Review of worked out examples

XI. Submittal to MN/DOT and Review of Process

XII. Common mistakes and questions

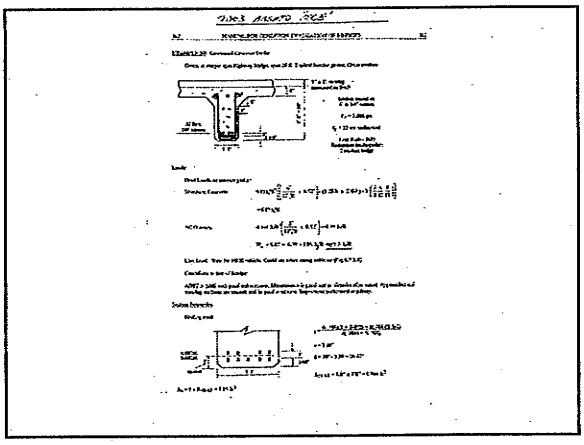
Other Examples

- Simple Span, Interior Steel Stringer with Composite Deck



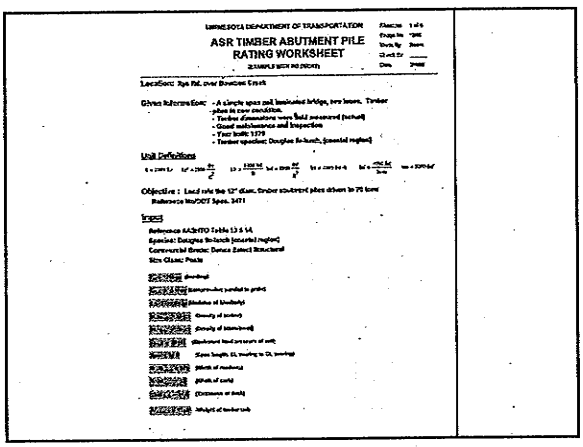
Other Examples

- Simple Span, Reinforced Concrete T-Beam



Other Examples

- ASR, Timber Abutment Pile with No Decay


MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER ABUTMENT PILE RATING WORKSHEET

(EXAMPLE WITH NO DECAY)

Location: Rye Rd. over Bourbon Creek

Given Information: - A simple span rail laminated bridge, two lanes. Timber piles in raw condition.
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built: 1979
- Timber species: Douglas fir-larch, (coastal region)

Unit Definitions

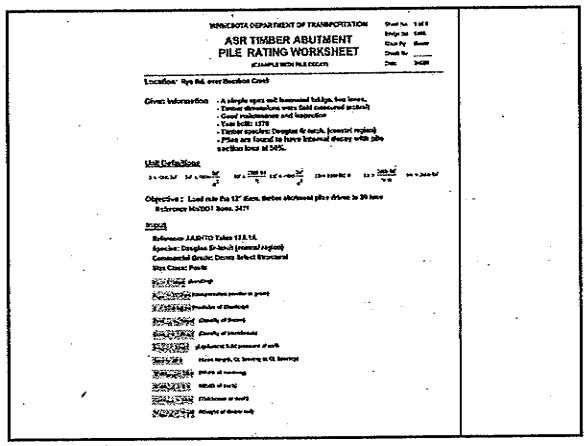
Objective: Load rate the 12" diam. timber abutment piles driven to 70 tons
Reference MnDOT Spec. 3471

Input

Reference AASHTO Table 13.5.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: Dense Select Structural
Size Class: Posts

Other Examples

- ASR, Timber Abutment Pile with Decay


MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER ABUTMENT PILE RATING WORKSHEET

(EXAMPLE WITH PILE DECAY)

Location: Rye Rd. over Bourbon Creek

Given Information: - A simple span rail laminated bridge, two lanes.
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built: 1979
- Timber species: Douglas fir-larch, (coastal region)
- Piles are found to have internal decay with pile section loss of 50%.

Unit Definitions

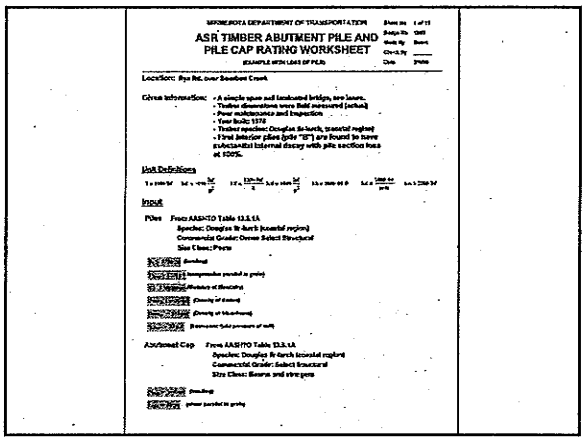
Objective: Load rate the 12" diam. timber abutment piles driven to 20 tons
Reference MnDOT Spec. 3471

Input

Reference AASHTO Table 13.5.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: Dense Select Structural
Size Class: Posts

Other Examples

- ASR, Timber Abutment Cap with Loss of Pile



MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER ABUTMENT PILE AND PILE CAP RATING WORKSHEET

(EXAMPLE WITH LOSS OF PILE)

Location: Bye Rd. over Bowdoin Creek

Given information:
- A simple span nail laminated bridge, see leave.
- Timber dimensions were field measured (actual)
- Poor maintenance and inspection
- Year built: 1978
- Timber species: Douglas fir-larch, (coastal region)
- First interior piles (pile "B") are found to have substantial internal decay with pile section loss at 100%.

Unit Definitions

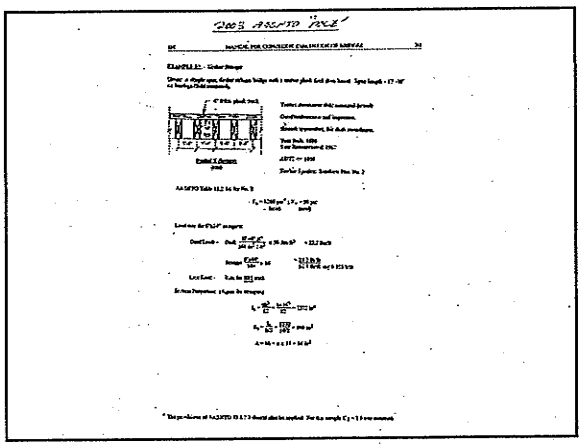
Input

Piles From AASHTO Table 13.3.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: Dense Select Structural
Size Class: Piers

Abutment Cap From AASHTO Table 13.3.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: Select Structural
Size Class: Beams and stringers

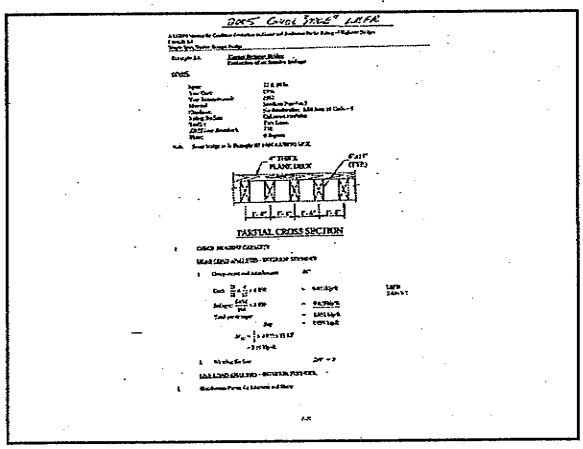
Other Examples

- ASR, Simple Span Timber Stringer



Other Examples

- LRFR, Simple Span Timber Stringer

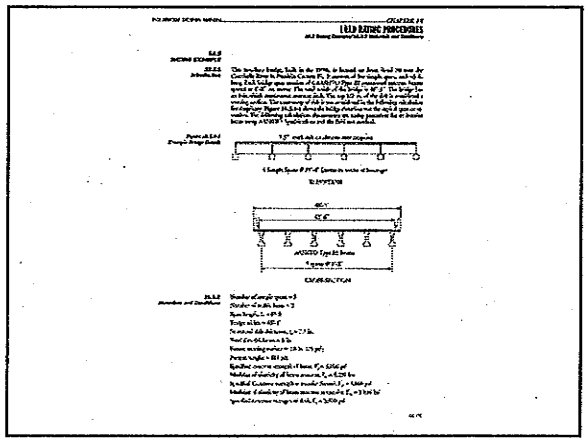


Other Examples

- ASR (Using Old Forms)
  - Timber Beam
  - Timber Longitudinal Nailed Panel
  - Timber Longitudinal Glulam Deck
  - Timber Transverse Plank Deck

Other Examples

- LFR & LRFR, Simple Prestressed Concrete Beams



2003 AASHTO "MCE"

APPENDIX B
ILLUSTRATIVE EXAMPLES

Several load rating examples are illustrated in this Appendix. Included are the following:

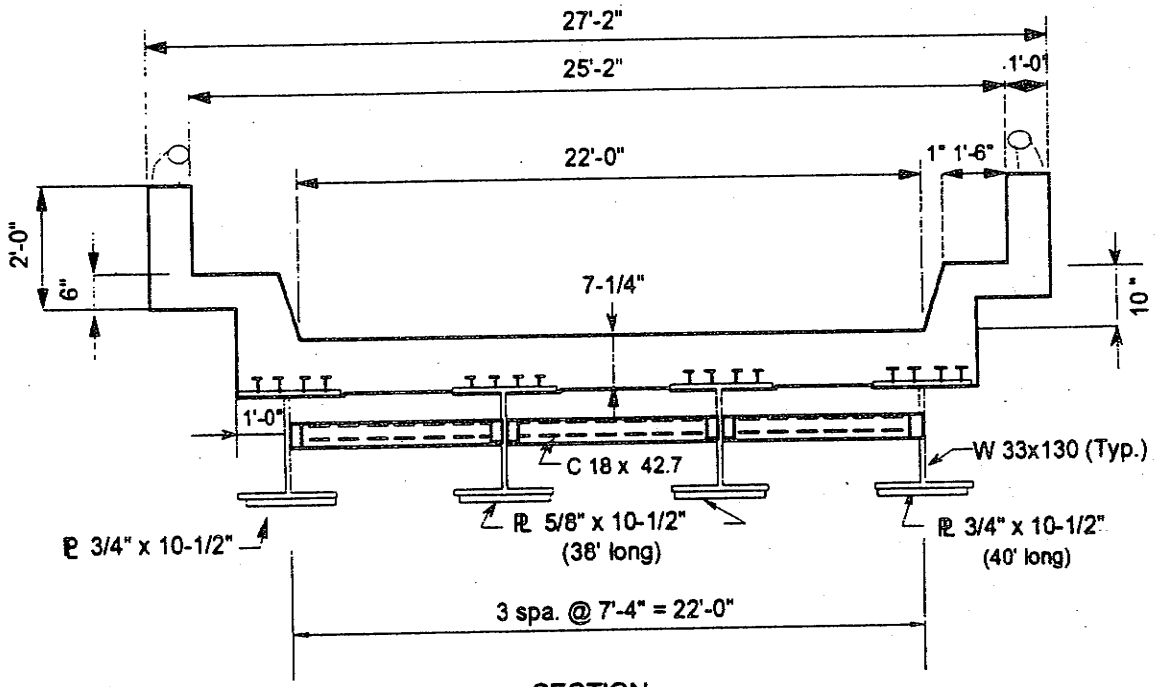
B1 - Simple Span, Interior Steel Stringer with a Composite Deck.
B2 - Simple Span, Reinforced Concrete T-Beam.
B3 - Timber Stringer

The examples represent typical bridge members. Each of the rating methods, including the Load and Resistance Factor rating, is illustrated. The examples are not complete since the rating of connections and investigation of shear and bearing are generally not considered.

In the examples which follow, "AASHTO" refers to the AASHTO "Standard Specifications for Highway Bridges," "MANUAL" refers to the proposed "Manual for Condition Evaluation of Bridges," and "Guide" refers to the AASHTO "Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges."

EXAMPLE B1: Composite Steel Stringer (Adapted from West Virginia Department of Highways)

Given: A 65' long, simple span highway bridge as shown below.



SECTION
No Scale

Materials: A36 Steel - F_y = 36 ksi
f'_c = 3,000 psi

Year Built : 1964
Redundant (multi-Stringer)

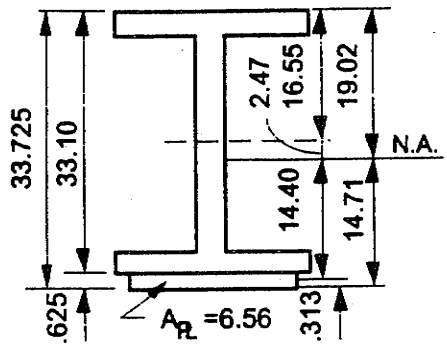
Conditions at Site of Bridge:

ADTT > 1000 with good enforcement.
Maintenance is good and no deterioration was noted.
The approaches and wearing surface are smooth and in good condition.
Inspections are routinely performed.

Rate a typical Interior Stringer

Section Properties: In unshored construction, the steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of n. To account for the effect of creep, superimposed dead load stresses are carried by the composite section using a modular ratio of 3n. (AASHTO 10.38.1). The as-built section properties are used in this analysis.

Noncomposite: W33 x 130 & ℄ 5/8" x 10-1/2"
$t_f = 0.855"$; $b_f = 11.51"$; $t_w = 0.58"$
$A = 38.26\ in^2$



$$\bar{y} = \frac{(17.175)^{W}(38.26) + (.313)^{PL}(6.56)}{38.26 + 6.56}$$

$$\bar{y} = 14.71"$$

$$I_x = {}^{W}6699 + {}^{W}38.26(2.47)^2 + {}^{PL}6.56(14.40)^2$$
$$= 8293\ in^4$$

$$S_t = \frac{8293}{19.02} = 436.0\ in^3 = S_t^{DL}$$

$$S_b = \frac{8293}{14.71} = 563.7\ in^3 = S_b^{DL}$$

Composite Section Properties:

Effective Flange Width: (AASHTO 10.38.3.1)

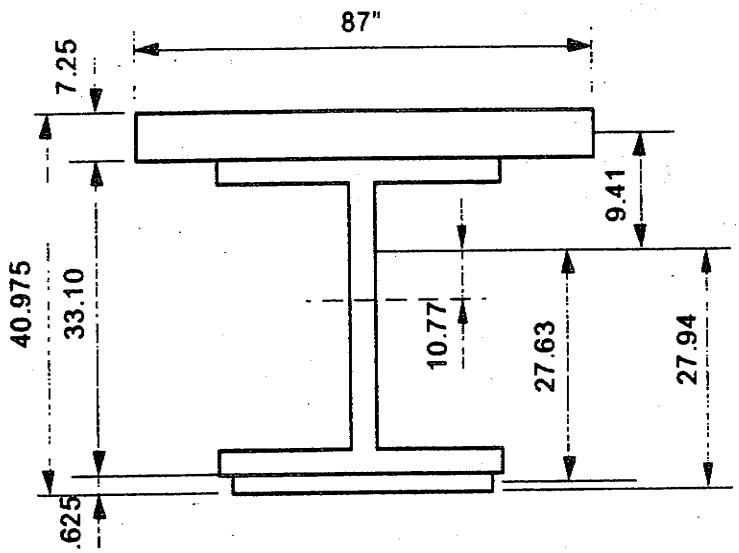
1/4(65)(12) = 195"
(7.33)(12) = 88"
(7.25)(12) = 87" ⇐ Controls

Modular Ratio (n): (MANUAL 6.6.2.4)
for $f'_c = 3{,}000$ psi - n = 10

Composite Section Properties cont.:

Typical Interior Stringer:

Composite n = n: W33 x 130, ℄ 5/8" x 10-1/2" & Conc. 7-1/4" x 87"



$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{℄}{(.313)(6.56)} + \overset{\text{Conc.}}{(87 \times 7.25 \div 10)(37.35)}}{38.26 + 6.56 + (87 \times 7.25) \div 10}$$

$$\bar{y} = 27.94"$$

$$I_X = \overset{W}{6699} + \overset{W}{(38.26)(10.77)^2} + \overset{℄}{(6.56)(27.63)^2} + \overset{\text{Conc.}}{\frac{(87 \div 10)(7.25)^3}{12}} + \overset{\text{Conc.}}{(87 \times 7.25) \div 10\,(9.41)^2}$$

$$= 22007 \text{ in}^4$$

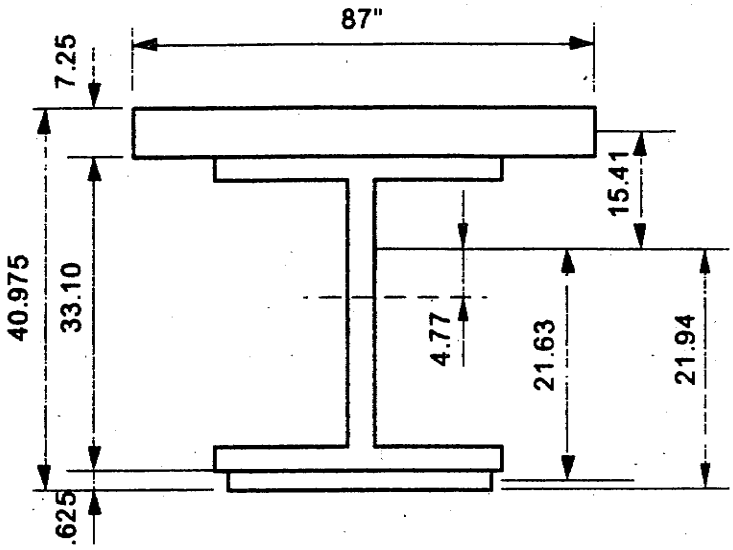
$S_t = \frac{22007}{5.79} = 3801 \text{ in}^3$ Section modulus at top of steel

$S_b = \frac{22007}{27.94} = 787.7 \text{ in}^3 = S_b^L$

use with Live Load

Composite Section Properties cont.:

Composite n = 3n: W33 x 130, PL 5/8" x 10-1/2" & Conc. 7-1/4" x 87"



$$\bar{y} = \frac{(17.175)(38.26) + (.313)(6.56) + (87 \times 7.25 + 30)(37.35)}{38.26 + 6.56 + (87 \times 7.25) + 30}$$

$$\bar{y} = 21.94"$$

$$I_x = 6699 + (38.26)(4.77)^2 + (6.56)(21.63)^2 + \frac{(87 + 30)(7.25)^3}{12} + \left(\frac{87 \times 7.25}{30}\right)(15.41)^2$$

$$I_x = 15{,}725 \text{ in}^4$$

$$S_t = \frac{15725}{11.79} = 1333.8 \text{ in}^3 \text{ (Section modulus at top of steel)}$$

$$S_b = \frac{15725}{21.94} = 716.7 \text{ in}^3 = S_b^{SDL}$$

use with Superimposed Dead Load (SDL)

Loads:

Dead Loads (includes an allowance of 6% of steel weight for connections):

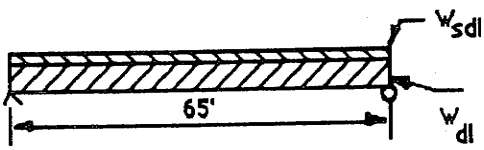
Deck (7.33) $\left(\frac{7.25}{12}\right)$(150 pcf)	=	664.3 lbs/ft
Stringer (130)(1.06)	=	137.8 lbs/ft
Cover ℄ (.625)(10.5)(490/144)(1.06)(38) ÷ 65	=	13.8 lbs/ft
Diaphragms (3)(42.7)(7.33)(1.06) ÷ 65	=	15.4 lbs/ft
Total per stringer		831.3 lbs/ft

Superimposed Dead Loads: (see AASHTO 3.23.2.3.1.1)

Curb (1)$\left(\frac{10}{12}\right)$(150 pcf) ÷ 2	=	62.5 lbs/ft
Parapet $\left[\left(\frac{6 \times 19}{144}\right) + \left(\frac{18 \times 12}{144}\right)\right]$ (150 pcf) ÷ 2	=	171.9 lbs/ft
Railing (assume 20 plf) ÷ 2	=	10.0 lbs/ft
Wearing Surface	=	0.0
Total per stringer		244.4 lbs/ft

Live Load: Rate for HS20

Moments:



$$M_{DL} = \frac{w_{dl} L^2}{8} = \frac{.831(65)^2}{8} = 439 \text{ ft-k}$$

$$M_{SDL} = \frac{w_{sdl} L^2}{8} = \frac{.244(65)^2}{8} = 129 \text{ ft-k}$$

M_L - From MANUAL, Appendix A3, page 74(1)

Span	M_L	
60	403.3	
		⇐ 65'
70	492.8	

$$M_L = \frac{403.3 + 492.8}{2}$$

M_L = 448 ft-k

(without Impact, without Dist.)

(1) Note the moments given in MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUALvalue.

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(Consider Maximum Moment Section only for this example - See general notes.)

Impact - MANUAL 6.7.4. Use standard AASHTO

AASHTO 3.8.3.1

$$I = \frac{50}{L+125} \leq 0.3$$

$$I = \frac{50}{65+125} = 0.26$$

Distribution - MANUAL 6.7.3 indicates that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1

$$DF = \frac{S_s}{5.5} = \frac{7.33^{FT}}{5.5} = 1.33$$

Thus:

$$M_{L+I} = M_L(1+I) * DF = 448(1+0.26)(1.33)$$

$$M_{L+I} = 751 \text{ ft-k}$$

Inventory Level: MANUAL 6.6.2.1, Table 6.6.2.1-1 (bottom steel in tension controls)

For steel with $F_y = 36$ ksi $\rightarrow f_I = 0.55\, f_y$

Thus:

$$f_I = 0.55(36) = 20 \text{ ksi}$$

The Resisting Capacity $(M_{RI}) = f_I\, S_x^L$

$$M_{RI} = 20 \text{ ksi } (787.7 \text{ in.})^3 = 15754 \text{ in-k} = 1313 \text{ ft-k}$$

Then:

$$RF_I = \frac{M_{RI} - M_{DL}\dfrac{S_b^L}{S_b^{DL}} - M_{SDL}\dfrac{S_b^L}{S_b^{SDL}}}{M_{L+I}}$$

$$= \frac{1313 - 439\,\dfrac{787.7}{563.7} - 129\,\dfrac{787.7}{716.7}}{751} = \frac{557.8}{751}$$

$$= \underline{0.74} \text{ or } 0.74 \times 36 \text{ tons} = \underline{26.7 \text{ tons}}$$

Alternatively, in terms of stress:

$$RF_I = \frac{f_s - \frac{M_{DL}}{S_b^{DL}} - \frac{M_{SDL}}{S_b^{SDL}}}{\frac{M_{L+I}}{S_b^{L+I}}}$$

$$= \frac{20\ \text{ksi} - \frac{439^{\text{ft-k}} \times 12^{\text{in/ft}}}{563.7^{\text{in}^3}} - \frac{129^{\text{ft-k}} \times 12^{\text{in/ft}}}{716.7^{\text{in}^3}}}{\frac{751^{\text{ft-k}} \times 12^{\text{in/ft}}}{787.7^{\text{in}^3}}}$$

$$= \frac{20 - 9.345 - 2.160}{11.441}$$

$$= \frac{8.495}{11.441} = 0.74 \text{ as above}$$

Operating Level: MANUAL 6.6.2.1, Table 6.6.2.1-2

For steel with $F_y = 36$ ksi $\rightarrow f_o = 0.75\, f_y$

Thus:

$$f_o = 0.75(36) = 27 \text{ ksi}$$

and

$$M_{RO} = 27(787.7) = 21268 \text{ in-k} = 1772 \text{ ft-k}$$

and

$$R_{FO} = \frac{1772 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{1016.8}{751}$$

$$RF_O = \underline{\underline{1.35}} \text{ or } 1.35 \times 36 \text{ tons} = \underline{\underline{48.7 \text{ tons}}}$$

Load Factor Rating: MANUAL 6.4.2, 6.5.3 & 6.6.3

(Consider maximum moment section only for this example - see General Notes.)

Impact - MANUAL 6.7.4 – use standard AASHTO

From AS Rating I = 0.26

Distribution – MANUAL 6.7.3 – use standard AASHTO

From AS Rating DF = 1.33

$$M_{L+I} = M_L (1 + I)\, DF = 448(1 + 0.26)(1.33)$$

$$= 751 \text{ ft-k (as for AS Rating)}$$

Capacity of Section: (M_R) – MANUAL 6.6.3.1

For braced, compact, composite sections:

$$M_R = M_u \text{ (AASHTO 10.50.1.1)}$$

where M_u is found in accordance with applicable load factor provisions of AASHTO.

Check assumptions:

(1) Section is fully braced along top flange by composite deck (for Live Load & SDL)

(2) To check if section is compact, need to apply provisions of AASHTO 10.50.1.1.2. These checks follow.

Eqn: 10-123: $C_{CONC} = 0.85 f_c b_{eff} t_s = 0.85(3\,ksi)(87\,in)(7.25\,in) = 1608\,k^*$

Eqn: 10-124: $C_{STL} = A_s f_y = (38.26\,in^2 + 6.56\,in^2)(36\,ksi) = 1613.5\,k$

$C_{CONC} < C_{STL} \therefore C_{CONC} = 1608$ controls (10.50.1.1.1(a))

Capacity – per AASHTO 10.50.1.1.1(c)

Eqn: 10-126: $C' = \frac{\Sigma (AF_y) - C}{2} = \frac{1613.5 - 1608}{2} = 2.75\,k$

and applying AASHTO 10.50.1.1.1(d)

$(AF_y)_{TF} = (11.51 \times .855)(36) = 354\,k >>> 2.75k \therefore$ NA in top flange

Eqn: 10-127: $\bar{y} = \frac{C'}{(AF_y)_{TF}} t_{TF} = \frac{2.75}{354}(.855) = 0.007$ in neglect. Say NA at top of steel.

Since the PNA is at the top of the top flange, the depth of the web in compression at the plastic moment, D_{cp} is equal to zero. Hence, the web slenderness requirement given by Equation (10-129) in Article 10.50.1.1.2 is automatically satisfied.

* Neglects reinforcement in slab.

Check the ductility requirement given by Equation (10-129a) in Article 10.50.1.1.2:

Eqn: 10-129a $\left(\frac{D_p}{D'}\right) \le 5$

$$D' = \beta \frac{(d + t_s + t_h)}{7.5} \qquad \beta = 0.9 \text{ for } F_y = 36{,}000 \text{ psi}$$

$$D' = 0.9 \frac{(33.725 + 7.25 + 0.0)}{7.5} = 4.92$$

$$D_p = 7.25 \text{ in}$$

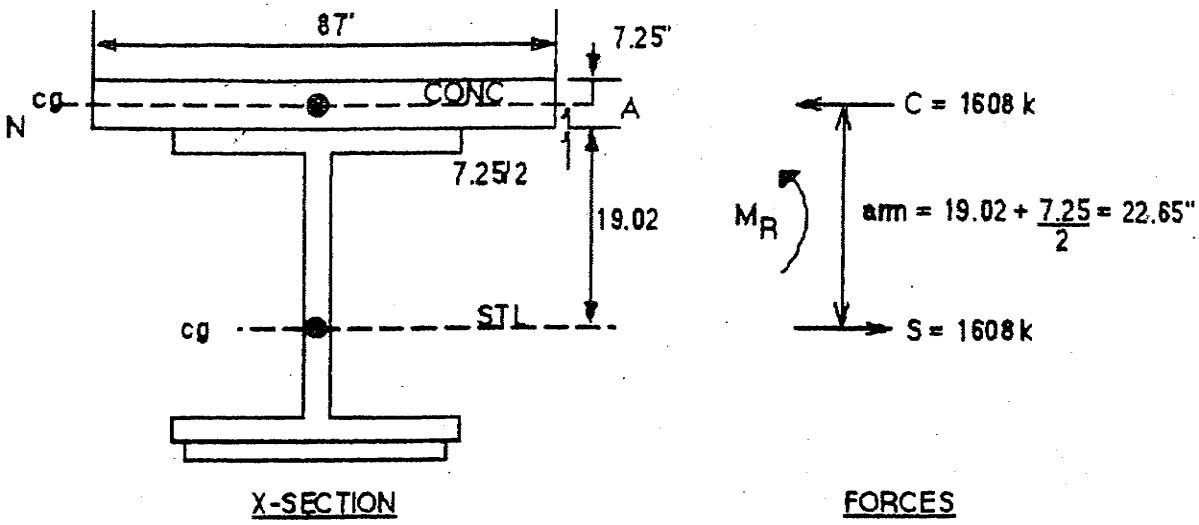
$$\left(\frac{D_p}{D'}\right) = \frac{7.25}{4.92} = 1.47 < 5 \quad \text{ok}$$

Since the top flange is braced by the hardened concrete deck, local and lateral buckling requirements need not be checked. The capacity of composite beams in simple spans satisfying the preceding web slenderness and ductility requirements is given by Equation (10-129c) in Article 10.50.1.1.2 when D_p exceeds D':

$$C = M_R = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right)$$

$$M_y = F_y S = (36)\frac{787.7}{12} = 2363 \text{ ft-k}$$

Compute the plastic moment capacity M_p.



M_p = C*arm = S*arm = 1608(22.65) = 36421 in-k = 3035 ft-k

$$M_R = \frac{5(3035) - 0.85(2363)}{4} + \frac{0.85(2363) - 3035}{4}(1.47)$$

$$= 2914 \text{ ft-k}$$

Inventory Level: MANUAL 6.5.1 and 6.6.3

$$RF_I^{L+I} = \frac{M_R - A_1 M_D}{A_2 M_{L+I}} \quad \text{(MANUAL Eqn : 6-1a)}$$

where: (MANUAL 6.5.3)

$A_1 = 1.3$
$A_2 = 2.17$

Thus:
$$RF_I^{LF} = \frac{(2914) - 1.3(439 + 129)}{2.17(751)}$$

$$RF_I^{LF} = \underline{1.33} \text{ or } 1.33 \text{ x } 36 \text{ tons} = \underline{47.9 \text{ tons}}$$

Operating Level: MANUAL 6.5.3

Only change is $A_2 = 1.3$

Thus:
$$RF_O^{LF} = \frac{2.17}{1.3} RF_I^{LF} = \frac{2.17}{1.3}(1.33)$$

$$RF_O^{LF} = \underline{2.22} \text{ or } 2.22 \text{ x } 36 \text{ tons} = \underline{79.9 \text{ tons}}$$

Check Serviceability Criteria – AASHTO 10.57.2

At Inventory Level (bottom steel in tension controls)

$$f_D + 1.67 (RF_I^{LF}) f_{L+I} \leq \text{Serv. Strength} = 0.95 F_y$$

$$RF_I^{LF} = \frac{0.95 F_y - f_D - f_{SDL}}{1.67 f_{L+I}}$$

$$= \frac{0.95 (36 \text{ ksi}) - \dfrac{439(12)}{563.7} - \dfrac{129(12)}{716.7}}{1.67 \dfrac{751(12)}{787.7}}$$

$$RF_I^{LF} = \underline{1.19} \text{ or } 1.19 \text{ x } 36 \text{ tons} = \underline{42.8 \text{ tons}}$$

Check the web compressive stress:

Eqn: 10-173

$$C = F_{cr} = \frac{26,200,000 \, \alpha \, k}{\left(\dfrac{D}{t_w}\right)^2} \leq F_{yw}$$

where:
$$k = 9(D/D_c)^2$$
$$\alpha = 1.3$$

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of Article 10.50(b), an iterative procedure may be necessary to determine the rating factor:

Compute the compressive stresses at the top of the web:

$$f_D = \frac{439(12)(18.165)}{8293} = 11.5 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(10.935)}{15,725} = 1.1 \text{ ksi}$$

$$f_{L+I} = \frac{(1.67)(751)(12)(4.935)}{22,007} = 3.4 \text{ ksi}$$

$$\sum = 16.0 \text{ ksi}$$

Compute the tensile stresses at the bottom of the web:

$$f_D = \frac{439(12)(13.23)}{8293} = 8.4 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(20.46)}{15,725} = 2.0 \text{ ksi}$$

$$f_{L+I} = \frac{(1.67)(751)(12)(26.46)}{22,007} = 18.1 \text{ ksi}$$

$$\sum = 28.5 \text{ ksi}$$

$$D_c = 31.39\left(\frac{16.0}{16.0+28.5}\right) = 11.29''$$

$$k = 9(D/D_c)^2 = 9(31.39/11.29)^2 = 69.9$$

$$C = F_{cr} = \frac{26,200,000(1.3)(69.6)}{\left(\frac{31.39}{0.58}\right)^2(1000)} = 809.3 \text{ ksi} > F_{yw}$$

$$\therefore F_{cr} = F_{yw} = 36 \text{ ksi}$$

$$RF_I^{LF} = \frac{36-11.5-1.1}{3.4} = \underline{\underline{6.9}} \text{ or } (6.9)(36 \text{ tons}) = \underline{\underline{248.4 \text{ tons}}}$$

Since the computed rating factor would cause the total stresses in the tension flange to far exceed F_y (causing the neutral axis to be higher on the web), further iterations are not necessary in this case. The web compressive stress does not govern the serviceability rating.

At Operating Level:

$$f_D + RF_O^{LF}(f_{L+I}) \leq \text{Serv. Strength}$$

Thus: $$R_O^{LF} = RF_I^{LF} \times 1.67 = 1.19 \times 1.67$$

$$R_O^{LF} = \underline{1.98} \text{ or } 1.98 \times 36 \text{ tons} = \underline{71.3 \text{ tons}}$$

Load Factor Summary

	RE	TONS	CONTROLLED
Inventory	1.19	42.8	Art. 10.57.2
Operating	1.98	71.3	Art. 10.57.2

Load and Resistance Factor Rating: (See *AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*)

(Again consider maximum moment section only for this example – see General Notes.)

Impact - Guide 3.3.2.3 may vary based on condition of wearing surface. But for comparison purposes use same I as for AS method:

$$I = 0.26$$

Distribution - Guide 3.3.3 use standard AASHTO

$$DF = 1.33$$

Live Load – Guide 3.3.2.2 use HS20 to be consistent with other rating methods. Normally would use rating vehicles (Guide Fig. 2) or lane loading (Guide Fig 3).

Thus: $$M_{L+I} = M_R\ (1+I) \times DF = 448 \times (1+0.26)(1.33)$$

$$= 751 \text{ ft-k}$$

Capacity of Section – Guide 3.3.2.4

M_R is based on AASHTO 10.50 as for Load Factor

Thus: $$M_R = 2914 \text{ ft-k (Point Page 98.1)}$$

$$RF^{LRF} = \frac{\phi M_R - \gamma_D M_D}{\gamma_L M_{L+I}} \quad \text{(Guide Eqn. 2)}$$

where:

ϕ	=	0.95	(Guide 3.3.4.2, Table 3(b))
γ_D	=	1.2	(Guide, Table 2)
γ_L	=	1.45	(Guide, Table 2)

Load and Resistance Factor Rating (*continued*):

then:

$$RF^{LRF} = \frac{0.95(2914) - 1.2(439 + 129)}{1.45(751)}$$

$$RF^{LRF} = \underline{1.91} \text{ or } 1.91 \times 36 \text{ tons} = \underline{68.6 \text{ tons}}$$

SUMMARY OF RESULTS

	RF	HS20 Truck	H20[(1)] Truck
Allowable Stress:			
Inventory	0.74	26.7	21
Operating	1.35	48.7	38.3
Load Factor:			
Inventory	1.19	42.8	33.8
Operating	1.98	71.3	56.2
Load and Resistance Factor	1.91	68.8	54.2

(1) $H_{TR} = RF \times \frac{M_L^{HS20}}{M_L^{H20}} \times 20^T$ $\qquad M_L^{HS20} = 448$ (page 87)

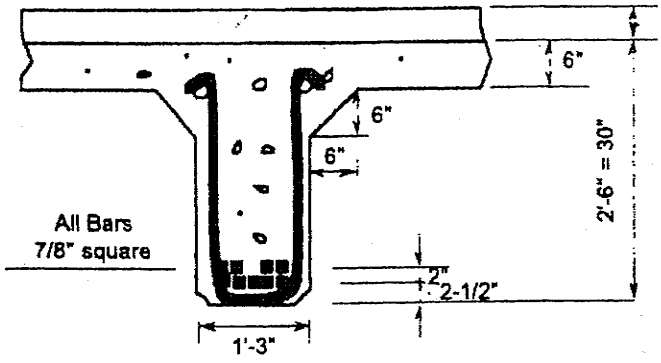
$= RF\frac{448}{316} \times 20^T$ $\qquad M_L^{H20} = \frac{20}{15} M_L^{H15} = \frac{20}{15} \frac{(265.1 + 209.2)}{2} = 316$ ft-k

$H_{TR} = RF \times (1.42) \times 20^T$

2003 AASHTO "MCE"

EXAMPLE B2: Reinforced Concrete Girder

Given: A simple span highway bridge, span 26 ft. Typical interior girder. Cross section:



5" A.C. overlay
(measured in field)

Girders spaced on
6'-6-1/4" centers

$f'_c = 3{,}000$ psi

$f_y = 33$ ksi (unknown)

Year Built - 1925
Redundant (multi-girder)
Two-lane bridge

Loads:

Dead Loads on interior girder:

Structural Concrete:

$$0.15\,\text{k/ft}^3\left[\left(\frac{6''}{12''/\text{ft}}\ \text{x}\ 6.52'\right)+(1.25\,\text{ft}\ \text{x}\ 2.0\,\text{ft})+2\left(\frac{1}{2}\frac{6}{12}\frac{6}{12}\right)\right]$$

$$= 0.87\,\text{k/ft}$$

AC Overlay:

$$0.144\ \text{k/ft}^3\left(\frac{5''}{12''/\text{ft}}\ \text{x}\ 6.52'\right) = 0.39\ \text{k/ft}$$

$$W_{dL} = 0.87 + 0.39 = 1.26\ \text{k/ft}\ \ \underline{\text{say } 1.3\ \text{k/ft}}$$

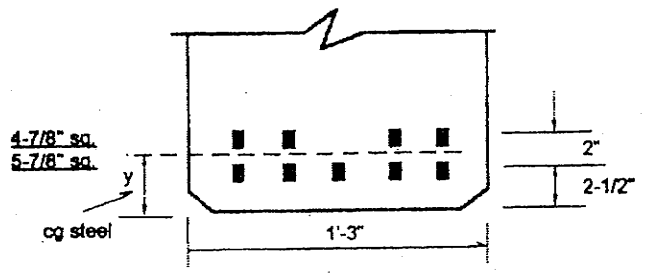
Live Load - Rate for HS20 vehicle. Could use other rating vehicles (Fig.6.7.2.4).

Conditions at Site of Bridge:

ADTT > 1000 with good enforcement. Maintenance is good and no deterioration noted. Approaches and wearing surfaces are smooth and in good condition. Inspections performed regularly.

Section Properties

Find cg steel



$$y = \frac{4(.766)(2 + 2\text{-}1/2) + 5(.766)(2\text{-}1/2)}{4(.766) + 5(.766)}$$

$y = 3.39"$

$d = 30" - 3.39 = 26.61"$

$A_{1BAR} = 7/8" \text{ x } 7/8" = 0.766\ \text{in}^2$

$A_s = 9 \text{ x } A_{1BAR} = 6.89\ \text{in}^2$

Effective Slab Width (for T-Girder)

AASHTO 8.10.1.1

$$1/4\ L = \frac{26\ ft \times 12\ in\text{-}ft}{4} = 78"$$

or

$$CC\ SPCG = 6'— 6\text{-}1/4" = 78.25"$$

or

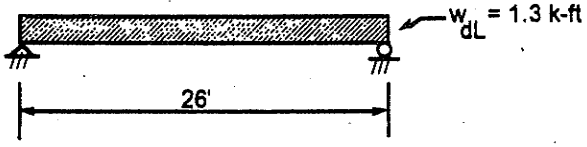
$$12\ t_s = 12 \times 6" = 72" \Leftarrow \text{Controls}$$

$$\rho_{act} = \frac{A_s}{b_{eff}\ d} = \frac{6.89}{72" \times 26.61} = 0.0036$$

(if compression within flange)

Midspan Moments:

Live Load - HS20



$$M_d = \frac{w_{dL}\ L^2}{8} = \frac{1.3\ k/ft \times 26^2 ft^2}{8} = 109.9\ k\text{-}ft$$

For HS20 - From MANUAL, Appendix A3, page 74(1). Using Table, select from column "Without Impact"

M_L = 111.1 k-ft (without impact and without distribution)

(1) Note the moments given in the MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUAL values.

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(For this example we consider only the maximum moment section - see General Notes)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.2.1

$$I = \frac{50}{L + 125} \leq 0.30$$

$$I = \frac{50}{26 + 125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1

$$DF = \frac{S_G}{6.0} \quad \text{Concrete T-Beam}$$

$$DF = \frac{6'— 6\text{-}1/4"}{6.0} = \frac{6.52'}{6} = 1.087$$

Thus:

$$M_{L+I} = M_L\,(1+I)\,(DF) = 111.1(1 + .30)(1.087)$$

$$= 157 \text{ ft-k}$$

Inventory Level: MANUAL 6.5.2 & 6.6.2.4 - The inventory unit stresses are determined in accordance with AASHTO "Service Load Design Method" Article 8.15 or taken from MANUAL 6.6.2.4.

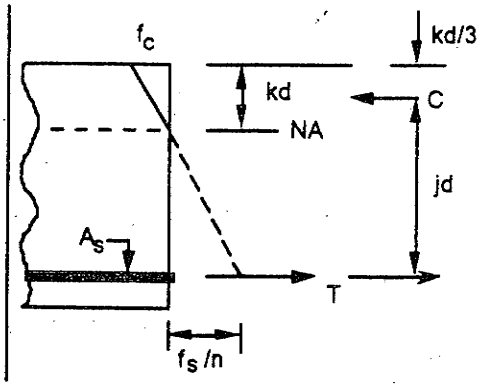
Thus Inventory allowable stresses, AASHTO 8.15.2.1.1

$$f_c^I = 0.4\, f'_c = 0.4\,(3000 \text{ psi}) = 1200 \text{ psi} = 1.2 \text{ ksi}$$

For Reinforcing Steel, MANUAL 6.6.2.3 controls

$$f_s^I = 18000 \text{ psi} = 18 \text{ ksi (unknown steel prior to 1954)}$$

Capacity (Traditional Approach):



Stress & Force Diagram
(nts)

The actual steel and concrete stresses are not known and must be found. Since this is a T-beam, assume neutral axis (na) is within slab. Thus, rectangular beam formulas apply. Check this assumption later.

Position of Neutral Axis:

$$k = \sqrt{2\ \rho n + (\rho n)^2} - \rho n$$

where: $\rho = \frac{As}{bd} = \frac{6.89\ in^2}{(72\ in)(26.61\ in)}$

$$\rho = 0.0036$$

$$k = \sqrt{2(.0036)(10) + ((.0036)(10))^2} - (.0036)(10)$$

$$n = \frac{E_s}{E_c}$$

$$n = 10 \text{ (from Article 6.6.2.4)}$$

$$k = 0.235$$

$$j = 1 - \frac{k}{3} = 1 - \frac{.235}{3} = 0.922$$

Then

Capacity if concrete allowable stress controls—

$$M_c = 1/2\ f_c\ jk\ bd^2$$

$$= 1/2\ (1.2\ ksi)(0.922)(0.235)(72\ in)(26.61\ in)^2$$

$$= 6622.8\ in\text{-}k = 552\ ft\text{-}k$$

Capacity if steel reinforcement allowable stress controls—

$$M_s = A_s\ f_s\ j\ d$$

$$M_s = (6.89\ in^2)(18\ ksi)(0.922)(26.61\ in)$$

$$M_s = 3042.8\ in\text{-}k = 253\ ft\text{-}k \Leftarrow \text{Controls since } M_s < M_c$$

Check neutral axis assumption:

$k_d = (0.235)(26.61\text{ in}) = 6.25" > 6"$ the slab thickness $\therefore$ NA is below bottom of slab and slightly into web. This could be ignored in this case. However for the sake of completeness, capacity will be figured below based on the NA below the slab and ignoring the compression in the stem concrete.

$$kd = \frac{2nd\,A_s + bt^2}{2n\,A_s + 2bt}$$

$$kd = \frac{2(10)(26.61\text{ in})(6.89\text{ in}) + (72\text{ in})(6\text{ in}^2)}{2(10)(6.89\text{ in}) + 2(72\text{ in})(6\text{ in})} = \frac{6258.9}{1001.8}$$

$$kd = 6.25\text{ in} \rightarrow k = \frac{kd}{d} = \frac{6.25\text{ in}}{26.61\text{ in}} = 0.235$$

$$Z = \left(\frac{3kd - 2t}{2kd - t}\right)\frac{t}{3}$$

$$Z = \left(\frac{3(6.25\text{ in}) - 2(6\text{ in})}{2(6.25\text{ in}) - (6\text{ in})}\right)\frac{6\text{ in}}{3} = \frac{6.75\text{ in}}{6.5\text{ in}}(2\text{ in})$$

$$Z = 2.077\text{ in.}$$

$$j\,d = d - Z$$

$$j\,d = 26.61\text{ in} - 2.077\text{ in} = 24.53\text{ in}$$

$$M_s = A_s\,f_s\,j\,d$$

$$M_s = (6.89\text{ in}^2)(18\text{ ksi})(24.53\text{ in}) = 3042.2\text{ in-k}$$

$$M_s = 253\text{ ft-k} \quad \text{as before}$$

(Note concrete was not checked since capacity of section is limited by steel allowable stress.)

$$RF_I^A = \frac{M_{RI} - M_D}{M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

$$RF_I^A = \frac{253\text{ ft-k} - 109.9\text{ ft-k}}{157\text{ ft-k}} = 0.91$$

Operating Level: MANUAL 6.5.2 & 6.6.2.4

The operating allowable stresses, MANUAL 6.6.2.4 for $f'_c = 3{,}000$ psi:

$$f_c^o = 1900\text{ psi} = 1.9\text{ ksi}$$

For Reinforcing Steel, MANUAL 6.6.2.3 controls:

$$f_s^o = 25{,}000 \text{ psi} = 25 \text{ ksi (unknown steel, prior to 1954)}$$

The basic relationships defined previously apply:

Since ρ and n do not change, the neutral axis, k, j and Z terms do not change.

Thus:

$$M_s = A_s f_s j d$$

$$= (6.89 \text{ in}^2)(25 \text{ ksi})(24.53 \text{ in})$$

$$= 4225.3 \text{ in-k} = 352 \text{ ft-k}$$

and checking concrete stress to ensure that concrete does not control

$$f_c = \frac{f_s}{n}\left(\frac{k}{1 - k}\right)$$

$$f_c = \left(\frac{25 \text{ ksi}}{10}\right)\left(\frac{0.235}{1 - .235}\right) = 0.77 \text{ ksi} << 1.9 \text{ ksi allowable}$$

Therefore, capacity of section is controlled by allowable steel stress.

$M_{R_O} = 352$ ft-k

$$RF_O^A = \frac{M_{R_O} - M_D}{M_{L+I}} = \frac{352 \text{ ft-k} - 109.9 \text{ ft-k}}{157 \text{ ft-k}}$$

$RF_O^A = 1.54$

Load Capacity Based on Allowable Stress

Inventory: $0.91 \times 36^T = 32.8^T$ HS

Operating: $1.54 \times 36^T = 55.4^T$ HS

To transform "HS" rating to "H" rating multiply HS rating factor by ratio of "HS" moment to "H" moment:

For 26' span: $M_L^{HS20} = 111.1$ ft-k (see Sheet 97)

and using MANUAL Appendix A3, pg. 74 → $M_L^{H15} = 78$ ft-k

Then $$M_L^{H20} = \frac{20^T}{15^T} \times 78 \text{ ft-k} = 104 \text{ ft-k}$$

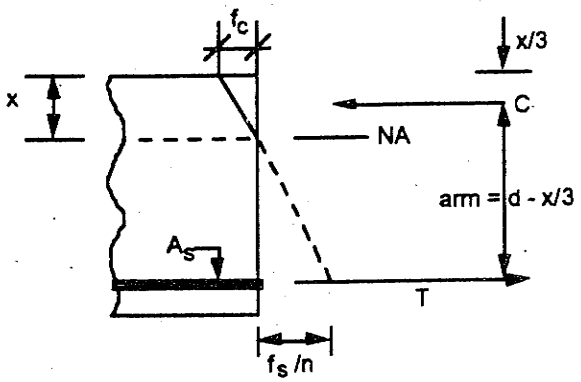
and $$\text{Ratio} = \frac{M_L^{HS20}}{M_L^{H20}} = \frac{111.1}{104} = 1.068$$

Thus for H20 Truck:

Inventory: $0.91 \times 1.068 \times 20^T = 19.4^T$ H

Operating: $1.54 \times 1.068 \times 20^T = 32.9^T$ H

Capacity (Alternate Approach):



Stress & Force Diagram
(nts)

Since the location of the neutral axis (NA) and the corresponding stresses in the steel and concrete are not known, these must be determined consistent with the principles of equilibrium of the cross section.

(1) From the stresses on the cross section:

$$\frac{f_c}{x} = \frac{f_s/n}{d - x} \rightarrow f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) \qquad \text{Eqn. 1}$$

(2) Assume the steel allowable stress controls the capacity of the section. This will be checked later. Then

$$T = A_s f_s = (6.89 \text{ in}^2)(18 \text{ ksi}) = 124 \text{ k}$$

and

$$C = 1/2 f_c b x$$

but

$$C = T$$

thus,

$$1/2 f_c b x = A_s f_s$$

$$x = \frac{A_s f_s}{1/2 f_c b} \qquad \text{Eqn. 2}$$

Solve equations 1 and 2 to find location of neutral axis. This may be done by trail and error as follows.

Assume f_s = 18 ksi, i.e. steel allowable stress controls.

Try x = 6.0 in. Then by Eqn. 1:

$$f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) = \frac{18 \text{ ksi}}{10}\left(\frac{6.0 \text{ in}}{26.61 \text{ in} - 6.0 \text{ in}}\right) = 0.524 \text{ ksi} < 1.2 \text{ ksi} \quad \text{allowable OK}$$

and by Eqn. 2:

$$x = \frac{A_s f_s}{1/2\, f_c\, b} = \frac{(6.89 \text{ in}^2)(18 \text{ ksi})}{1/2\,(.524 \text{ ksi})(72 \text{ in})} = 6.57" > 6.0 \quad \text{assumed. Try again}$$

Try x = 6.25 in.

$$f_c = \frac{18}{10}\left(\frac{6.25}{26.61 - 6.25}\right) = 0.552 < 1.2 \text{ ksi} \quad \text{allowable OK}$$

and

$$x = \frac{(6.89)(18)}{1/2\,(.552)(72)} = 6.24 \doteq 6.25 \quad \text{assumed OK}$$

(3) Since x = 6.24 > t = 6.0, NA is below bottom of slab and slightly into web. If web concrete in compression is neglected,

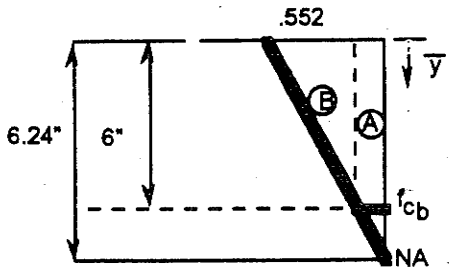
$$\text{arm} \doteq d - \frac{x}{3} \text{ for this example.}$$

$$\text{arm} \doteq 26.61 - \frac{6.24}{3} = 24.53 \text{ in}$$

and capacity is

$$M = A_s f_s (\text{arm}) = (6.89)(18)(24.53) = 3042.2 \text{ in-k} = 253 \text{ ft-k} \quad \text{as before.}$$

The exact "arm" may be determined from the concrete stress diagram as follows:



@ bottom of slab

$$f_{cb} = .552\left(\frac{.24}{6.24}\right) = 0.021$$

Next find centroid of stress diagram from top of slab.

$$\bar{y} = \frac{\Sigma A_y}{\Sigma A} = \frac{(0.021)(6)(6/2) + (0.552 - 0.021)(6)(1/2)(6/3)}{(0.021)(6) + (0.552 - 0.021)(6)(1/2)}$$

$$\bar{y} = \frac{3.576}{1.722} = 2.08 \text{ in}$$

$\therefore$ arm = 26.61 - 2.08 = 24.53 in as found previously.

(4) The Operating capacity may be found as above and will be the same as for the "traditional method." The rating calculations are not shown here since they too will be the same as for the traditional method.

Load Factor Rating (MANUAL 6.4.2, 6.5.3 & 6.6.3)

(For this example we consider only the maximum moment section - see General Notes.)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.2.1

$$I = \frac{50}{L + 125} \leq 0.30$$

$$I = \frac{50}{26 + 125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1, Concrete T-Beam

$$DF = \frac{S_G}{6.0} = \frac{6.52'}{6} = 1.087$$

Thus:

$$M_{LL+I} = M_L (1+I) \times DF = 111.1 (1+0.30)(1.087)$$

$$= 157 \text{ ft-k}$$

Capacity of Section - MANUAL 6.6.3.2

For unknown steel, prior to 1954 f_y = 33,000 psi = 33 ksi

M_u is found in accordance with applicable strength requirements of AASHTO Article 8.16.

Consider a rectangular section with compression limited to top slab. Then check MANUAL 6.6.3.2 requirement for 75% of balanced condition.

$$\rho_{max} = 0.75\, \rho_{bal} = 0.75 \frac{0.85\, \beta_1\, f'_c}{f_y} \frac{87000}{87000 + f_y} \text{ (AASHTO Eqn. 8-18)}$$

$$\rho_{max} = 0.75 \frac{0.85(.85)(3000)}{33000} \left(\frac{87000}{87000 + 33000} \right)$$

$$\rho_{max} = 0.0357$$

$\rho_{act} = 0.0036 << \rho_{max}$ OK (see Sheet 97)

Then:

$$a = \frac{A_s f_y}{0.85 f'_c b_{eff}} \qquad \text{(AASHTO Eqn. 8-17)}$$

$$a = \frac{6.89 \text{ in}^2 (33 \text{ ksi})}{0.85(3 \text{ ksi}) 72 \text{ in}} = 1.24" < 6" \qquad \text{OK within slab}$$

$$M_R = A_s f_y (d - a/2) \qquad \text{(AASHTO Eqn. 8-16)}$$

$$M_R = (6.89 \text{ in}^2)(33 \text{ ksi})\left(26.61 \text{ in} - \frac{1.24}{2}\right)$$

$$M_R = 5909 \text{ in-k} = \underline{\underline{492 \text{ ft-k}}}$$

$$M_u = \phi M_R \qquad \phi\text{: AASHTO 8.16.1.2.2} \rightarrow \phi = 0.90$$

$$M_u = 0.90 \times 492 = 443 \text{ ft-k}$$

Inventory Level: MANUAL 6.5.1 & 6.6.3

$$R_I^{LF} = \frac{M_u - A_1 M_D}{A_2 M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where in accordance with MANUAL 6.5.3

$$A_1 = 1.3$$
$$A_2 = 2.17$$

Thus:

$$RF_I^{LF} = \frac{443 - 1.3 (109.9)}{2.17(157)} = 0.88$$

Operating Level: MANUAL 6.5.1 & 6.6.3

$$R_O^{LF} = \frac{M_u - A_1 M_D}{A_2 M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where in accordance with MANUAL 6.5.3

$$\gamma_D = 1.3$$
$$\gamma_L = 1.3$$

Thus:

$$RF_O^{LF} = \frac{443 - 1.3(109.9)}{1.3(157)} = 1.47$$

Load capacity based on Load Factor Method, HS20 truck

Inventory: $0.88 \times 36^T = 31.7^T$ HS
Operating: $1.47 \times 36^T = 52.9^T$ HS

and

Inventory: $0.88 \times 1.068^X \times 20 = 18.8^T$ H (see Sheet 102)
Operating: $1.47 \times 1.068^X \times 20 = 31.4^T$ H

Load and Resistance Factor Rating (See AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*.)

(For this example we consider only the maximum moment section - see General Notes.)

Impact - Guide 3.3.2.3

Based on conditions at site (see sheet 96) select:

$$I = 0.1$$

Distribution - Guide 3.3.3 use standard AASHTO with correction factor of 1.0 (Guide Table 1).

AASHTO 3.23.2.2 and Table 3.23.1, Concrete T-Beam

$$DF = \frac{S_G}{6.0} = \frac{6.52}{6} = 1.087$$

Live Load - Guide 3.3.2.2 use HS20 to be consistent with other rating methods. Normally would use rating vehicles (Guide Fig. 2) or lane loading (Guide Fig. 3).

Thus:

$$M_{LL+I} = M_L\,(1+I)\,DF = 111.1(1+0.1)(1.087)$$
$$= 133 \text{ ft-k}$$

Capacity of Section - MANUAL 6.6.3.2

$$M_R = M_N = \frac{M_u}{\phi} \text{ found in accordance with AASHTO Article 8.16}$$

$$M_R = 492 \text{ ft-k (from sheet 105)}$$

Rating Level

$$RF^{LRF} = \frac{\phi\, M_R - A_1\, M_D}{A_2\, M_{LL+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where:

ϕ = 0.95 (Guide 3.3.4.2, Table 3(b)) From Sheet 96, concrete girder, redundant, good inspection and maintenance and good condition.

A_1 = 1.2 (Guide, Table 2) AC overlay measured in field (from sheet 96)

A_2 = 1.45 (Guide, Table 2) ADTT > 1000 and good enforcement (from sheet 96)

Then:

$$R^{LRF} = \frac{0.95(492) - 1.2(109.9)}{1.45(133)} = 1.74$$

Load Capacity Based on LRF:

$$1.74 \times 36^T = 62.6 \text{ tons HS}$$

and

$$1.74 \times 1.068 \times 20^T = 37.2^T \text{ H (see sheet 102)}$$

SUMMARY OF RESULTS

Method	RF	HS Truck Max. Load (tons)	H Truck Max. Load (tons)
Allowable Stress:			
Inventory	0.91	32.8	19.4
Operating	1.54	55.4	32.9
Load Factor:			
Inventory	0.88	31.7	18.8
Operating	1.47	52.9	31.4
Load and Resistance Factor	1.74	62.6	37.2

MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER ABUTMENT PILE RATING WORKSHEET

(EXAMPLE WITH NO DECAY)

Sheet No. 1 of 6
Bridge No. 1895
Made By Beam
Check By _____
Date 3/4/08

Location: Rye Rd. over Bourbon Creek

Given information:
- A simple span nail laminated bridge, two lanes. Timber piles in new condition.
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built: 1970
- Timber species: Douglas fir-larch, (coastal region)

Unit Definitions

$k \equiv 1000 \cdot lbf$ $ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $klf \equiv \frac{1000 \cdot lbf}{ft}$ $kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $kft \equiv 1000 \cdot lbf \cdot ft$ $ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $ton \equiv 2000 \cdot lbf$

Objective : Load rate the 12" diam. timber abutment piles driven to 20 tons

Reference Mn/DOT Spec. 3471

Input

Reference AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)

Commercial Grade: Dense Select Structural

Size Class: Posts

$F_{B} := 1750 \cdot psi$ (bending)

$F_{Cpar} := 1150 \cdot psi$ (compression parallel to grain)

$E := 1700000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$\gamma_{FPsoil} := 33 \cdot pcf$ (Equivalent fluid pressure of soil)

$Span := 30 \cdot ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 28 \cdot ft$ (Width of roadway)

$Width_{curb} := 1 \cdot ft$ (Width of curb)

$Thick_{deck} := 14 \cdot in$ (Thickness of deck)

$W_{TimRail} := 50 \cdot plf$ (Weight of timber rail)

Input (cont)

$Thick_{WC} := 3 \cdot in$ (Thickness of Bituminous Wearing Course)

$PileHt_{above} := 2 \cdot ft$ (Height of pile above top of berm)

$PileHt_{below} := 6 \cdot ft$ (Height of pile from top of berm to assumed point of fixity)

$Ht_{BW} := 5 \cdot ft$ (Height of abutment backwall from bottom of deck downward)

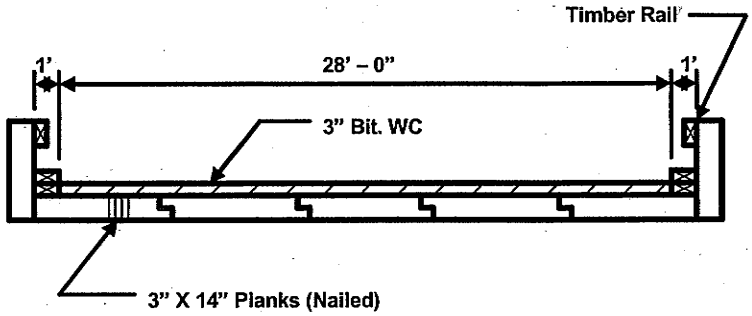
$PileSpa := 7.75 \cdot ft$ (Pile spacing)

$PileDiam := 12in$ (Pile diameter)

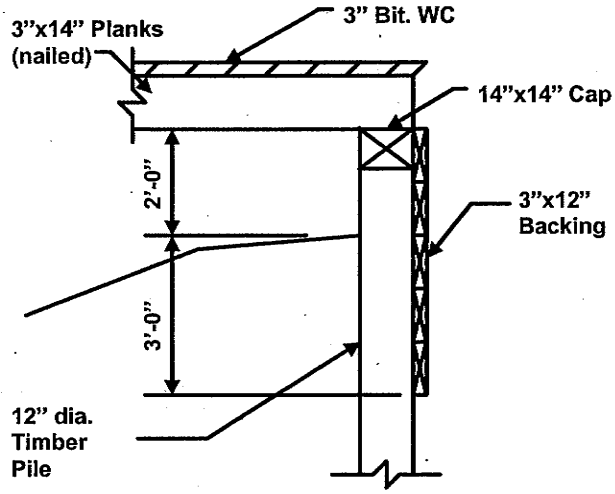
$Piles := 5$ (Number of piles)

$Depth_{abutcap} := 14 \cdot in$ (Depth of abutment cap)

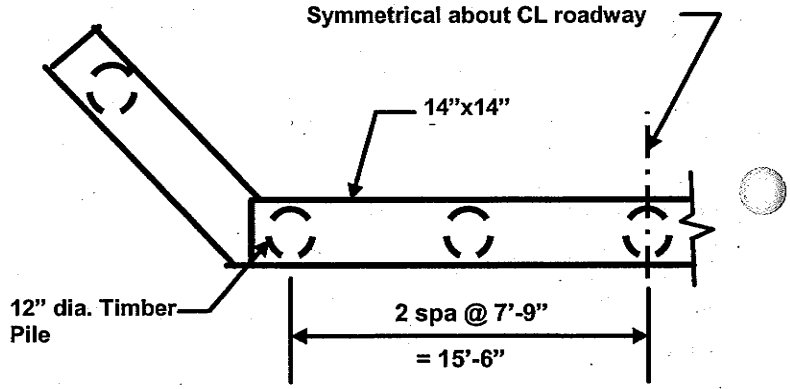
$Width_{abutcap} := 14 \cdot in$ (Width of abutment cap)



SECTION THRU TIMBER DECK



SECTION THRU ABUTMENT



HALF PLAN OF ABUTMENT

Dead Loads

Superstructure

DL of Deck

$$DL_{deck} := \left[Width_{rdwy} + \left(Width_{curb} \cdot 2\right)\right] \cdot Thick_{deck} \cdot Dens_{timb} \qquad DL_{deck} = 1750\,plf$$

DL of rail

$$DL_{rail} := W_{TimRail} \cdot 2 \qquad DL_{rail} = 100\,plf$$

DL of wearing course

$$DL_{WC} := Width_{rdwy} \cdot Thick_{WC} \cdot Dens_{bit} \qquad DL_{WC} = 1050\,plf$$

Total Dead Load of superstructure

$$DL_T := DL_{deck} + DL_{rail} + DL_{WC} \qquad DL_T = 2900\,plf$$

Substructure

DL of Abut. Cap

$$DL_{Abutcap} := Depth_{abutcap} \cdot Width_{abutcap} \cdot [[PileSpa \cdot (Piles - 1)] + (2 \cdot 1.5 \cdot ft)] \cdot Dens_{timb}$$

$$DL_{Abutcap} = 2.31\,k$$

Compute total dead load reaction to piles

$$R_{DLTotal} := \frac{DL_T \cdot Span}{2} + DL_{Abutcap}$$

$R_{DLTotal} = 45.814\,k$

Dead load reaction per pile

$$R_{DLpile} := \frac{R_{DLTotal}}{Piles}$$

$R_{DLpile} = 9.163\,k$

Live Loads

Determine number of lanes

$$Lanes := floor\left(\frac{Width_{rdwy}}{12 \cdot ft}\right)$$

$Lanes = 2$

Reference AASHTO Appendix A for HS20-44 truck loading

$$R_{LL} := if\left[Span \le 14 \cdot ft, 32 \cdot k, if\left[Span \le 28 \cdot ft, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span}, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span} + \frac{8 \cdot k \cdot (Span - 28 \cdot ft)}{Span}\right]\right]$$

$R_{LL} = 49.6\,k$ **(Live Load End Reaction per lane due to HS20 Truck Loading)**

$R_{LLTotal} := R_{LL} \cdot Lanes$ **(Total Live Load End Reaction per abutment due to HS20 Truck Loading)**

$R_{LLTotal} = 99.2\,k$

Note: Per AASHTO 3.8.1.2 - No impact for timber members

Live load reaction per pile

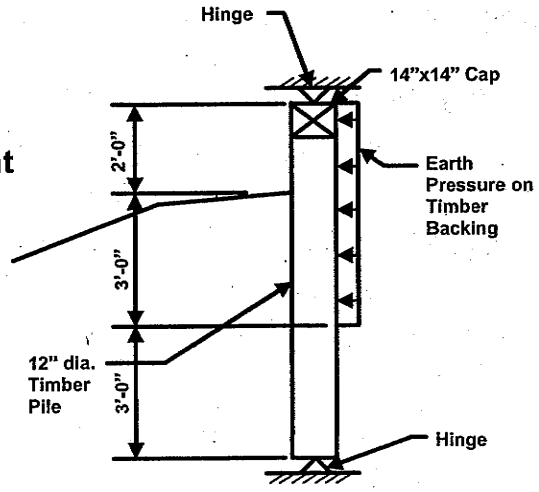
$$R_{LLpile} := \frac{R_{LLTotal}}{Piles}$$

$R_{LLpile} = 19.84\,k$

Lateral Earth Pressure

Assume:

- Free draining granular fill behind backwall
- Equivalent fluid pressure γsoil=33 pcf
- Pile support point approximately 6 foot below abutment berm
- Hinge support conditions



SECTION THRU PILE

$P_{Top} := \gamma_{FPsoil} \cdot (Thick_{deck} + Thick_{WC})$ **(Lateral pressure at bearing)** $P_{Top} = 46.75\,psf$

$P_{Bottom} := \gamma_{FPsoil} \cdot (PileHt_{above} + PileHt_{below})$ **(Lateral pressure at pile support point)** $P_{Bottom} = 264\,psf$

$P_{Ave} := \frac{P_{Top} + P_{Bottom}}{2}$ **(Average lateral pressure of P_{Top} and P_{Bottom})** $P_{Ave} = 155.38\,psf$

$P_{Pile} := P_{Ave} \cdot PileSpa$ **(Lateral pressure on pile)** $P_{Pile} = 1204.16\,plf$

$R_{Top} := \frac{P_{Pile} \cdot Ht_{BW}}{2 \cdot (PileHt_{above} + PileHt_{below})} \cdot [2 \cdot (PileHt_{above} + PileHt_{below}) - Ht_{BW}]$ **(Reaction at top of pile)** $R_{Top} = 4.14\,k$

$M_{EPress} := \frac{R_{Top}^2}{2 \cdot P_{Pile}}$ **(Moment in pile from earth pressure (k-ft) Ref. AISC ASD 9th Ed. Beam Diagrams No. 5, pg 2-297)** $M_{EPress} = 7.114\,kft$

Pile Section Properties

Area : $A_{pile} := \frac{\pi \cdot PileDiam^2}{4}$ $A_{pile} = 113.1\,in^2$

Moment of Inertia : $I_{pile} := \frac{\pi \cdot PileDiam^4}{64}$ $I_{pile} = 1017.88\,in^4$

Section Modulus : $S_{pile} := \frac{\pi \cdot PileDiam^3}{32}$ $S_{pile} = 169.65\,in^3$

Radius of Gyration : $r_{pile} := \frac{PileDiam}{4}$ $r_{pile} = 3\,in$

Equivalent Square column dimension : $d_{equiv} := \sqrt{A_{pile}}$ $d_{equiv} = 10.63\,in$

Note: The load on a round pile may be taken as the same as that for a square column with the same cross sectional area.

Timber Adjustment Factors

C_M = Wet service factor - Art. 13.5.5.1 **(Assume all bridge timbers to exceed 19% moisture)**

$C_{Mc} := 0.91$ **(For Compression)**

$C_{Mb} := 1.0$ **(For Bending)**

C_D = Load duration factor - Art 13.5.5.2

$C_D := 0.9$

C_F = Bending Size factor - Art. 13.5.1A Footnotes

$C_F := 1.0$

C_f = Bending form factor for members with circular cross section- Art. 13.6.4.5

$C_f := 1.18$

Calculate Column Stability factor (C_P) - Art. 13.7.3.3

$F_{Ctab} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F$ **(Tabulated stress in compression parallel to grain)** $F_{Ctab} = 941.85\,psi$

$L_{pile} := PileHt_{above} + PileHt_{below}$ **(L_{pile}= Actual column length between points of lateral support in inches)**

$L_{pile} = 96\,in$

$K_{pile} := 2.0$ **(K_{pile}= Effective length factor from AASHTO Table C-1, Appendix C**

$L_{eff} := K_{pile} \cdot L_{pile}$ $L_{eff} = 192\,in$

$K_{CE} := 0.30$ **(Visually graded lumber)**

$c_{pile} := 0.85$ **(For round piles)**

$C_{ME} := 1.0$ **(Wet service factor for Mod. of Elasticity (E) - from Table 13.5.1A, for timber 5" x 5" and larger)**

$E_{Prime} := E \cdot C_{ME}$ $E_{Prime} = 1700000\,psi$

$$F_{CE} := \frac{K_{CE} \cdot (E_{Prime})}{\left(\frac{L_{eff}}{d_{equiv}}\right)^2} \qquad F_{CE} = 1564.66\,psi$$

$$C_P := \left(\frac{1 + \frac{F_{CE}}{F_{Ctab}}}{2 \cdot c_{pile}}\right) - \sqrt{\frac{\left(1 + \frac{F_{CE}}{F_{Ctab}}\right)^2}{(2 \cdot c_{pile})^2} - \frac{\frac{F_{CE}}{F_{Ctab}}}{c_{pile}}}$$ **(Column Stability factor (C_P))** $C_P = 0.861$

Notes: - For short columns, crushing of wood fibers would likely control
- For intermediate columns, crushing of wood fibers or lateral buckling may control
- For long columns, lateral buckling would likely control

Determine allowable unit stresses for compression parallel to grain

$$F_C' = F_C \times C_M \times C_D \times C_F \times C_P$$

$F_{Cprime} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F \cdot C_P$ $F_{Cprime} = 810.96\,psi$

Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_f$$

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F \cdot C_f$ $F_{Bprime} = 1858.5\,psi$

Determine actual bending stress

$$f_b := \frac{M_{EPress}}{S_{pile}} \qquad f_b = 503.24\,psi$$

Columns must satisfy the following for combined bending and axial compression stresses

$$f_c/F_C' + f_b/F_b' <= 1.0$$

Solve for f_c to find maximum allowed compression parallel to grain

$$f_{cmax} := \left(1.0 - \frac{f_b}{F_{Bprime}}\right) \cdot F_{Cprime} \qquad f_{cmax} = 591.37\,psi$$

Calculate Inventory Level Rating

$$P_{RI} := f_{cmax} \cdot A_{pile} \qquad P_{RI} = 33.44\,ton$$

$$P_{RIcont} := if\left(P_{RI} > 20 \cdot ton, 20 \cdot ton, P_{RI}\right)$$ **(20 ton max. allowed for driving)** $P_{RIcont} = 20\,ton$ **or** $P_{RIcont} = 40\,k$

$$RF_I := \frac{P_{RIcont} - R_{DLpile}}{R_{LLpile}} \qquad RF_I = 1.55$$

Calculate Operating Level Rating

$$P_{RO} := P_{RIcont} \cdot 1.33 \qquad P_{RO} = 53.2\,k$$

$$RF_O := \frac{P_{RO} - R_{DLpile}}{R_{LLpile}} \qquad RF_O = 2.22$$

Inventory Rating

$$Inv_{Rating} := RF_I \cdot 20$$ **HS** $Inv_{Rating} = 31$

Operating Rating

$$Op_{Rating} := RF_O \cdot 20$$ **HS** $Op_{Rating} = 44$

MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER ABUTMENT PILE RATING WORKSHEET

(EXAMPLE WITH PILE DECAY)

Sheet No. 1 of 6
Bridge No. 1895
Made By Beam
Check By _____
Date 3/4/08

Location: Rye Rd. over Bourbon Creek

Given information:
- A simple span nail laminated bridge, two lanes.
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built: 1970
- Timber species: Douglas fir-larch, (coastal region)
- Piles are found to have internal decay with pile section loss at 50%.

Unit Definitions

$k \equiv 1000 \cdot lbf$ $ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $klf \equiv \frac{1000 \cdot lbf}{ft}$ $kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $kft \equiv 1000 \cdot lbf \cdot ft$ $ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $ton \equiv 2000 \cdot lbf$

Objective : Load rate the 12" diam. timber abutment piles driven to 20 tons
Reference Mn/DOT Spec. 3471

ıput

Reference AASHTO Table 13.5.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: Dense Select Structural
Size Class: Posts

$F_B := 1750 \cdot psi$ (bending)

$F_{Cpar} := 1150 \cdot psi$ (compression parallel to grain)

$E := 1700000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$\gamma_{FPsoil} := 33 \cdot pcf$ (Equivalent fluid pressure of soil)

$Span := 30 \cdot ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 28 \cdot ft$ (Width of roadway)

$Width_{curb} := 1 \cdot ft$ (Width of curb)

$Thick_{deck} := 14 \cdot in$ (Thickness of deck)

$W_{TimRail} := 50 \cdot plf$ (Weight of timber rail)

Input (cont)

$Thick_{WC} := 3 \cdot in$ (Thickness of Bituminous Wearing Course)

$PileHt_{above} := 2 \cdot ft$ (Height of pile above top of berm)

$PileHt_{below} := 6 \cdot ft$ (Height of pile from top of berm to assumed point of fixity)

$Ht_{BW} := 5 \cdot ft$ (Height of abutment backwall from bottom of deck downward)

$PileSpa := 7.75 \cdot ft$ (Pile spacing)

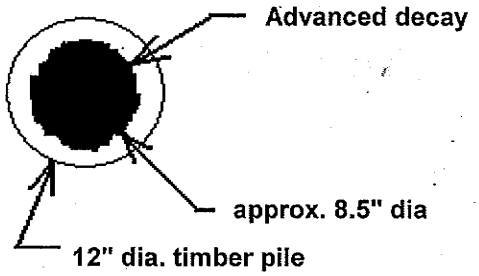
$PileDiam := 12in$ (Pile diameter)

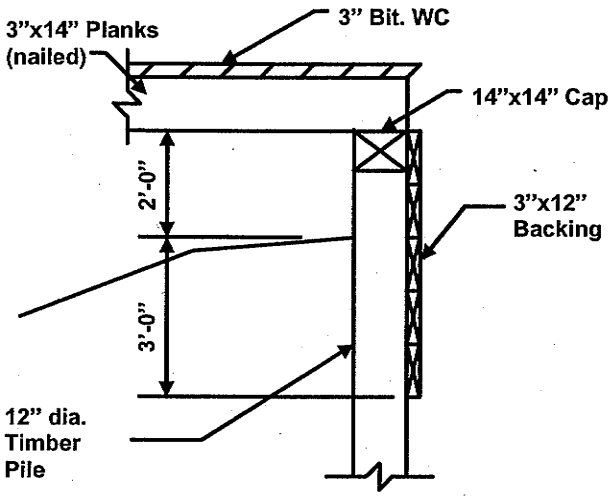
$PileDiam_{decay} := 8.5 \cdot in$ (Diameter of internal decayed section)

$Piles := 5$ (Number of piles)

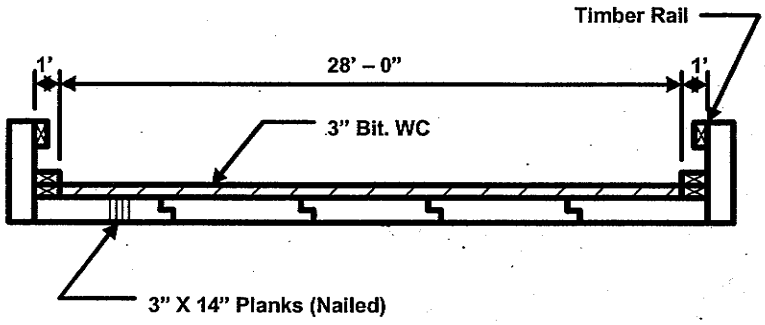
$Depth_{abutcap} := 14 \cdot in$ (Depth of abutment cap)

$Width_{abutcap} := 14 \cdot in$ (Width of abutment cap)

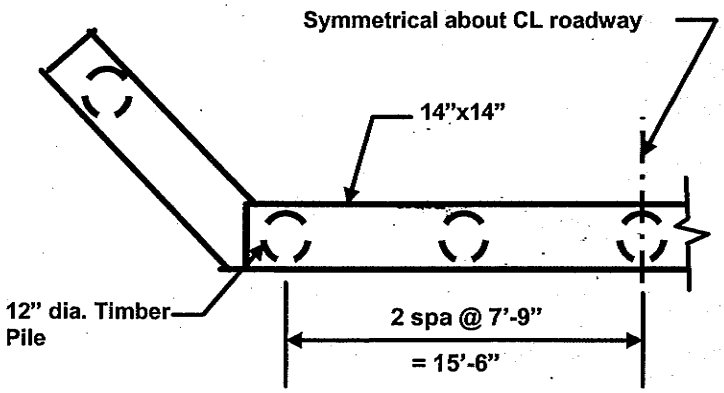




SECTION THRU ABUTMENT



SECTION THRU TIMBER DECK



HALF PLAN OF ABUTMENT

Dead Loads

Superstructure

DL of Deck	$DL_{deck} := \left[Width_{rdwy} + \left(Width_{curb} \cdot 2\right)\right] \cdot Thick_{deck} \cdot Dens_{timb}$	$DL_{deck} = 1750\,plf$
DL of rail	$DL_{rail} := W_{TimRail} \cdot 2$	$DL_{rail} = 100\,plf$
DL of wearing course	$DL_{WC} := Width_{rdwy} \cdot Thick_{WC} \cdot Dens_{bit}$	$DL_{WC} = 1050\,plf$
Total Dead Load of superstructure	$DL_T := DL_{deck} + DL_{rail} + DL_{WC}$	$DL_T = 2900\,plf$

Substructure

DL of Abut. Cap $DL_{Abutcap} := Depth_{abutcap} \cdot Width_{abutcap} \cdot [[PileSpa \cdot (Piles - 1)] + (2 \cdot 1.5 \cdot ft)] \cdot Dens_{timb}$

$DL_{Abutcap} = 2.31\,k$

Compute total dead load reaction to piles

$$R_{DLTotal} := \frac{DL_T \cdot Span}{2} + DL_{Abutcap}$$

$R_{DLTotal} = 45.81\,k$

Dead load reaction per pile

$$R_{DLpile} := \frac{R_{DLTotal}}{Piles}$$

$R_{DLpile} = 9.16\,k$

Live Loads

Determine number of lanes

$$Lanes := floor\left(\frac{Width_{rdwy}}{12 \cdot ft}\right)$$

$Lanes = 2$

Reference AASHTO Appendix A for HS20-44 truck loading

$$R_{LL} := if\left[Span \le 14 \cdot ft, 32 \cdot k, if\left[Span \le 28 \cdot ft, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span}, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span} + \frac{8 \cdot k \cdot (Span - 28 \cdot ft)}{Span}\right]\right]$$

$R_{LL} = 49.6\,k$ **(Live Load End Reaction per lane due to HS20 Truck Loading)**

$R_{LLTotal} := R_{LL} \cdot Lanes$ **(Total Live Load End Reaction per abutment due to HS20 Truck Loading)** $R_{LLTotal} = 99.2\,k$

Note: Per AASHTO 3.8.1.2 - No impact for timber members

Live load reaction per pile

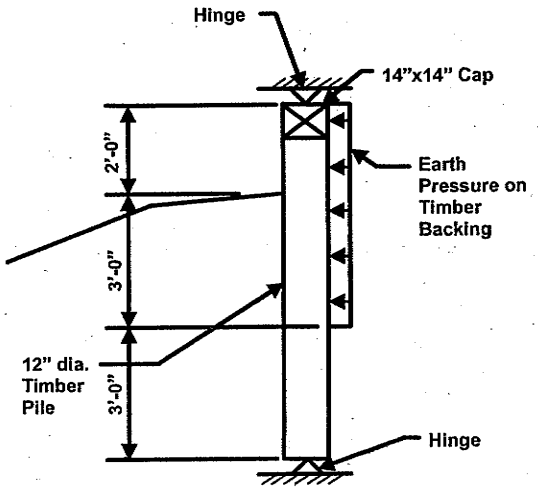
$$R_{LLpile} := \frac{R_{LLTotal}}{Piles}$$

$R_{LLpile} = 19.84\,k$

Lateral Earth Pressure

Assume:

- Free draining granular fill behind backwall
- Equivalent fluid pressure $\gamma soil$=33 pcf
- Pile support point approximately 6 foot below abutment berm
- Hinge support conditions



SECTION THRU PILE

$P_{Top} := \gamma_{FPsoil} \cdot (Thick_{deck} + Thick_{WC})$ **(Lateral pressure at bearing)** $P_{Top} = 46.75\,psf$

$P_{Bottom} := \gamma_{FPsoil} \cdot (PileHt_{above} + PileHt_{below})$ **(Lateral pressure at pile support point)** $P_{Bottom} = 264\,psf$

$P_{Ave} := \frac{P_{Top} + P_{Bottom}}{2}$ **(Average lateral pressure of P_{Top} and P_{Bottom})** $P_{Ave} = 155.38\,psf$

$P_{Pile} := P_{Ave} \cdot PileSpa$ **(Lateral pressure on pile)** $P_{Pile} = 1204.16\,plf$

$R_{Top} := \frac{P_{Pile} \cdot Ht_{BW}}{2 \cdot (PileHt_{above} + PileHt_{below})} \cdot [2 \cdot (PileHt_{above} + PileHt_{below}) - Ht_{BW}]$ **(Reaction at top of pile)** $R_{Top} = 4.14\,k$

$M_{EPress} := \frac{R_{Top}^2}{2 \cdot P_{Pile}}$ **(Moment in pile from earth pressure (k-ft) Ref. AISC ASD 9th Ed. Beam Diagrams No. 5, pg 2-297)** $M_{EPress} = 7.114\,kft$

Pile Section Properties

Area : $A_{pile} := \frac{\pi \cdot (PileDiam^2 - PileDiam_{decay}^2)}{4}$ $A_{pile} = 56.35\,in^2$

Moment of Inertia : $I_{pile} := \frac{\pi \cdot (PileDiam^4 - PileDiam_{decay}^4)}{64}$ $I_{pile} = 761.64\,in^4$

Section Modulus : $S_{pile} := \frac{\pi \cdot (PileDiam^4 - PileDiam_{decay}^4)}{32 \cdot PileDiam}$ $S_{pile} = 126.94\,in^3$

Radius of Gyration : $r_{pile} := \frac{\sqrt{PileDiam^2 + PileDiam_{decay}^2}}{4}$ $r_{pile} = 3.676\,in$

Equivalent Square column dimension : $d_{equiv} := \sqrt{A_{pile}}$ $d_{equiv} = 7.51\,in$

Note: The load on a round pile may be taken as the same as that for a square column with the same cross sectional area.

Timber Adjustment Factors

C_M = Wet service factor - Art. 13.5.5.1 **(Assume all bridge timbers to exceed 19% moisture)**

$C_{Mc} := 0.91$ **(For Compression)**

$C_{Mb} := 1.0$ **(For Bending)**

C_D = Load duration factor - Art 13.5.5.2

$C_D := 0.9$

C_F = Bending Size factor - Art. 13.5.1A Footnotes

$C_F := 1.0$

C_f = Bending form factor for members with circular cross section- Art. 13.6.4.5

$C_f := 1.18$

Calculate Column Stability factor (C_P) - Art. 13.7.3.3

$F_{Ctab} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F$ **(Tabulated stress in compression parallel to grain)** $F_{Ctab} = 941.85\,psi$

$L_{pile} := PileHt_{above} + PileHt_{below}$ **(L_{pile}= Actual column length between points of lateral support in inches)**

$L_{pile} = 96\,in$

$K_{pile} := 2.0$ **(K_{pile}= Effective length factor from AASHTO Table C-1, Appendix C**

$L_{eff} := K_{pile} \cdot L_{pile}$ $L_{eff} = 192\,in$

$K_{CE} := 0.30$ **(Visually graded lumber)**

$c_{pile} := 0.85$ **(For round piles)**

$C_{ME} := 1.0$ **(Wet service factor for Mod. of Elasticity (E) - from Table 13.5.1A, for timber 5" x 5" and larger)**

$E_{Prime} := E \cdot C_{ME}$ $E_{Prime} = 1700000\,psi$

$$F_{CE} := \frac{K_{CE} \cdot (E_{Prime})}{\left(\frac{L_{eff}}{d_{equiv}}\right)^2} \qquad F_{CE} = 779.61\,psi$$

$$C_P := \left(\frac{1 + \frac{F_{CE}}{F_{Ctab}}}{2 \cdot c_{pile}}\right) - \sqrt{\frac{\left(1 + \frac{F_{CE}}{F_{Ctab}}\right)^2}{(2 \cdot c_{pile})^2} - \frac{\frac{F_{CE}}{F_{Ctab}}}{c_{pile}}}$$ **(Column Stability factor (C_P))** $C_P = 0.648$

Notes: - For short columns, crushing of wood fibers would likely control
- For intermediate columns, crushing of wood fibers or lateral buckling may control
- For long columns, lateral buckling would likely control

Determine allowable unit stresses for compression parallel to grain

$$F_C' = F_C \times C_M \times C_D \times C_F \times C_P$$

$F_{Cprime} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F \cdot C_P$ $F_{Cprime} = 610.69\,psi$

Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_f$$

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F$ **Note: C_f was set to 1.0 conservatively due to rotting** $F_{Bprime} = 1575\,psi$

Determine actual bending stress

$$f_b := \frac{M_{EPress}}{S_{pile}} \qquad f_b = 672.55\,psi$$

Columns must satisfy the following for combined bending and axial compression stresses

$$f_c/F_C' + f_b/F_b' <= 1.0$$

Solve for f_c to find maximum allowed compression parallel to grain

$$f_{cmax} := \left(1.0 - \frac{f_b}{F_{Bprime}}\right) \cdot F_{Cprime} \qquad f_{cmax} = 349.92\,psi$$

Calculate Inventory Level Rating

$$P_{RI} := f_{cmax} \cdot A_{pile} \qquad P_{RI} = 9.86\,ton$$

$$P_{RIcont} := if\left(P_{RI} > 20 \cdot ton, 20 \cdot ton, P_{RI}\right)$$ **(20 ton max. allowed for driving)**

$P_{RIcont} = 9.86\,ton$ **or** $P_{RIcont} = 19.719\,k$

$$RF_I := \frac{P_{RIcont} - R_{DLpile}}{R_{LLpile}} \qquad RF_I = 0.53$$

Calculate Operating Level Rating

$$P_{RO} := P_{RIcont} \cdot 1.33 \qquad P_{RO} = 26.23\,k$$

$$RF_O := \frac{P_{RO} - R_{DLpile}}{R_{LLpile}} \qquad RF_O = 0.86$$

Inventory Rating

$$Inv_{Rating} := RF_I \cdot 20 \qquad \text{HS} \quad Inv_{Rating} = 11$$

Operating Rating

$$Op_{Rating} := RF_O \cdot 20 \qquad \text{HS} \quad Op_{Rating} = 17$$

MINNESOTA DEPARTMENT OF TRANSPORTATION

ASR TIMBER ABUTMENT PILE AND PILE CAP RATING WORKSHEET

(EXAMPLE WITH LOSS OF PILE)

Sheet No. 1 of 11
Bridge No. 1895
Made By Beam
Check By _____
Date 3/4/08

Location: Rye Rd. over Bourbon Creek

Given information:
- A simple span nail laminated bridge, two lanes.
- Timber dimensions were field measured (actual)
- Poor maintenance and inspection
- Year built: 1970
- Timber species: Douglas fir-larch, (coastal region)
- First interior piles (pile "B") are found to have substantial internal decay with pile section loss at 100%.

Unit Definitions

$k \equiv 1000 \cdot lbf$ $ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $klf \equiv \frac{1000 \cdot lbf}{ft}$ $kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $kft \equiv 1000 \cdot lbf \cdot ft$ $ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $ton \equiv 2000 \cdot lbf$

Input

Piles From AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)
Commercial Grade: Dense Select Structural
Size Class: Posts

$F_{B} := 1750 \cdot psi$ (bending)

$F_{Cpar} := 1150 \cdot psi$ (compression parallel to grain)

$E := 1700000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$\gamma_{FPsoil} := 33 \cdot pcf$ (Equivalent fluid pressure of soil)

Abutment Cap From AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)
Commercial Grade: Select Structural
Size Class: Beams and stringers

$F_{Bcap} := 1600 \cdot psi$ (bending)

$F_{Vpar} := 85 \cdot psi$ (shear parallel to grain)

Objective : Load rate the 12" diam. timber abutment piles driven to 20 tons

Reference Mn/DOT Spec. 3471

Input (cont)

$Span := 30 \cdot ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 28 \cdot ft$ (Width of roadway)

$Width_{curb} := 1 \cdot ft$ (Width of curb)

$Thick_{deck} := 14 \cdot in$ (Thickness of deck)

$W_{TimRail} := 50 \cdot plf$ (Weight of timber rail)

$Thick_{WC} := 3 \cdot in$ (Thickness of Bituminous Wearing Course)

$PileHt_{above} := 2 \cdot ft$ (Height of pile above top of berm)

$PileHt_{below} := 6 \cdot ft$ (Height of pile from top of berm to assumed point of fixity)

$Ht_{BW} := 5 \cdot ft$ (Height of abutment backwall from bottom of deck downward)

$PileSpa := 7.75 \cdot ft$ (Pile spacing)

$PileDiam := 12 in$ (Pile diameter)

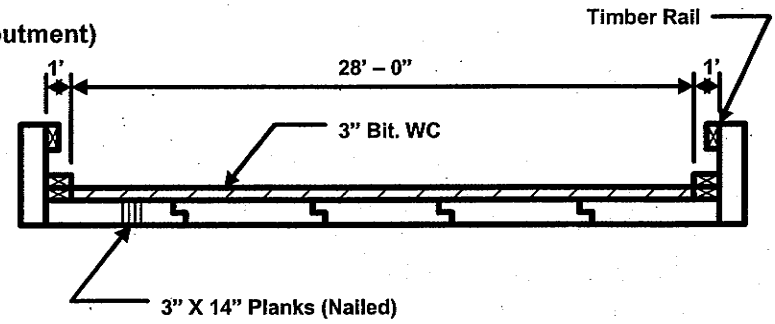
$PileDiam_{decay} := 0 \cdot in$ (Diameter of internal decayed section)

$Piles := 5$ (Total Number of piles per abutment)

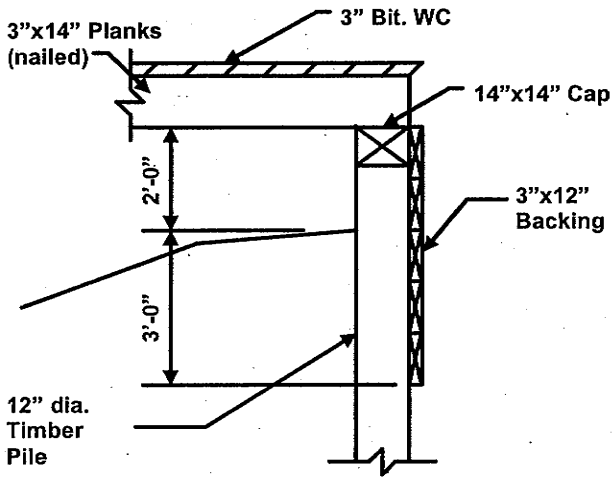
$Piles_{rotted} := 2$ (Number of rotted piles at abutment)

$Depth_{abutcap} := 14 \cdot in$ (Depth of abutment cap)

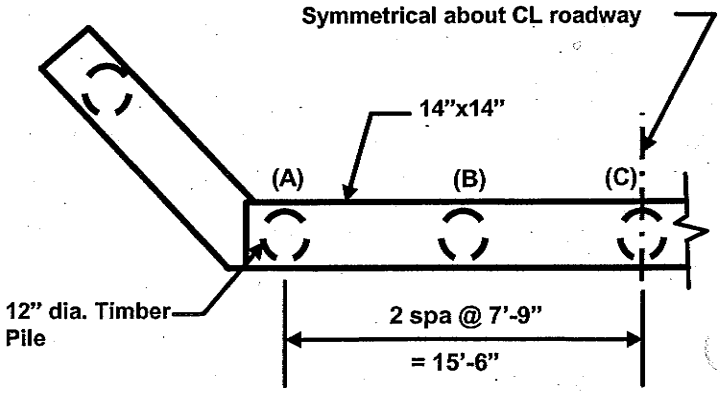
$Width_{abutcap} := 14 \cdot in$ (Width of abutment cap)



SECTION THRU TIMBER DECK



SECTION THRU ABUTMENT



HALF PLAN OF ABUTMENT

Dead Loads

Superstructure

DL of Deck $DL_{deck} := \left[Width_{rdwy} + \left(Width_{curb} \cdot 2\right)\right] \cdot Thick_{deck} \cdot Dens_{timb}$ $DL_{deck} = 1750\,plf$

DL of rail $DL_{rail} := W_{TimRail} \cdot 2$ $DL_{rail} = 100\,plf$

DL of wearing course $DL_{WC} := Width_{rdwy} \cdot Thick_{WC} \cdot Dens_{bit}$ $DL_{WC} = 1050\,plf$

Total Dead Load of superstructure $DL_T := DL_{deck} + DL_{rail} + DL_{WC}$ $DL_T = 2900\,plf$

Substructure

DL of Abut. Cap $DL_{Abutcap} := Depth_{abutcap} \cdot Width_{abutcap} \cdot [[PileSpa \cdot (Piles - 1)] + (2 \cdot 1.5 \cdot ft)] \cdot Dens_{timb}$

$DL_{Abutcap} = 2.31\,k$

Compute total dead load reaction to piles

$$R_{DLTotal} := \frac{DL_T \cdot Span}{2} + DL_{Abutcap}$$ $R_{DLTotal} = 45.81\,k$

Dead load reaction per pile

$$R_{DLpile} := \frac{R_{DLTotal}}{Piles - Piles_{rotted}}$$ $R_{DLpile} = 15.27\,k$

Live Loads

Determine number of lanes

$$Lanes := floor\left(\frac{Width_{rdwy}}{12 \cdot ft}\right)$$ $Lanes = 2$

Reference AASHTO Appendix A for HS20-44 truck loading

$$R_{LL} := if\left[Span \le 14 \cdot ft, 32 \cdot k, if\left[Span \le 28 \cdot ft, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span}, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span} + \frac{8 \cdot k \cdot (Span - 28 \cdot ft)}{Span}\right]\right]$$

$R_{LL} = 49.6\,k$ **(Live Load End Reaction per lane due to HS20 Truck Loading)**

$R_{LLTotal} := R_{LL} \cdot Lanes$ **(Total Live Load End Reaction per abutment due to HS20 Truck Loading)** $R_{LLTotal} = 99.2\,k$

Note: Per AASHTO 3.8.1.2 - No impact for timber members

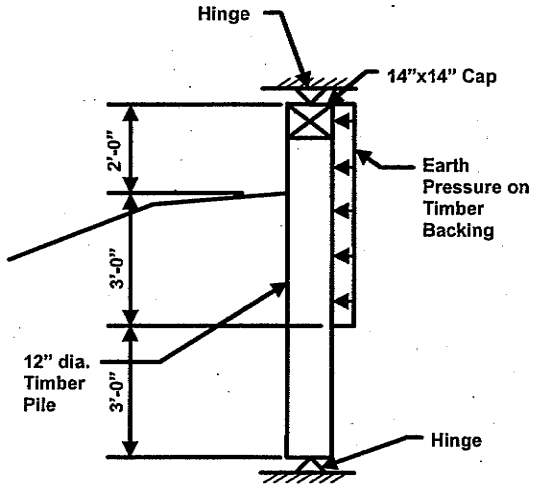
Live load reaction per pile

$$R_{LLpile} := \frac{R_{LLTotal}}{Piles - Piles_{rotted}}$$ $R_{LLpile} = 33.067\,k$

Lateral Earth Pressure

Assume

- Free draining granular fill behind backwall
- Equivalent fluid pressure γsoil=33 pcf
- Pile support point approximately 6 foot below abutment berm
- Hinge support conditions



SECTION THRU PILE

$P_{Top} := \gamma_{FPsoil} \cdot (Thick_{deck} + Thick_{WC})$ **(Lateral pressure at bearing)** $P_{Top} = 46.75\,psf$

$P_{Bottom} := \gamma_{FPsoil} \cdot (PileHt_{above} + PileHt_{below})$ **(Lateral pressure at pile support point)** $P_{Bottom} = 264\,psf$

$P_{Ave} := \frac{P_{Top} + P_{Bottom}}{2}$ **(Average lateral pressure of P_{Top} and P_{Bottom})** $P_{Ave} = 155.38\,psf$

$P_{Pile} := P_{Ave} \cdot PileSpa$ **(Lateral pressure on pile)** $P_{Pile} = 1204.16\,plf$

$R_{Top} := \frac{P_{Pile} \cdot Ht_{BW}}{2 \cdot (PileHt_{above} + PileHt_{below})} \cdot [2 \cdot (PileHt_{above} + PileHt_{below}) - Ht_{BW}]$ **(Reaction at top of pile)** $R_{Top} = 4.14\,k$

$M_{EPress} := \frac{R_{Top}^2}{2 \cdot P_{Pile}}$ **(Moment in pile from earth pressure (k-ft) Ref. AISC ASD 9th Ed. Beam Diagrams No. 5, pg 2-297)** $M_{EPress} = 7.114\,kft$

Pile Section Properties

Area : $A_{pile} := \frac{\pi \cdot (PileDiam^2 - PileDiam_{decay}^2)}{4}$ $A_{pile} = 113.1\,in^2$

Moment of Inertia : $I_{pile} := \frac{\pi \cdot (PileDiam^4 - PileDiam_{decay}^4)}{64}$ $I_{pile} = 1017.88\,in^4$

Section Modulus : $S_{pile} := \frac{\pi \cdot (PileDiam^4 - PileDiam_{decay}^4)}{32 \cdot PileDiam}$ $S_{pile} = 169.65\,in^3$

Radius of Gyration : $r_{pile} := \frac{\sqrt{PileDiam^2 + PileDiam_{decay}^2}}{4}$ $r_{pile} = 3\,in$

Equivalent Square column dimension : $d_{equiv} := \sqrt{A_{pile}}$ $d_{equiv} = 10.63\,in$

Note: The load on a round pile may be taken as the same as that for a square column with the same cross sectional area.

Timber Adjustment Factors

C_M = Wet service factor - Art. 13.5.5.1 & Table 13.5.1A (Assume all bridge timbers to exceed 19% moisture)

$C_{Mc} := 0.91$ (For Compression)

$C_{Mb} := 1.0$ (For Bending)

C_D = Load duration factor - Art 13.5.5.2

$C_D := 0.9$

C_F = Bending Size factor - Art. 13.5.1A Footnotes

$C_F := 1.0$

C_f = Bending form factor for members with circular cross section- Art. 13.6.4.5

$C_f := 1.18$

Calculate Column Stability factor (C_P) - Art. 13.7.3.3

$F_{Ctab} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F$ (Tabulated stress in compression parallel to grain) $F_{Ctab} = 941.85\,psi$

$L_{pile} := PileHt_{above} + PileHt_{below}$ (L_{pile}= Actual column length between points of lateral support in inches)

$L_{pile} = 96\,in$

$K_{pile} := 2.0$ (K_{pile}= Effective length factor from AASHTO Table C-1, Appendix C

$L_{eff} := K_{pile} \cdot L_{pile}$ $L_{eff} = 192\,in$

$K_{CE} := 0.30$ (Visually graded lumber)

$c_{pile} := 0.85$ (For round piles)

$C_{ME} := 1.0$ (Wet service factor for Mod. of Elasticity (E) - from Table 13.5.1A, for timber 5" x 5" and larger)

$E_{Prime} := E \cdot C_{ME}$ $E_{Prime} = 1700000\,psi$

$$F_{CE} := \frac{K_{CE} \cdot (E_{Prime})}{\left(\frac{L_{eff}}{d_{equiv}}\right)^2} \qquad F_{CE} = 1564.66\,psi$$

$$C_P := \left(\frac{1 + \frac{F_{CE}}{F_{Ctab}}}{2 \cdot c_{pile}}\right) - \sqrt{\frac{\left(1 + \frac{F_{CE}}{F_{Ctab}}\right)^2}{(2 \cdot c_{pile})^2} - \frac{\frac{F_{CE}}{F_{Ctab}}}{c_{pile}}}$$ (Column Stability factor (C_P)) $C_P = 0.861$

Notes: - For short columns, crushing of wood fibers would likely control
- For intermediate columns, crushing of wood fibers or lateral buckling may control
- For long columns, lateral buckling would likely control

Determine allowable unit stresses for compression parallel to grain

$$F_C' = F_C \times C_M \times C_D \times C_F \times C_P$$

$F_{Cprime} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F \cdot C_P$ $\quad F_{Cprime} = 810.96\,psi$

Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_f$$

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F$ **Note: C_f was set to 1.0 conservatively due to rotting** $\quad F_{Bprime} = 1575\,psi$

Determine actual bending stress

$$f_b := \frac{M_{EPress}}{S_{pile}} \qquad f_b = 503.24\,psi$$

Columns must satisfy the following for combined bending and axial compression stresses

$$f_c/F_C' + f_b/F_b' <= 1.0$$

Solve for f_c to find maximum allowed compression parallel to grain

$$f_{cmax} := \left(1.0 - \frac{f_b}{F_{Bprime}}\right) \cdot F_{Cprime} \qquad f_{cmax} = 551.85\,psi$$

Calculate Inventory Level Rating

$P_{RI} := f_{cmax} \cdot A_{pile}$ $\quad P_{RI} = 31.21\,ton$

$P_{RIcont} := if(P_{RI} > 20 \cdot ton, 20 \cdot ton, P_{RI})$ **(20 ton max. allowed for driving)** $\quad P_{RIcont} = 20\,ton$ **or** $P_{RIcont} = 40\,k$

$$RF_I := \frac{P_{RIcont} - R_{DLpile}}{R_{LLpile}} \qquad RF_I = 0.75$$

Calculate Operating Level Rating for piles

$P_{RO} := P_{RIcont} \cdot 1.33$ $\quad P_{RO} = 53.2\,k$

$$RF_O := \frac{P_{RO} - R_{DLpile}}{R_{LLpile}} \qquad RF_O = 1.152$$

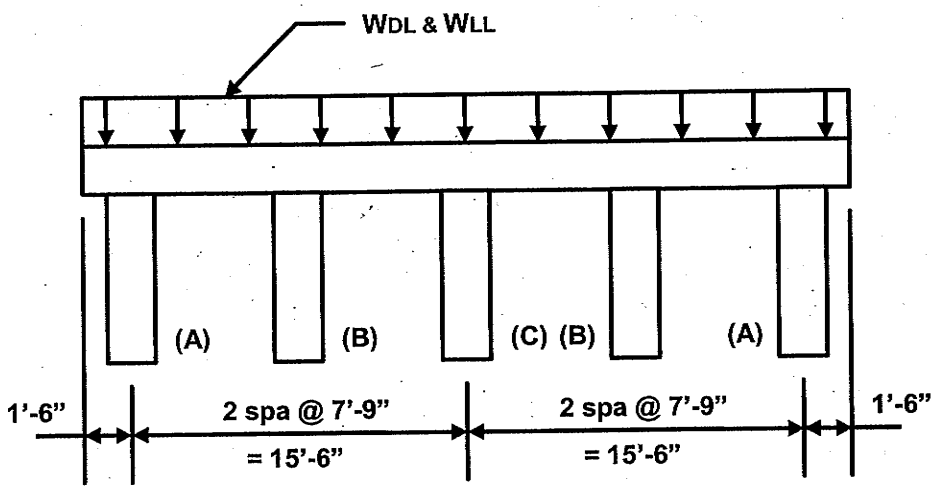
Inventory Rating

$Inv_{Rating} := RF_I \cdot 20$ **HS** $Inv_{Rating} = 15$

Operating Rating

$Op_{Rating} := RF_O \cdot 20$ **HS** $Op_{Rating} = 23$

Check Rating based on 14" x 14" abutment cap



Dead Load on abutment cap

$$W_{DLcap} := \frac{R_{DLTotal}}{[PileSpa\cdot(Piles - 1)] + (2\cdot 1.5\cdot ft)}$$ (Dead Load per foot of cap) $W_{DLcap} = 1.35\,klf$

Live Load on abutment cap

$$W_{LLcap} := \frac{R_{LLTotal}}{[PileSpa\cdot(Piles - 1)] + (2\cdot 1.5\cdot ft)}$$ (Live Load per foot of cap - 2 lanes of HS20 truck loading) $W_{LLcap} = 2.92\,klf$

- Reference continuous beam charts for two equal spans with uniform load on both spans
- Moment maximum at center pile support (C).

Moments in abutment cap

$$M_{DLSupC} := \frac{W_{DLcap}\cdot(PileSpa\cdot 2)^2}{8}$$ (Dead load moment at pile support) $M_{DLSupC} = 40.47\,kft$

$$M_{LLSupC} := \frac{W_{LLcap}\cdot(PileSpa\cdot 2)^2}{8}$$ (Live load moment at pile support) $M_{LLSupC} = 87.62\,kft$

Shear in abutment cap

$$V_{DLSupC} := \frac{5}{8} W_{DLcap}\cdot(PileSpa\cdot 2)$$ (Dead load shear at CL pile support) $V_{DLSupC} = 13.05\,k$

$$V_{LLSupC} := \frac{5}{8} W_{LLcap}\cdot(PileSpa\cdot 2)$$ (Live load shear at CL pile support) $V_{LLSupC} = 28.26\,k$

- Per AASHTO 13.6.5.2 - For uniformly distributed loads, the maximum shear occurs at a distance from the support equal to the bending member depth "d".

- Therefore, check shear at: $x_{shear} := (PileSpa \cdot 2) - \frac{PileDiam}{2} - Depth_{abutcap}$ $x_{shear} = 13.83\,ft$

$$V_{DLSupd} := V_{DLSupC} \cdot \frac{x_{shear} - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}{(PileSpa \cdot 2) - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}$$ **(Dead load shear at "d" from pile support)** $V_{DLSupd} = 10.81\,k$

$$V_{LLSupd} := V_{LLSupC} \cdot \frac{x_{shear} - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}{(PileSpa \cdot 2) - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}$$ **(Live load shear at "d" from pile support)** $V_{LLSupd} = 23.4\,k$

Abutment Cap Section Properties

Area : $A_{Abutcap} := Width_{abutcap} \cdot Depth_{abutcap}$ $A_{Abutcap} = 196\,in^2$

Moment of Inertia : $I_{Abutcap} := \frac{Width_{abutcap} \cdot Depth_{abutcap}^3}{12}$ $I_{Abutcap} = 3201.33\,in^4$

Section Modulus : $S_{Abutcap} := \frac{I_{Abutcap}}{\frac{Depth_{abutcap}}{2}}$ $S_{Abutcap} = 457.33\,in^3$

Timber Adjustment Factors

C_M = Wet service factor - Art. 13.5.5.1 & Table 13.5.1A (Assume all bridge timbers to exceed 19% moisture)

$C_{Mb} := 1.0$ **(For Bending - Timbers 5" x 5" or larger)**

$C_{Mv} := 1.0$ **(For Shear - Timbers 5" x 5" or larger)**

C_D = Load duration factor - Table 13.5.5A (Veh LL, 2 months)

$C_D := 1.15$

C_F = Bending Size factor - For lumber thicker than 5" - Art. 13.6.4.2.2

$$C_F := \left(\frac{12 \cdot in}{Depth_{abutcap}}\right)^{\frac{1}{9}}$$ $C_F = 0.98$

C_L = Beam Stability factor from Art. 13.6.4.4

$C_L := 1.0$ **(Assume no danger of lateral buckling)**

Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_L$$

$F_{BprimeCap} := F_{Bcap} \cdot C_{Mb} \cdot C_D \cdot C_F \cdot C_L$ $F_{BprimeCap} = 1808.75\,psi$

Determine allowable unit stresses for shear

$$F_V' = F_V \times C_M \times C_D$$

$F_{VprimeCap} := F_{Vpar} \cdot C_{Mv} \cdot C_D$ $F_{VprimeCap} = 97.75\,psi$

Check Inventory and Operating Rating of Abutment Cap

Calculate Inventory and Operating Stresses

Bending

$F_{BInvCap} := F_{BprimeCap}$ **(Inventory stress = Allowable unit stress)** $F_{BInvCap} = 1808.75\,psi$

$F_{BOpCap} := F_{BprimeCap} \cdot 1.33$ **(Operating stress)** $F_{BOpCap} = 2405.64\,psi$

Shear

$F_{VInvCap} := F_{VprimeCap}$ **(Inventory stress = Allowable unit stress)** $F_{VInvCap} = 97.75\,psi$

$F_{VOpCap} := F_{VprimeCap} \cdot 1.33$ **(Operating stress)** $F_{VOpCap} = 130.01\,psi$

Calculate Inventory Rating Factor for Bending in Cap

$M_{RIb} := F_{BInvCap} \cdot S_{Abutcap}$ $M_{RIb} = 68.93\,kft$

$$RF_{Ib} := \frac{M_{RIb} - M_{DLSupC}}{M_{LLSupC}}$$ $RF_{Ib} = 0.32$

Calculate Operating Rating Factor for Bending in Cap

$M_{ROb} := F_{BOpCap} \cdot S_{Abutcap}$ $M_{ROb} = 91.68\,kft$

$$RF_{Ob} := \frac{M_{ROb} - M_{DLSupC}}{M_{LLSupC}}$$ $RF_{Ob} = 0.58$

Inventory Rating for Bending in Cap

$Inv_{RatingB} := RF_{Ib} \cdot 20$ **HS** $Inv_{RatingB} = 6$

Operating Rating for Bending in Cap

$Op_{RatingB} := RF_{Ob} \cdot 20$ **HS** $Op_{RatingB} = 12$

Calculate Inventory Rating Factor for Shear in Cap

$V_{RI} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VInvCap}$ $V_{RI} = 12.77\,k$

$$RF_{IV} := \frac{V_{RI} - V_{DLSupd}}{V_{LLSupd}}$$ $RF_{IV} = 0.08$

Calculate Operating Rating Factor for Shear in Cap

$V_{RO} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VOpCap}$ $V_{RO} = 16.99\,k$

$RF_{OV} := \frac{V_{RO} - V_{DLSupd}}{V_{LLSupd}}$ $RF_{OV} = 0.26$

Inventory Rating for Shear in Cap

$Inv_{RatingV} := RF_{IV} \cdot 20$ HS $Inv_{RatingV} = 2$

Operating Rating for Shear in Cap

$Op_{RatingV} := RF_{OV} \cdot 20$ HS $Op_{RatingV} = 5$

For comparison purposes, check Inventory and Operating Rating for shear in abutment cap with all 5 piles in sound condition

(Reference AISC ASD 9th Ed. Beam Diagrams No. 39, pg 2-309.)

Shear in abutment cap

$V_{DLSupC} := 0.607 W_{DLcap} \cdot (PileSpa)$ **(Dead load shear at CL pile support)** $V_{DLSupC} = 6.34\,k$

$V_{LLSupC} := 0.607 W_{LLcap} \cdot (PileSpa)$ **(Live load shear at CL pile support)** $V_{LLSupC} = 13.73\,k$

- Check shear at "d" from CL pile : $x_{shear} := (PileSpa) - \frac{PileDiam}{2} - Depth_{abutcap}$ $x_{shear} = 6.08\,ft$

$V_{DLSupd} := V_{DLSupC} \cdot \frac{x_{shear} - [0.393 \cdot (PileSpa)]}{(PileSpa) - [0.393 \cdot (PileSpa)]}$ **(Dead load shear at "d" from pile support)** $V_{DLSupd} = 4.09\,k$

$V_{LLSupd} := V_{LLSupC} \cdot \frac{x_{shear} - [0.393 \cdot (PileSpa)]}{(PileSpa) - [0.393 \cdot (PileSpa)]}$ **(Live load shear at "d" from pile support)** $V_{LLSupd} = 8.86\,k$

Calculate Inventory Rating Factor for Shear in Cap (5 piles in sound condition)

$V_{RI} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VInvCap}$ $V_{RI} = 12.77\,k$

$RF_{IV} := \frac{V_{RI} - V_{DLSupd}}{V_{LLSupd}}$ $RF_{IV} = 0.98$

Calculate Operating Rating Factor for Shear in Cap (5 piles in sound condition)

$V_{RO} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VOpCap}$ $V_{RO} = 16.99\,k$

$RF_{OV} := \frac{V_{RO} - V_{DLSupd}}{V_{LLSupd}}$ $RF_{OV} = 1.45$

Inventory Rating for Shear in Cap (5 piles in sound condition)

$Inv_{RatingV} := RF_{IV} \cdot 20$ HS $Inv_{RatingV} = 20$

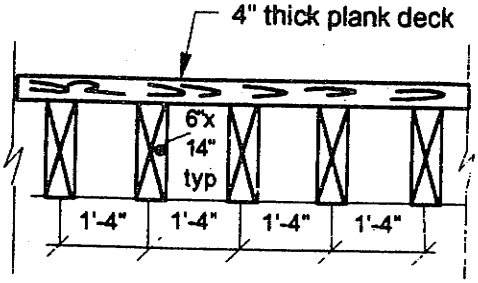
Operating Rating for Shear in Cap (5 piles in sound condition)

$Op_{RatingV} := RF_{OV} \cdot 20$ HS $Op_{RatingV} = 29$

2003 AASHTO "MCE"

EXAMPLE B3: - Timber Stringer

Given: A simple span, timber stringer bridge with a timber plank deck (two lanes). Span length - 17'-10" c-c bearings (field measured).



Partial X-Section
(nts)

Timber dimensions field measured (actual)

Good maintenance and inspection.

Smooth approaches, fair deck smoothness.

Year Built: 1930
Year Reconstructed: 1967

ADTT << 1000

Timber Species: Southern Pine No. 2

AASHTO Table 13.2.1A for No. 2:

$$F_b = 1200 \text{ psi}^* \text{ (new)} \; ; \; F_v = 90 \text{ psi (new)}$$

Load rate the 6"x14" stringers:

Dead Loads - Deck $\frac{(1'-4")4"}{144 \text{ in}^2/\text{ft}^2} \times 50 \text{ lbs/ft}^3 = 22.2 \text{ lbs/ft}$

Stringer $\frac{6"x14"}{144} \times 50 = \frac{29.2 \text{ lb/ft}}{51.4 \text{ lb/ft}}$ say 0.055 k/ft

Live Load - Rate for H15 truck

Section Properties: (Again for stringers)

$$I_x = \frac{bh^3}{12} = \frac{6x14^3}{12} = 1372 \text{ in}^4$$

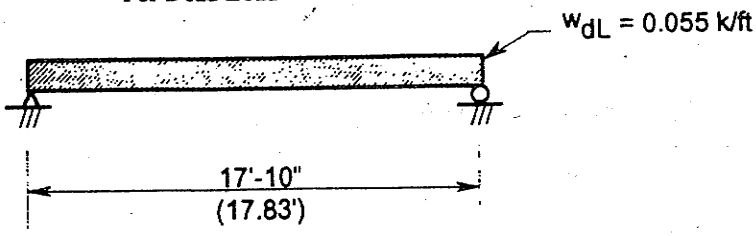
$$S_x = \frac{I_x}{h/2} = \frac{1372}{14/2} = 196 \text{ in}^3$$

$$A = bh = 6 \times 14 = 84 \text{ in}^2$$

* The provisions of AASHTO 13.3.7.1 should also be applied. For this sample $C_F = 1.0$ was assumed.

Midspan Moments:

For Dead Load -



$$M_D = \frac{w_{dL}\ L^2}{8} = \frac{0.055\ (17.83)^2}{8}$$

$$M_D = 2.19^{'k}$$

For H15 - From MANUAL Appendix A3, page 74(1)

Span	M_L	
17'	$51^{'k}$	
		← For 17.83' span, interpolate
18'	$54^{'k}$	

$$M_L = 51 + \frac{17.83 - 17}{18-17}(54-51) = 53.5^{'k}$$

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(Consider stringer only; consider maximum moment and shear sections only for this example - see General Notes.)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.1.2 - No impact for timber members

$$I = 0$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1
For two lanes and plank deck:

$$DF = \frac{S}{3.75} = \frac{16"/12"/ft}{3.75} = 0.36$$

Thus:

$$M_{LL+I} = M_L\ (1+I) \times DF = 53.5^{'k} \times (1+0) \times 0.36$$

$$M_{LL+I} = 19.26^{'k}$$

Stresses to be used:

Inventory: MANUAL 6.6.2.7(1) → use AASHTO

(1) Note the moment given in the MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUAL values.

For this specie F_b = 1200 psi

However since this bridge is more than 10 years old, AASHTO 13.2.4 applies.

$$F_b^{inv} = 0.9(1200)\, C_F = 1080 \text{ psi} = 1.08 \text{ ksi}$$

(C_F = 1.0 (see note sh. 108))

and

$$F_v^{inv} = 90 \times 0.9 = 81 \text{ psi}$$

Operating: MANUAL 6.6.2.7(2)

$$F_b^{op} = F_b^{inv} \times 1.33 \times C_F = 1080 \times 1.33 \times 1.0$$

(C_F = 1.0 (see note sh. 108))

$$F_b^{op} = 1436 \text{ psi say } 1440 \text{ psi} = 1.44 \text{ ksi}$$

and

$$F_v^{op} = 1.33\, F_v^{inv} = 1.33 \times 81 \text{ psi} = 108 \text{ psi}$$

Inventory Level Rating

Capacity: $M_{R_I} = F_b^{inv} S_x = 1.08 \text{ ksi} \times 196 \text{ in}^3 = 211.7 \text{ in-k}$

$M_{R_I} = 17.64$ ft-k

then

(MANUAL Eqn. 6-1a) $$RF_I^M = \frac{M_{R_I} - M_D}{M_{L+I}} = \frac{17.64^{'k} - 2.19^{'k}}{19.26^{'k}}$$

RF_I^M = 0.80 or 0.80 x 15 tons = 12 tons H truck

Operating Level Rating

Capacity: $M_{R_O} = F_b^{op} S_x = 1.44 \text{ ksi} \times 196 \text{ in}^3 = 282.2 \text{ in-k}$

$M_{R_O} = 23.52^{'k}$

then

(MANUAL Eqn. 6-1a) $$RF_O^M = \frac{M_{R_O} - M_D}{M_{L+I}} = \frac{23.52^{'k} - 2.19^{'k}}{19.26^{'k}}$$

RF_O^M = 1.11 or 1.11 x 15 tons = 16.6 tons H truck

Check Horizontal Shear

AASHTO 13.3.1 suggests that shear be computed at:

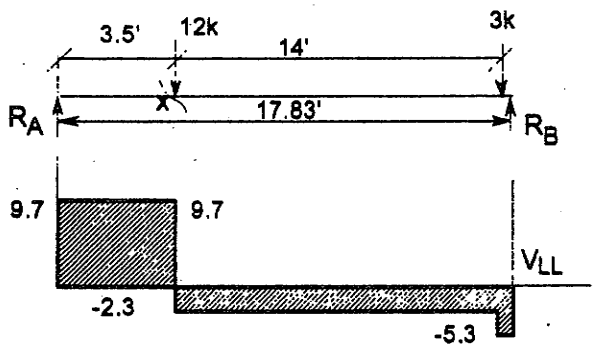
(1) A distance from the support equal to three times the depth of the stringer; or

(2) At the quarter point, whichever is less.

Thus by: (1) $3(14") = 42"$ ← Controls = 3.5 ft

(2) $\frac{17.83' \times 12"/ft}{4} = 53.5"$

For H15 Truck: MANUAL, Appendix A8, pg. 79



$$V_x = \frac{15(x - 2.8)}{L}$$

where L = 17.83' x = 17.83 - 3.5 = 14.33

$$V_x = \frac{15(14.33 - 2.8)}{17.83} = 9.7 \text{ k},$$

per wheel line without distribution

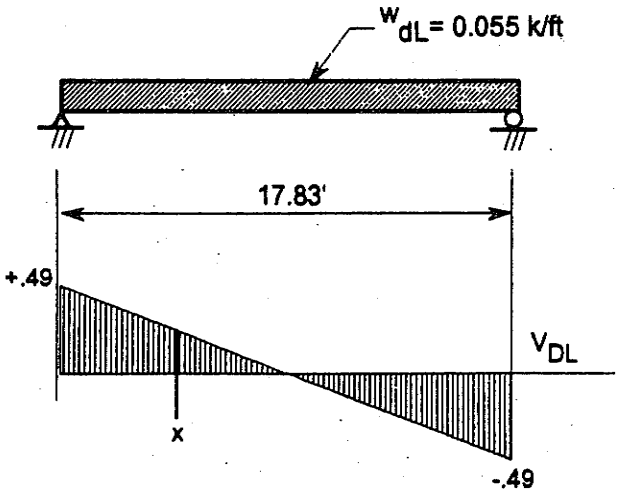
then per AASHTO 13.3.1

$$V_{Lx} = 1/2 \left[0.6\, V_x^{L \text{ no dist.}} + DF\, V_x^{L \text{no dist.}} \right]$$

$$V_{Lx} = 1/2 \left[0.6\,(9.7) + 0.36\,(9.7)\right]$$

$$V_{Lx} = 4.7 \text{ k}$$

For $w_{dL} = 0.055$ k/ft



$R_A = R_B = 1/2\, w_{dL}\, L$

$= 1/2\,(0.055) \times 17.83$

$= 0.49$ k

$V_{D_x} = 0.49 - .055 \times 3.5$

$V_{D_x} = 0.3$ k

Rating Based on Shear

Inventory:

Capacity: AASHTO Eqn. 13-1 solve for V_R

$$V_R = \frac{2}{3} bd f_v$$

then

$$V_{R_I} - \frac{2}{3}(6)(14)\ 81\ psi = 4536\ lbs. = 4.54\ k$$

(MANUAL Eqn. 6-1a): $$RF_I^V = \frac{V_{R_I} - V_{D_x}}{V_{L_x}} = \frac{4.54\ k - 0.3\ k}{4.7\ k}$$

$$RF_I^V = \underline{0.90} \text{ or } 0.90 \times 15 \text{ tons} = \underline{13.5 \text{ tons}} \text{ H truck}$$

Operating:

Capacity:

$$V_{R_O} = \frac{2}{3}(6)(14)(108\ psi) = 6048\ lbs. = 6.05\ k$$

(MANUAL Eqn. 6-1a): $$RF_O^V = \frac{V_{R_O} - V_{D_x}}{V_{L_x}} = \frac{6.05\ k - 0.3\ k}{4.7\ k}$$

$$RF_O^V = \underline{1.22} \text{ or } 1.22 \times 15 \text{ tons} = \underline{18.5 \text{ tons}} \text{ H truck}$$

Load Factor Rating

Not currently available for timber.

Load and Resistance Factor Rating

Not currently available for timber.

SUMMARY OF RESULTS

Method/Force	RF	H Truck Max. Load (tons)
Allowable Stress Moment:		
Inventory	0.80	12.0
Operating	1.11	16.6
Allowable Stress Shear:		
Inventory	0.90	13.5
Operating	1.22	18.3

∴ Rating governed by moment rather than shear

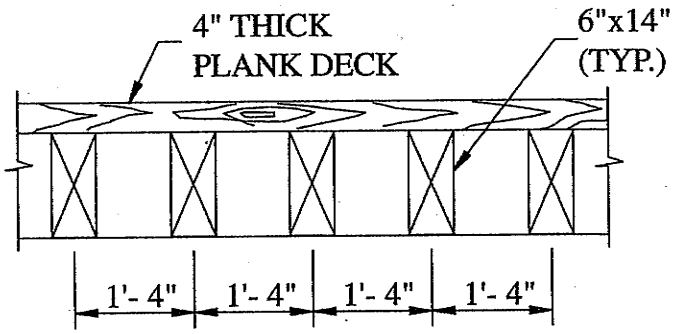
2005 Guide "MCE" LRFR

Example A4: **Timber Stringer Bridge**
Evaluation of an Interior Stringer

GIVEN:

Span:	17 ft. 10 in.
Year Built:	1930
Year Reconstructed:	1967
Material	Southern Pine No. 2
Condition:	No deterioration. NBI Item 59 Code = 6
Riding Surface:	Unknown condition
Traffic:	Two Lanes
ADTT (one direction):	150
Skew:	0 degrees

Note: Same bridge as in Example B3 1994 AASHTO MCE.



PARTIAL CROSS SECTION

I. CHECK MOMENT CAPACITY

DEAD LOAD ANALYSIS—INTERIOR STRINGER

1. Components and Attachments *DC*

Deck: $\frac{16}{12} \times \frac{4}{12} \times 0.050$ = 0.022 kip/ft. LRFD Table 3-3

Stringer: $\frac{6 \times 14}{144} \times 0.050$ = 0.029 kip/ft.

Total per stringer = 0.051 kip/ft.

Say = 0.055 kip/ft.

$$M_{DC} = \frac{1}{8} \times 0.055 \times 17.83^2$$

$$= 2.19 \text{ kip-ft.}$$

2. Wearing Surface *DW* = 0

LIVE LOAD ANALYSIS—INTERIOR STRINGER

I. Distribution Factor for Moment and Shear

The stringers are continuously braced by the deck and diaphragms (6 in. x 14 in.) are present at the Bearings; $C_s = 1.0$

$F_b = F_{bo} C_F C_M C_D$

$F'_{bo} = 3.3$ ksi Southern Pine No. 2 — LRFD Table 8-1

Size Factor: $C_F = 1.02$ 6×14 stringers — LRFD Table 8-7
(value in LRFD Table 8-7 for Width = 14 in. and Thickness = 4 in.)

Moisture Content Factor: $C_M = 1.0$ — LRFD 8.4.4.3

Deck Factor: $C_D = 1.0$ Plank Deck — LRFD 8.4.4.4

$F_b = (3.3)\ (1.02)\ (1.0)\ (1.0)$
$= 3.36$ ksi

Nominal Resistance: $M_n = F_b\ S\ C_s$

$$= (3.36)(196)(1.0)\left(\frac{1}{12}\right)$$

$= 54.9$ kip-ft.

GENERAL LOAD RATING EQUATION — 6.4.2

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_p)(P)}{(\gamma_L)(LL + IM)}$$ Eq. (6-1)

EVALUATION FACTORS (For Strength Limit State) — LRFD 8.5.2.2

a) Resistance Factor φ
$\varphi = 0.85$ for Flexure
$\varphi = 0.75$ for Shear

b) Condition Factor φ_c — 6.4.2.3
$\varphi_c = 1.0$ Good Condition

c) System Factor φ_s — 6.4.2.4
$\varphi_s = 1.0$ for flexure and shear in timber bridges

1. **DESIGN LOAD RATING** — 6.4.3

A) Strength I Limit State — 6.7.4.1

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi) R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + IM)}$$

a) Inventory Level

LOAD	LOAD FACTOR
DC	1.25
LL	1.75

Table 6-1

V_{LU} = maximum vertical shear at $3d$ or $L/4$ due to undistributed wheel loads (kips)

= For undistributed wheel loads, one line of wheels is assumed to be carried by one bending member. LRFD 4.6.2.2.2a

$$= \frac{V_{TANDEM}}{2} = (34.6 \times 1.165)/2 = 20.15 \text{ kips}$$

V_{LD} = maximum vertical shear at $3d$ or $L/4$ due to wheel loads distributed laterally as specified herein (kips)

$= V_{LL+IM} = 8.8$ kips

$$V_{LL} = 0.50[(0.60 \times 20.15) + 8.8] = 10.45 \text{ kips}$$

COMPUTE NOMINAL SHEAR RESISTANCE

LRFD Eq. (8-14)

$$V_n = \frac{F_v b d}{1.5}$$

$F_v = F_{vo}\ C_F\ C_M\ C_D$

F_{vo} = 0.300 ksi Southern Pine No. 2 — LRFD Eq. (8-1)

C_F = 1.02 — LRFD Table 8-1

C_M = 1.00

C_D = 1.00

F_v = (0.3) (1.02) (1.0) (1.0)

= 0.306 ksi

$$V_n = \frac{(0.306)(6)(14)}{1.5} = 17.1 \text{ kip}$$

a) Inventory Level

LOAD	LOAD FACTOR
DC	1.25
LL	1.75

$$\text{Shear } RF = \frac{(1.0)(1.0)(0.75)(17.1) - (1.25)(0.426)}{(1.75)(10.45)}$$

$$= 0.67$$

b) Operating Level

$$\text{Shear } RF = 0.67 \times \frac{1.75}{1.35} = 0.87$$

No service limit states apply.

Summary

Truck:	Type 3	Type 3S2	Type 3-3
Weight (tons)	25	36	40
RF	1.09	1.19	1.32
Safe Load Capacity (tons)	27	43	53

Summary of Rating Factors

EXAMPLE A4

INTERIOR STRINGER

Limit State		Design Load Rating		Legal Load Rating		
		Inventory	Operating	T3	T3S2	T3-3
Strength I	Flexure	0.55	0.71	1.09	1.19	1.32
	Shear	0.67	0.87	1.24	1.36	1.50

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

Sheet No. 1 of 1
Rated by JWD
Checked by
DATE 2-20-98

TIMBER BEAM RATING SHEET EXAMPLE

Side 1

BRIDGE LOCATION AND DESCRIPTION

Bridge No: 92841	Description: 1 SPAN OVER STREAM
Route: 175	
Roadway Width: 18'	Location:
Year Built: 1968	

Year Remodeled: — Slab Thickness: 5" PLAIN W.C.Thickness: 3" BIT

Span Rated: 1 Span Length(L): 25' Beam Spacing (S): 2.25' Impact: 0

SUMMARY OF RATING AND LOAD POSTING

INVENTORY RATING	OPERATING RATING	LOAD POSTING REQ'D ?	LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED)		
			Vehicle Type M3 Weight = 24T	Semi-Trailer Comb. Type M3S2 Weight = 36T	Truck & full Trailer Type M3-3 Weight = 40T
HS 13.2	HS 19.0	YES ____ NO ✓	____ Tons	____ Tons	____ Tons

RATING DATA

Dist. Factor (p. 32, AASHTO, 1997) = S/4 = .563

		INVENTORY	OPERATING
Critical Point Location	Number	℄	℄
Section Modulus (-loss)	in.^{3}	423.7	423.7
Allowable Stress (See reverse side)	Kips/in.^{2}	fi = 1.465	fo = 1.33 fi = 1.949
Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$	Ft. Kips	51.7 ①	68.8 ①
Dead Load Moment/Bm	Ft. Kips	13.2 ②	13.2 ②
Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + \text{Impact})(\text{Dist. Factor})}$	Ft. Kips	$\frac{51.7-13.2}{(1)(.563)} = 68.4$ ④	98.78 ④
HS 20 Moment per wheel line(See Ill. #9)	Ft. Kips	103.7 ⑤	103.7 ⑤
HS Rating $\frac{④}{⑤}$(20)	Number	HS 13.2	HS 19.0

DEAD LOAD DESCRIPTION	DL per ft. of Beam
Overburden = .25' x 2.25' x 144 #/C.F.	= 81.0 lb./ft.
Slab = 5" (50 #/C.F.)(2.25')	= 47.0 lb./ft.
Beam (Nominal Size = 6"x22") SIZED = 5.5" x 21.5" x 50 #/CF/144	= 41.0 lb./ft.
Railing, Diaphragms, etc. =	= — lb./ft.
Total Dead Load = W	= 169 lb./ft.
Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = $\frac{169 (25)^2}{8000}$	= 13.2 Ft-Kips

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BR 92841

TIMBER BEAM RATING SHEET EXAMPLE

ALLOWABLE STRESS FOR TIMBER BEAMS -- AASHTO 13.6.4.1

$fi = Fb' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$ For sawn lumber, only C_M, C_D C_F & C_r apply.

Note: For Glulam timber beams, C_F is not used. Use C_V per Section 13.6.4.3.

C_M- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, C_M = **0.85** unless Fb(C_F) are less then 1150, then C_M = 1.0.
(For glulam beam, from Table 13.5.3A, C_M = 0.80)

C_D-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), C_D = **1.15**.

C_F- Sect. 13.6.4.2 Lumber 2" to 4" thick use Table 13.5.1A. For lumber thicker then 5" $C_F = (12/d)^{1/9}$.
I.E. for 8 x 14 beam, $C_F = (12/14)^{1/9}$ = **0.983.**
C_r -Table 13.5.1A. For lumber 2" to 4" thick, C_r = 1.15. Otherwise C_r = 1.0.

As an example, if F_b = 1500psi, and beam is 8" x 14", then
fi = F'b = 1500(.85)(1.15)(0.983) =**1441 psi** allowable bending stress.
And fo = 1.33fi = **1917 psi**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 (allowable Shear parallel to grain).
fv(inv) = F'v = $F_V C_M C_D$. If F_V = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

Use Table 3.23.1 for Live load distribution for bending analysis.
Use Section 13.6.5.2 for live load distribution for shear analysis

ALLOWABLE STRESS FOR BENDING:

FROM PLAN; F_b = 1600 psi, C_M = 0.85, C_D = 1.15, $C_F = \left(\frac{12}{21.5}\right)^{1/9}$ = .937, C_V = 1.0

∴ $F_b' = fi$ = 1600(.85)(1.15)(.937)(1.0) = 1465 psi & fo = 1.33(1465) = 1949 psi

$SM = \frac{1}{6}(5.5)(21.5)^2$ = 423.7 in³

CHECK SHEAR: V_{DL} (at 'd' out) = .169(25' − 1.79') = 1.81ᵏ d = 1.79'

V_{LL} (UNDIST) = $\frac{16^k(19.13' + 5.63')}{25}$ = 16.16ᵏ

V_{LL} (DIST.) = 16.16 × .563 = 9.1ᵏ

SECT. 13.6.5.2

$V_{LL} = .5(1.6 \times 16.16 + 9.1)$ = 9.4ᵏ

INV. RATING = $\frac{V_{CAP} - V_{DL}(20)}{V_{LL}}$

$V_{CAP} = \frac{2}{3} F'v\,(b)(d)$

$V_{CAP} = 109.25(5.5)(21.5)\frac{2}{3}$ = 8612# = 8.612ᵏ

∴ INV. RATING (SHEAR) = $\frac{8.612 - 1.81(20)}{9.4}$ = HS 14.47 - NOT CRITICAL

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TIMBER BEAM RATING SHEET

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BRIDGE LOCATION AND DESCRIPTION

Bridge No:	Description:
Route:	
Roadway Width:	Location: _____
Year Built:	_____

Year Remodeled: _____ Slab Thickness: _____ W.C.Thickness: _____

Span Rated: _____ Span Length(L): _____ Beam Spacing (S): _____ Impact: _____

SUMMARY OF RATING AND LOAD POSTING

INVENTORY RATING	OPERATING RATING	LOAD POSTING REQ'D ?	LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED)		
			Vehicle Type M3 Weight = 24T	Semi-Trailer Comb. Type M3S2 Weight = 36T	Truck & full Trailer Type M3-3 Weight = 40T
HS_____	HS_____	YES_____ NO_____	_____Tons	_____Tons	_____Tons

RATING DATA

Dist. Factor (p. 32, AASHTO, 1997) = S/ = _____

		INVENTORY	OPERATING
Critical Point Location	Number		
Section Modulus (-loss)	in.^{3}		
Allowable Stress (See reverse side)	Kips/in.^{2}	fi =	fo = 1.33 fi =
Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$	Ft. Kips	①	①
Dead Load Moment/Bm	Ft. Kips	②	②
Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + Impact)(Dist. Factor)}$	Ft. Kips	④	④
HS 20 Moment per wheel line(See Ill. #9)	Ft. Kips	⑤	⑤
HS Rating $\frac{④}{⑤}$(20)	Number	HS _____	HS _____

DEAD LOAD DESCRIPTION	DL per ft. of Beam
Overburden =	= lb./ft.
Slab =	= lb./ft.
Beam (Nominal Size =)	= lb./ft.
Railing, Diaphragms, etc. =	= lb./ft.
Total Dead Load = W	= lb./ft.
Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ =	= Ft-Kips

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TIMBER BEAM RATING SHEET

ALLOWABLE STRESS FOR TIMBER BEAMS -- AASHTO 13.6.4.1

$fi = Fb' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$ For sawn lumber, only C_M, C_D C_F & C_r apply.

Note: For Glulam timber beams, C_F is not used. Use C_V per Section 13.6.4.3.

C_M- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, C_M = **0.85** unless Fb(C_F) are less then 1150, then C_M = 1.0.
(For glulam beam, from Table 13.5.3A, C_M = 0.80)

C_D-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), C_D = **1.15**.

C_F- Sect. 13.6.4.2 Lumber 2" to 4" thick use Table 13.5.1A. For lumber thicker then 5" $C_F = (12/d)^{1/9}$.
I.E. for 8 x 14 beam, $C_F = (12/14)^{1/9}$ = **0.983.**
C_r-Table 13.5.1A. For lumber 2" to 4" thick, C_r = 1.15. Otherwise C_r = 1.0.

As an example, if F_b = 1500psi, and beam is 8" x 14", then
fi = F'b = 1500(.85)(1.15)(0.983) =**1441 psi** allowable bending stress.
And fo = 1.33fi = **1917 psi**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 (allowable Shear parallel to grain).
fv(inv) = F'v = F_V C_M C_D. If F_V = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

Use Table 3.23.1 for Live load distribution for bending analysis.
Use Section 13.6.5.2 for live load distribution for shear analysis

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LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

EXAMPLE
NAILED PANEL

BRIDGE LOCATION AND DESCRIPTION

Bridge No: 03506	Description: 3 SPANS OVER OTTER TAIL RIVER
Route: TWN 300	
Roadway Width: 28'	Location: 1.4 M. N.E of Jct CSAH 29
Year Built: 1981	

Year Remodeled: — Slab Thickness: 3"x14" LAM. W.C.Thickness: 3" Asphalt

Span Rated: Span Length(L): 31'-2" Panel Width (Wp): 6'-0" Impact: 0

SUMMARY OF RATING AND LOAD POSTING

INVENTORY RATING	OPERATING RATING	LOAD POSTING REQ'D ?	LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED)		
			Vehicle Type M3 Weight = 24T	Semi-Trailer Comb. Type M3S2 Weight = 36T	Truck & full Trailer Type M3-3 Weight = 40T
HS 27.1	HS 38.8	YES NO V	Tons	Tons	Tons

RATING DATA

Dist. Factor (See Reverse Side) = D.F. = 1.5

		INVENTORY	OPERATING
Critical Point Location	Number	4	4
Section Modulus (-~~loss~~) $\frac{1}{6}$ 72 (14)^2	in.^{3}	2352	2352
Allowable Stress (See reverse side)	Kips/in.^{2}	fi = 1.517	fo = 1.33 fi = 2.018
Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$	Ft. Kips	297.3 ①	395.5 ①
Dead Load Moment/panel	Ft. Kips	70.0 ②	70.0 ②
Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + \text{Impact})(\text{Dist. Factor})}$	Ft. Kips	$\frac{297.3 - 70}{1(1.5)} = 151.55$ ④	217.0 ④
HS 20 Moment per wheel line(See Ill. #9)	Ft. Kips	111.8 ⑤	111.8 ⑤
HS Rating $\frac{④}{⑤}$(20)	Number	HS 27.1	HS 38.8

DEAD LOAD DESCRIPTION	DL per ft. of panel.
Overburden = .25'x6'x144 #/CF	= 216 lb./ft.
Lam. Deck (Nominal Size = 6'x14") 6'x1.17'x50 #/CF	= 351 lb./ft.
Railing, Diaphragms, etc. = Sp. Bm: .5x.83x6x50/15.6	= 8 lb./ft.
Total Dead Load = W	= 575 lb./ft.
Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = $\frac{.575(31.17)^2}{8}$	= 70 Ft-Kips

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NAILED PANEL EXAMPLE

LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

BR03506

LIVE LOAD DISTRIBUTION FACTOR:

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

Continuous Nailed Panels: AASHTO Section 3.25.2.2.

Dist. Factor/Wh. = Wp/wheel dist. width = $\frac{6'0}{4'0} = 1.5$

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness. = 14"

∴ Wh DIST = 20" + 2(14) = 48"

Wp = panel width PANEL 'C' = 6'-0"

Glue Lam. Panels: AASHTO Section 3.25.3.1.

2 lanes: D.F. = $\frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$: Use Greater, D. F. =

1 lane: D.F. = $\frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$: Use Greater, D. F. =

D.F. = Lateral Live Load distribution factor.

ALLOWABLE STRESS FOR TIMBER DESIGN -- AASHTO Section 13.6.4.1

fi = Fb' = $F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$ For sawn lumber and glulam, only C_M, C_D, C_F & C_r apply.

C_M- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, C_M = **0.85** for F_b unless F_b(C_F)are less then 1150, then C_M = 1.0.
(For glulam, Table 13.5.3A, C_M = 0.80)

C_D-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), C_D = **1.15**.

C_F- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., C_F = **0.9.**
C_r-Table 13.5.1A. For lumber 2" to 4" thick, C_r = 1.15

As an example, if Fb = 1500 psi, then
fi = F'b = 1500(.85)(1.15)(0.9)(1.15) =**1517 psi** allowable bending stress.
And fo = 1.33fi = **2018 psi.**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 (allowable Shear parallel to grain).
fv(inv) = F'v = $F_V C_M C_D$. If F_V = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

FOR THIS EXAMPLE, fi & fo ARE AS SHOWN ABOVE.

SHEAR CHECK: V_{DL} (at 'd' out) = .575$\left(\frac{31.17}{2} - 1.17\right)$ = 8.29^k/6' Panel, 'd' = 1.17

V_{LL} (UNDIST) = 16^k (77.67' − 73.67')/31.17' = 10.95^k (SHEAR FOR LL @ 3 ('d') out)

V_{LL} (DIST) = 10.95 × 1.5 = 16.42^k ∴ V_{LL} = .5(.6 × 10.95 + 16.42) = 11.5^k/6' Panel

INV. RATING = $\frac{V_{CAP} - V_{DL}(20)}{V_{LL}}$. $V_{CAP} = \frac{2}{3} F_V(b)(d) = \frac{2}{3}\frac{109.25}{1000}(72)(14)$ = 73.4^k

∴ INV. RATING (SHEAR) = $\frac{73.4 - 8.29(20)}{11.5}$ = HS 113 - NOT CRITICAL.

DEPARTMENT OF TRANSPORTATION

LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

Rated by _____
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BRIDGE LOCATION AND DESCRIPTION

Bridge No:	Description:
Route:	
Roadway Width:	Location: _____
Year Built:	_____

Year Remodeled:_____ Slab Thickness:_____ W.C.Thickness:_____

Span Rated:_____ Span Length(L):_____ Panel Width (Wp):_____ Impact:_____

SUMMARY OF RATING AND LOAD POSTING

INVENTORY RATING	OPERATING RATING	LOAD POSTING REQ'D ?	LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED)		
			Vehicle Type M3 Weight = 24T	Semi-Trailer Comb. Type M3S2 Weight = 36T	Truck & full Trailer Type M3-3 Weight = 40T
HS_____	HS_____	YES_____ NO_____	_____Tons	_____Tons	_____Tons

RATING DATA

Dist. Factor (See Reverse Side) = D.F. = _____

		INVENTORY	OPERATING
Critical Point Location	Number		
Section Modulus (-loss)	in.^{3}		
Allowable Stress (See reverse side)	Kips/in.^{2}	fi =	fo = 1.33 fi =
Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$	Ft. Kips	①	①
Dead Load Moment/panel	Ft. Kips	②	②
Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + \text{Impact})(\text{Dist. Factor})}$	Ft. Kips	④	④
HS 20 Moment per wheel line(See Ill. #9)	Ft. Kips	⑤	⑤
HS Rating $\frac{④}{⑤}(20)$	Number	HS _____	HS _____

DEAD LOAD DESCRIPTION	DL per ft. of panel.
Overburden =	= lb./ft.
Lam. Deck (Nominal Size =)	= lb./ft.
Railing, Diaphragms, etc. =	= lb./ft.
Total Dead Load = W	= lb./ft.
Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ =	= Ft-Kips

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ONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

LIVE LOAD DISTRIBUTION FACTOR:

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

Continuous Nailed Panels: AASHTO Section 3.25.2.2.

Dist. Factor/Wh. = Wp/wheel dist. width = _____

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness.

Wp = panel width

Glue Lam. Panels: AASHTO Section 3.25.3.1.

2 lanes: D.F. = $\frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$: Use Greater, D. F. =

1 lane: D.F. = $\frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$: Use Greater, D. F. =

D.F. = Lateral Live Load distribution factor.

ALLOWABLE STRESS FOR TIMBER DESIGN -- AASHTO Section 13.6.4.1

$fi = Fb' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$ For sawn lumber and glulam, only C_M, C_D, C_F & C_r apply.

C_M- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, C_M = **0.85** for F_b unless $F_b(C_F)$ are less then 1150, then C_M = 1.0.
(For glulam, Table 13.5.3A, C_M = 0.80)

C_D-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), C_D = **1.15**.

C_F- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., C_F = **0.9.**
C_r-Table 13.5.1A. For lumber 2" to 4" thick, C_r = 1.15

As an example, if Fb = 1500 psi, then
fi = F'b = 1500(.85)(1.15)(0.9)(1.15) =**1517 psi** allowable bending stress.
And fo = 1.33fi = **2018 psi**.

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 (allowable Shear parallel to grain).
$fv(inv) = F'v = F_V C_M C_D$. If F_V = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi**.

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LOGITUDINAL LAMINATED TIMBER DECK RATING SHEET

BRIDGE LOCATION AND DESCRIPTION

Bridge No. 31531
Route Co. RD. 253
Rdwy. Width 28'-1"
Span Length 22'-6"
Year Built 1994
Year Remodeled —

Description 2 Span Nail Laminated Timber Panel Bridge, Interior Panel
Location Co. Rd. 253 over the Little Bowstring River, Itasca Co.

Slab Thickness 14" W.C. Thickness 4"

SUMMARY OF RATING AND LOAD POSTING

Inventory Rating	Operating Rating	Load Posting Req'd ?	LOAD POSTING LIMITS @ (Complete when load posting is required)		
			Vehicle Type M3 Weight=24T	Semi-Trailer Comb. Type M3S2 Weight=36T	Truck & Full Trailer Type M3-3 Weight=40T
H _____ or HS 37.8	HS 52.2	____ Yes X No	____ Tons	____ Tons	____ Tons

RATING DATA

Dist. factor (See Reverse Side) = 1.58 Beam Spacing = 6'-4"

		INVENTORY	OPERATING
Critical Point Location	Number	℄	℄
Impact (I=0 for timber S.S.)	Number	0	0
Section Modulus (-loss) $\frac{1}{6} * 76" * 14^2 in^2$	In.^{3}	2482.7 (see Plan Sheet Note #6)	2482.7
Allowable Stress	Kips/In.^{2}	fi = ~~1.800~~ (1.5)	fo= 1.33 fy = ~~2.128~~ 1.995
Resisting Moment/BM= $\frac{SM(fi \text{ or } fo)}{12}$	Ft. Kips	310.3 ①	412.7 ①
ⓑ Dead Load Moment/BM	Ft. Kips	41.0 ②	41.0 ②
Mom. Avail. for LL/Wh. Line = $\frac{① - ②}{(1+Impact)(Dist.Factor)}$	Ft. Kips	170.4 ④	235.2 ④
ⓒ HS 20 Moment per wheelline	Ft. Kips	90.0 ⑤	90.0 ⑤
HS Rating $\frac{④}{⑤}$ (20)	Number	HS 37.8	HS 52.2

ⓐ Load Posting : For simple spans see Illustration #'s 3, 4 & 10
ⓑ Dead Load Moment: For Simple Spans, DL Mom. = $\frac{1}{8} \frac{W(L)^2}{1000} = \frac{1}{8} * \frac{647.4 \frac{lb}{ft} * 22.5^2 ft^2}{1000} = 41.0'\text{-}k$
ⓒ HS 20 Moment: For Simple Spans see Illustration #9

DEAD LOAD DESCRIPTION

		DL per ft. of Beam
Overburden 4" asphalt = $9 \frac{lb}{sq.ft.in} * 4 in * (6'-4")$	=	228 lb./
Lam. Deck (Size = 14" x 6'-4") $14" * 1 ft/12 in * (6'-4") * 50 lb/ft^3$	=	369.4 lb./ft.
Railing, Diaphragms, etc. Rail to ext. panels, Say misc. = 50 lb/ft	=	50.0 lb./ft.
Total Dead Load = W	=	647.4 lb./ft.

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3/9/94

Live Load Distribution Factor:

No Longitudinal Distribution Of Wheel Loads.

Lateral Distribution Is As Follows.

Continuous Nailed Panels: AASHTO, 3.25.2.2

Dist. Factor/Wh = $\frac{Wp}{S} = \frac{6'-4''}{\frac{20''+2(14)}{12}''} = 1.58$

S = Tire Width + 2(t), Tire Width = 20", t = Panel Thickness

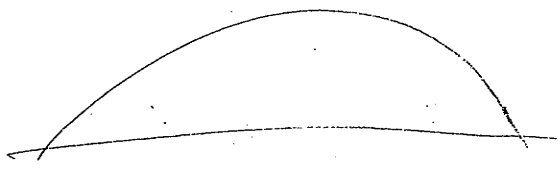
Wp = Panel ~~Thickness~~ Width

Glue Lam. Panels: AASHTO, 3.25.3.1

2 Lanes: D.F. = $\frac{Wp}{3.75+\frac{L}{28}}$ or $\frac{Wp}{5.00}$: Use Greater, D.F.=

1 Lane: D.F. = $\frac{Wp}{4.25+\frac{L}{28}}$ or $\frac{Wp}{5.50}$: Use Greater, D.F.=

D.F. = Lateral Live Load Distribution Factor.



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LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

EXAMPLE
GLULAM DECK

BRIDGE LOCATION AND DESCRIPTION

Bridge No: 10000	Description: 1 SPAN OVER STREAM
Route: —	
Roadway Width: 32'	Location: 2.0 M. W. of Jct. CSAH 4
Year Built: 1990	
Year Remodeled: — Slab Thickness: 3"x8¾" LAM. W.C.Thickness: 3" Asphalt	
Span Rated: Span Length(L): 20' Panel Width (Wp): 4'-0" Impact: 0	

SUMMARY OF RATING AND LOAD POSTING

INVENTORY RATING	OPERATING RATING	LOAD POSTING REQ'D?	LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED)		
			Vehicle Type M3 Weight = 24T	Semi-Trailer Comb. Type M3S2 Weight = 36T	Truck & full Trailer Type M3-3 Weight = 40T
HS 20.8	HS 29.0	YES ____ NO ✓	— Tons	— Tons	— Tons

RATING DATA

Dist. Factor (See Reverse Side) = D.F. = .896

		INVENTORY	OPERATING
Critical Point Location	Number	4	4
Section Modulus (-loss) $\frac{1}{6}(48)(8.75)^2$	in.^{3}	612.5	612.5
Allowable Stress (See reverse side)	Kips/in.^{2}	fi = 1.745	fo = 1.33 fi = 2.32
Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$	Ft. Kips	89.07 ①	118.41 ①
Dead Load Moment/panel	Ft. Kips	14.5 ②	14.5 ②
Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + \text{Impact})(\text{Dist. Factor})}$	Ft. Kips	$\frac{89.07 - 14.5}{.896} = 83.2$ ④	116.0 ④
HS 20 Moment per wheel line(See Ill. #9)	Ft. Kips	80.0 ⑤	80.0 ⑤
HS Rating $\frac{④}{⑤}$(20)	Number	HS 20.8	HS 29.0

DEAD LOAD DESCRIPTION	DL per ft. of panel.
Overburden = .25' x 4' x 144 #/CF	= 144 lb./ft.
Lam. Deck (Nominal Size = 8¾" x 4') (8.75 x 4 x 150 #/CF)/12	= 146 lb./ft.
Railing, Diaphragms, etc. = taken by Exterior Panel	= — lb./ft.
Total Dead Load = W	= 290 lb./ft.
Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = $\frac{290(20)^2}{8000}$ =	= 14.5 Ft-Kips

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

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GLULAM DECK EXAMPLE

Rated by JWD
Checked by
Date: 2-20-98
Side 2

LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

BR 10,000

LIVE LOAD DISTRIBUTION FACTOR:

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

Continuous Nailed Panels: AASHTO Section 3.25.2.2.

Dist. Factor/Wh. = Wp/wheel dist. width = _____

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness.

Wp = panel width

Glue Lam. Panels: AASHTO Section 3.25.3.1.

2 lanes: D.F. = $\frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$: Use Greater, D. F. = $\frac{4}{3.75 + \frac{20}{28}} = .896$ or $\frac{4}{5} = .80$ ← CRITICAL

1 lane: D.F. = $\frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$: Use Greater, D. F. = ∴ DF = .896

D.F. = Lateral Live Load distribution factor.

ALLOWABLE STRESS FOR TIMBER DESIGN -- AASHTO Section 13.6.4.1

$f_i = F_b' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$ For sawn lumber and glulam, only C_M, C_D, C_F & C_r apply.

C_M- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, C_M = **0.85** for F_b unless $F_b(C_F)$are less then 1150, then C_M = 1.0.
(For glulam, Table 13.5.3A, C_M = 0.80)

C_D-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), C_D = **1.15**.

C_F- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., C_F = **0.9.**
C_r-Table 13.5.1A. For lumber 2" to 4" thick, C_r = 1.15

As an example, if Fb = 1500 psi, then
$f_i = F'b$ = 1500(.85)(1.15)(0.9)(1.15) =**1517 psi** allowable bending stress.
And f_o = 1.33f_i = **2018 psi.**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 (allowable Shear parallel to grain).
$f_{v(inv)} = F'v = F_V C_M C_D$. If F_V = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

EXAMPLE: F_b = 1500 psi, C_M = 0.80, C_D = 1.15, C_F (for 3"x8¾") = 1.1, C_R = 1.15
∴ $f_i = F'b$ = 1500(.8)(1.15)(1.1)(1.15) = 1745 psi, f_o = 1.33×1745 = 2320 psi

CHECK SHEAR: V_{DL} (at 'd' out) = .29($\frac{20}{2}$ − .75) = 2.68 k/4' PANEL
V_{LL} (UNDIST) = 16k(17.75' + 3.75')/20' = 17.2 k, V_{LL} (DIST) = 17.2 × .896 = 15.4 k
Sect 13.6.5.2: V_{LL} = 1.5(.16×17.2 + 15.4) = 12.86 k/4' PANEL
INV. RATING = $\frac{V_{CAP} - V_{DL}(20)}{V_{LL}}$, $V_{CAP} = \frac{2}{3} F'v\,(b)(d) = \frac{2}{3} \frac{109.25}{1000}(4.5)(5.75) = 30.59$ k
∴ INV RATING (SHEAR) = $\frac{30.59 - 2.68(20)}{12.86}$ = HS 43.4 - NOT CRITICAL

DEPARTMENT OF TRANSPORTATION

LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

Rated by _____
Checked by _____
Date _____
Side 1

BRIDGE LOCATION AND DESCRIPTION

Bridge No:	Description:
Route:	
Roadway Width:	Location: _____
Year Built:	_____

Year Remodeled: _____ Slab Thickness: _____ W.C.Thickness: _____

Span Rated: _____ Span Length(L): _____ Panel Width (Wp): _____ Impact: _____

SUMMARY OF RATING AND LOAD POSTING

INVENTORY RATING	OPERATING RATING	LOAD POSTING REQ'D ?	LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED)		
			Vehicle Type M3 Weight = 24T	Semi-Trailer Comb. Type M3S2 Weight = 36T	Truck & full Trailer Type M3-3 Weight = 40T
HS_____	HS_____	YES_____ NO_____	_____Tons	_____Tons	_____Tons

RATING DATA

Dist. Factor (See Reverse Side) = D.F. = _____

		INVENTORY	OPERATING
Critical Point Location	Number		
Section Modulus (-loss)	$in.^3$		
Allowable Stress (See reverse side)	$Kips/in.^2$	fi =	fo = 1.33 fi =
Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$	Ft. Kips	①	①
Dead Load Moment/panel	Ft. Kips	②	②
Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + Impact)(Dist.\ Factor)}$	Ft. Kips	④	④
HS 20 Moment per wheel line(See Ill. #9)	Ft. Kips	⑤	⑤
HS Rating $\frac{④}{⑤}(20)$	Number	HS _____	HS _____

DEAD LOAD DESCRIPTION	DL per ft. of panel.
Overburden =	= lb./ft.
Lam. Deck (Nominal Size =)	= lb./ft.
Railing, Diaphragms, etc. =	= lb./ft.
Total Dead Load = W	= lb./ft.
Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ =	= Ft-Kips

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

Rated by
Checked by
Date:
Side 2

LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

LIVE LOAD DISTRIBUTION FACTOR:

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

Continuous Nailed Panels: AASHTO Section 3.25.2.2.

Dist. Factor/Wh. = Wp/wheel dist. width = _____

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness.

Wp = panel width

Glue Lam. Panels: AASHTO Section 3.25.3.1.

2 lanes: D.F. = $\frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$: Use Greater, D. F. =

1 lane: D.F. = $\frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$: Use Greater, D. F. =

D.F. = Lateral Live Load distribution factor.

ALLOWABLE STRESS FOR TIMBER DESIGN -- AASHTO Section 13.6.4.1

$f_i = Fb' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$ For sawn lumber and glulam, only C_M, C_D, C_F & C_r apply.

C_M- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, C_M = **0.85** for F_b unless $F_b(C_F)$ are less then 1150, then C_M = 1.0.
(For glulam, Table 13.5.3A, C_M = 0.80)

C_D-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), C_D = **1.15**.

C_F- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., C_F = **0.9.**
C_r-Table 13.5.1A. For lumber 2" to 4" thick, C_r = 1.15

As an example, if Fb = 1500 psi, then
$f_i = F'b = 1500(.85)(1.15)(0.9)(1.15)$ =**1517 psi** allowable bending stress.
And $f_o = 1.33 f_i$ = **2018 psi.**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 (allowable Shear parallel to grain).
$f_{v(inv)} = F'v = F_V C_M C_D$. If F_V = 95 psi, then $F'v = 95(1.0)(1.15)$ = **109.25 psi.**

EXAMPLE

2-20-98

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

Sheet No. 1
Rated by JWP
Checked by _____
Date 2-20-98
Side 1

RATING SHEET FOR
TRANSVERSE TIMBER PLANK DECK

BRIDGE LOCATION AND DESCRIPTION

Bridge No. 92888
Route 175
Year Built 1968
Year Remodeled _____

Long. Beam Spacing (C/C) 3.0
Transverse Plank Thickness (t) 3"
Transverse Plank Width (b) 12"

SUMMARY OF RATING AND LOAD POSTING

Inventory rating	Operating Rating	Load Posting Req'd?	LOAD POSTING LIMITS @ (Complete when load posting is required)		
			Vehicle Type M3 Weight = 24T	Semi-trailer Comb. Type M3S2 Weight = 36T	Truck & Full Trailer Type M3S3 Weight = 40T
H S 7.9	HS 10.5	✓ Yes	22 Tons	34 Tons	34 Tons
		No			

RATING FROM WORK SHEET TABLE OR CALCULATIONS BELOW

(See back side for explanation)

S = C/C - 0.23 = 2.77 (ft) No Impact

		INVENTORY	OPERATING
Allowable Stress (assumed)	kips/sq.in	f_i = ~~1.50~~ 1.64	f_o = 1.33 fi = ~~2.0~~ 2.18
Mom Avail. For LL/Plank = SM(fi or fo)/12 -.01 SM = 18.14³	Ft. Kips	2.45 ①	3.26 ①
M (HS) = 12.8 (S/4 - .2083)	Ft. Kips	6.2 ②	6.2 ②
RATING = ① (20) / ②	number	HS 7.9	HS 10.5

POSTING: M (LEGAL) = .5625M (HS) = 3.49

Posting Limits: 3.26

Vehicle (M3) = Mom Avail(Oper) (24) = 22.4 tons
M (legal) 3.49 3.26

Combination Vehicles (M3S2 or M3S3) = Mom Avail(Oper) (36.65) = 34.2 Tons
M(legal) 3.49

EXAMPLE
PLAN Fb = 1.4 ksi

TRANSVERSE TIMBER PLANK DECK Side 2

WORK SHEET EXPLANATION

S = C/C - 0.23: See AASHTO Sect. 3.25.1.2. Assume flange width = 5.5", then S = C/C - 5.5/2x12 = C/C - 0.23.

Allowable Stress: For older timber planks where timber stress is unknown, use fi = 1.5 ksi & fo = 2.0 ksi. If timber bending stress is known, see AASHTO 13.6.4.1. For 2" to 4" thick planks, Fb' = Fb,C_M,C_D,C_{fu} and if C_M =0.85, C_D = 1.15, C_{fu} = 1.2, then Fb' = Fb (1.17).

Example: 3" x 12" plank, Fb (plan) = 1.4 ksi, then fi = 1.4(1.17) = 1.64 ksi and fo = 1.33 x 1.64 = 2.18 ksi.

Moment available for LL/plank: Assume dead load moment of 0.01 ft-kip, then Mom avail. = SM(fi or fo)/12 - .01 where SM = (b)(t)²/6.

Example: 3" x 12" plank, fi = 1.5 ksi, then SM = (12 x 3 x 3)/6 = 18 in cu. and Mom avail. (Inv) = 18 x 1.5/12 - .01 = 2.24 ft kip/plank.

$M_{(HS)}$ = HS 20 live load moment/plank. For wheel load distribution to deck planks, dist. normal to traffic = 1 wheel/plank, dist. in direction of traffic = tire width (assume 20" wide tire). Therefore the moment for a uniform load spread over 20" at center of span length "S" and using a continuous factor of ".8" is: Mom = .8(P/2 x S/2 - P/2 x 5/12) = .8P(S/4 - .2083). For HS20 loading, P = 16 kips, Legal P = 9 kip (18/2).

Therefore $M_{(HS)}$ = .8 x 16(S/4 -.2083) = 12.8(S/4 - .2083) & M(legal) = 7.2(S/4 - .2083) = .5625 M_{HS}.

POSTING : M(legal) = .5625 $M_{(HS)}$. See explanation above.

TABLE OF RATING & POSTING LOADS FOR 3" x 12" TRANS. TIMBER PLANK ASSUME fi = 1.5 ksi, fo = 2.0 ksi

C/C	Spacing "S" feet	M_{HS} ft-kips	HS Inv. Rating = 2.24 (20) / M_{HS}	HS Oper Rating = 2.99 (20) / M_{HS}	M_{LEGAL} ft-kips	POSTING--TONS	
						M_3 = 2.99 (24) / M_{LEG}	M_{3S2} = 2.99(36.6) / M_{LEG}
2.0	1.77	3.0	HS 14.9	HS 19.9	1.69	LEGAL	LEGAL
2.25	2.02	3.8	11.8	15.7	2.13	LEGAL	LEGAL
2.5	2.27	4.6	9.7	13.0	2.59	LEGAL	LEGAL
2.75	2.52	5.4	8.3	11.0	3.04	LEGAL	LEGAL
→ 3.0	2.77	6.2	7.2	9.6	3.49	20	31
3.25	3.02	7.0	6.4	8.5	3.94	18	28

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STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

Sheet No. _____
Rated by _____
Checked by _____
Date _____
Side 1

RATING SHEET FOR
TRANSVERSE TIMBER PLANK DECK

BRIDGE LOCATION AND DESCRIPTION

Bridge No._____
Route _____
Year Built _____
Year Remodeled _____

Long. Beam Spacing (C/C)_____
Transverse Plank Thickness (t) _____
Transverse Plank Width (b) _____

SUMMARY OF RATING AND LOAD POSTING

Inventory rating	Operating Rating	Load Posting Req'd?	LOAD POSTING LIMITS @ (Complete when load posting is required)		
			Vehicle Type M3 Weight = 24T	Semi-trailer Comb. Type M3S2 Weight = 36T	Truck & Full Trailer Type M3S3 Weight = 40T
H S_____	HS _____	_____Yes _____No	_____Tons	_____Tons	_____Tons

RATING FROM WORK SHEET TABLE OR CALCULATIONS BELOW

(See back side for explanation)

S = C/C - 0.23 = _____(ft) No Impact

		INVENTORY	OPERATING
Allowable Stress (assumed)	kips/sq.in	f_i = 1.50	f_o = 1.33 fi = 2.0
Mom Avail. For LL/Plank = SM(fi or fo)/12 -.01	Ft. Kips	①	①
M (HS) = 12.8 (S/4 - .2083)	Ft. Kips	②	②
RATING = ① (20) / ②	number	HS	HS

POSTING: M (LEGAL) = .5625M (HS) = _____

Posting Limits:

Vehicle (M_3) = Mom Avail(Oper) (24) = _____tons
M (legal)

Combination Vehicles (M_{3S2} or M_{3S3}) = Mom Avail(Oper) (36.65) = _____Tons
M(legal)

WORK SHEET EXPLANATION

S = C/C - 0.23: See AASHTO Sect. 3.25.1.2. Assume flange width = 5.5", then S = C/C - 5.5/2x12 = C/C - 0.23.

Allowable Stress: For older timber planks where timber stress is unknown, use fi = 1.5 ksi & fo = 2.0 ksi. If timber bending stress is known, see AASHTO 13.6.4.1. For 2" to 4" thick planks, $Fb' = Fb, C_M, C_D, C_{fu}$ and if $C_M = 0.85$, $C_D = 1.15$, $C_{fu} = 1.2$, then $Fb' = Fb\ (1.17)$.
Example: 3" x 12" plank, Fb (plan) = 1.4 ksi, then fi = 1.4(1.17) = 1.64 ksi and fo = 1.33 x 1.64 = 2.18 ksi.

Moment available for LL/plank: Assume dead load moment of 0.01 ft-kip, then Mom avail. = SM(fi or fo)/12 - .01 where $SM = (b)(t)^2/6$.
Example: 3" x 12" plank, fi = 1.5 ksi, then SM = (12 x 3 x 3)/6 = 18 in cu. and Mom avail. (Inv) = 18 x 1.5/12 - .01 = 2.24 ft kip/plank.

$M_{(HS)}$ = HS 20 live load moment/plank. For wheel load distribution to deck planks, dist. normal to traffic = 1 wheel/plank, dist. in direction of traffic = tire width (assume 20" wide tire). Therefore the moment for a uniform load spread over 20" at center of span length "S" and using a continuous factor of ".8" is: Mom = .8(P/2 x S/2 - P/2 x 5/12) = .8P(S/4 - .2083). For HS20 loading, P = 16 kips, Legal P = 9 kip (18/2).
Therefore $M_{(HS)} = .8 \times 16(S/4 - .2083) = 12.8(S/4 - .2083)$
& $M(legal) = 7.2(S/4 - .2083) = .5625\ M_{HS}$.
POSTING : $M(legal) = .5625\ M_{(HS)}$. See explanation above.

TABLE OF RATING & POSTING LOADS FOR 3" x 12" TRANS. TIMBER PLANK
ASSUME fi = 1.5 ksi, fo = 2.0 ksi

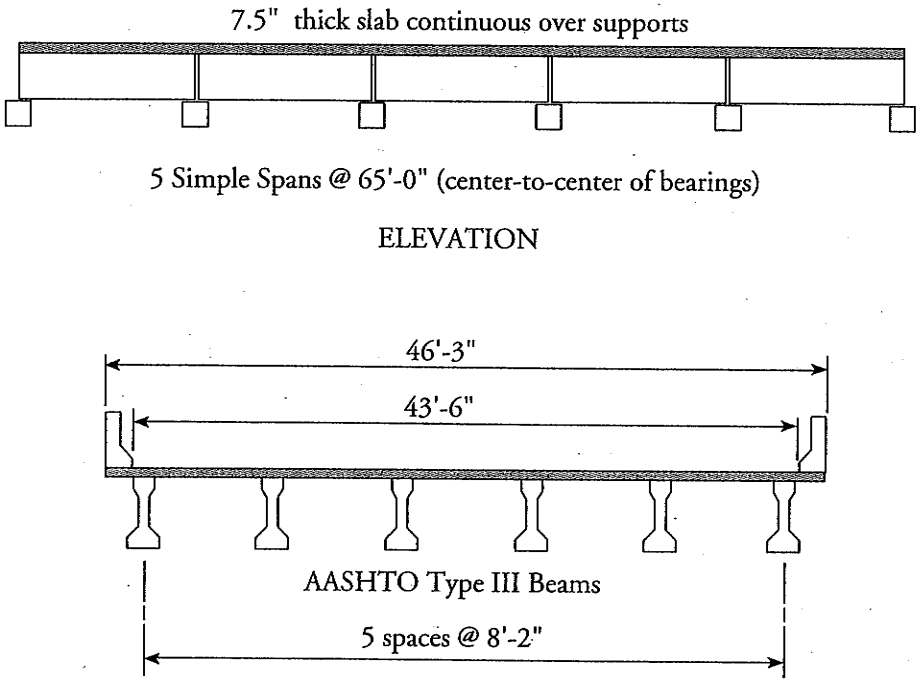
C/C	Spacing "S" feet	M_{HS} ft-kips	HS Inv. Rating $= \frac{2.24\ (20)}{M_{HS}}$	HS Oper Rating $= \frac{2.99\ (20)}{M_{HS}}$	M_{LEGAL} ft-kips	POSTING--TONS	
						$M_3 = \frac{2.99\ (24)}{M_{LEG}}$	$M_{3S2} = \frac{2.99(36.6)}{M_{LEG}}$
2.0	1.77	3.0	HS 14.9	HS 19.9	1.69	LEGAL	LEGAL
2.25	2.02	3.8	11.8	15.7	2.13	LEGAL	LEGAL
2.5	2.27	4.6	9.7	13.0	2.59	LEGAL	LEGAL
2.75	2.52	5.4	8.3	11.0	3.04	LEGAL	LEGAL
3.0	2.77	6.2	7.2	9.6	3.49	20	31
3.25	3.02	7.0	6.4	8.5	3.94	18	28

18.5 RATING EXAMPLE

18.5.1 Introduction

This two-lane bridge, built in the 1970s, is located on State Road 30 over the Carrabelle River in Franklin County, FL. It consists of five simple spans, each 65-ft. long. Each bridge span consists of 6 AASHTO Type III prestressed concrete beams spaced at 8'-2" on center. The total width of the bridge is 46'-3". The bridge has an 8-in.-thick continuous concrete deck. The top 1/2 in. of the slab is considered a wearing surface. The continuity of slab is not considered in the following calculation for simplicity. **Figure 18.5.1-1** shows the bridge elevation and the typical span cross-section. The following calculations demonstrate the rating procedure for an interior beam using AASHTO Specifications and the field test method.

Figure 18.5.1-1 Example Bridge Details



18.5.2 Materials and Conditions

Number of simple spans = 5
Number of traffic lanes = 2
Span length, L = 65 ft
Bridge width = 46'-3"
Structural slab thickness, t_s = 7.5 in.
Total slab thickness = 8 in.
Future wearing surface = 2.0 in. (25 psf)
Parapet weight = 411 plf
Specified concrete strength of beam, f'_c = 5,000 psi
Modulus of elasticity of beam concrete, E_c = 4,287 ksi
Specified Concrete strength at transfer (beam), f'_{ci} = 4,000 psi
Modulus of elasticity of beam concrete at transfer, E_{ci} = 3,834 ksi
Specified concrete strength of deck, f'_c = 3,400 psi

Modulus of elasticity of deck concrete, $E_c = 3{,}535$ ksi
Unit weight of concrete for beams and deck, $w_c = 150$ pcf
Allowable tensile stress at service (midspan) $= -6\sqrt{f'_c} = -0.424$ ksi
Prestressing strand strength, $f'_s = 270$ ksi
Modulus of elasticity of strand, $E_s = 28{,}500$ ksi
Area of 1/2 in. dia prestressing strand $= 0.153$ in.^{2}
Initial prestress, $f_{si} = 0.75f'_s = 202{,}500$ psi
Initial prestress force/strand, $P_j = (0.153)(0.75)(f'_s) = 30.98$ kips
Rating vehicle = HS20

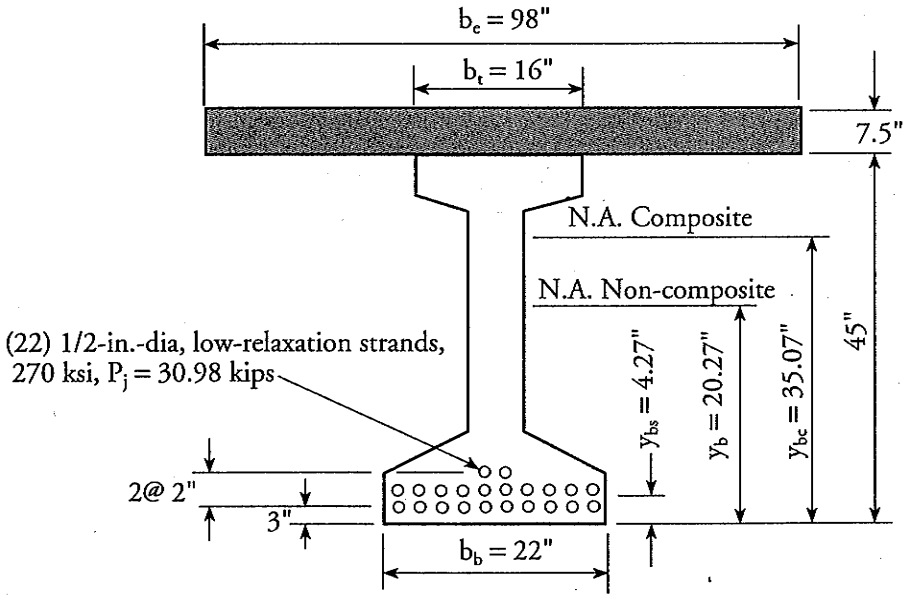
18.5.3 Section Properties

The beam properties and cross-section are shown in **Table 18.5.3-1** and **Fig. 18.5.3-1**. The section properties are calculated based on a 7.5 in. structural slab thickness, t_s. The difference in material properties between slab and beam are considered with transformed width of slab.

Table 18.5.3-1 Section Properties

Non-Composite Section	Composite Section
$y_t = 24.73$ in.	$y_{tc} = 17.42$ in.
$y_b = 20.27$ in.	$y_{bc} = 35.08$ in.
$I = 125{,}390$ in.^{4}	$I_c = 364{,}153$ in.^{4}
$A = 560$ in.^{2}	$A_c = 1{,}166$ in.^{2}

Figure 18.5.3-1 Cross-Section at Midspan



18.5.4 Load Calculations

18.5.4.1 Dead Loads

The non-composite section carries the beam self-weight and slab weight (8-in. thick), while the weights of the barrier and future wearing surface are uniformly distributed among the six beams and are carried by the composite section.

Beam moment:

$$M_g = \frac{wL^2}{8} = \frac{(560/144)(0.150)(65)^2}{(8)} = 308.07 \text{ ft-kips}$$

Slab moment:

$$M_s = \frac{wL^2}{8} = \frac{(8.17)(8/12)(0.150)(65)^2}{(8)} = 431.30 \text{ ft-kips}$$

Barrier moment:

$$M_b = \frac{wL^2}{8} = \frac{(0.411)(2)(65)^2}{(8)(6)} = 72.35 \text{ ft-kips}$$

Future wearing surface:

$$M_{ws} = \frac{wL^2}{8} = \frac{(43.5)(0.025)(65)^2}{(8)(6)} = 95.72 \text{ ft-kips}$$

Total dead load moment:

$M_d = 907.62$ ft-kips

18.5.4.2 Live Loading (HS20 Truck)

Truck loading governs for this span, so lane and alternate military loadings are not considered.

Maximum wheel-load moment:

$M_{WL\text{-}HS20} = 448$ ft-kips

Impact factor:

$$I = \frac{50}{(125+65)} = 0.26 \qquad \text{[STD Eq. 3-1]}$$

AASHTO wheel-load distribution factor:

$$WDF = \frac{S}{5.5} = \frac{8.17}{5.5} = 1.49 \qquad \text{[STD Table 3.23.1]}$$

Live load moment per beam:

$M_{LL+I} = (WDF)(M_{WL\text{-}HS20})(1 + I) = (1.49)(448)(1.26) = 841.1$ ft-kips

18.5.4.3 Prestress Losses

Initial prestress force immediately after transfer: [STD Art. 9.16.2.1.2]

$P_{si} = (22)(0.153)(0.69)(270) = 627.1$ kips

Eccentricity of prestress force:

$e = y_b - y_{bs} = 20.27 - 4.27 = 16.00$ in. [STD Art. 9.16.2.1.2]

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e^2}{I} - \frac{M_g e}{I}$$

$$= \left(\frac{627.1}{560}\right) + \left(\frac{(627.1)(16)^2}{125{,}390}\right) - \left(\frac{(308.07)16(12)}{125{,}390}\right) = +1.93 \text{ ksi}$$

$$f_{cds} = -\frac{M_s e}{I} - \frac{(M_b + M_{ws})(y_{bc} - y_{bs})}{I_c}$$

$$= -\left(\frac{(431.48)16(12)}{125{,}390}\right) - \frac{(72.35 + 95.72)(12)(35.07 - 4.27)}{364{,}324} = -0.831 \text{ ksi}$$

Elastic shortening loss:

$$ES = \frac{E_s}{E_{ci}} f_{cir} = \left(\frac{28{,}500}{3{,}834}\right)(1.93) = 14.35 \text{ ksi}$$ [STD Eq. 9-6]

Shrinkage loss (assume RH = 70%):

$SH = 17 - (0.15)(RH) = 17 - (0.15)(70) = 6.50$ ksi [STD Eq. 9-4]

Creep loss:

$CR_c = 12f_{cir} - 7f_{cds} = (12)(1.93) - (7)(0.831) = 17.34$ ksi [STD Eq. 9-9]

Relaxation loss:

$CR_s = 5 - 0.1(ES) - 0.05(SH + CR_c) = 5 - (0.1)(14.35) - (0.05)(6.50 + 17.34)$
$= 2.37$ ksi [STD Eq. 9-10]

Total prestress losses:

$SH + ES + CR_c + CR_s = 6.50 + 14.35 + 17.34 + 2.37 = 40.56$ ksi [STD Eq. 9-3]

Effective final prestress:

$f_{se} = 202.5 - 40.56 = 161.94$ ksi

Effective final prestress force:

$P_{se} = (22)(0.153)(161.94) = 545.09$ kips

18.5.5 Stresses and Strength

In a complete design process, strength checking (bending and shear) should be conducted for several sections along the span length. While a rating process should follow the same principles as design, the following calculation is limited to the bending strength at midspan and stress at the bottom of the beam, which are the locations that typically govern design.

18.5.5.1 Total Tensile Stress at Service - Inventory Rating

Dead load stress on non-composite section:

$$f_{NDL} = -\frac{(M_g + M_s)y_b}{I} = -\frac{(308.07 + 431.48)(12)(20.27)}{125{,}390} = -1.435 \text{ ksi}$$

Dead load stress on composite section:

$$f_{CDL} = -\frac{(M_b + M_{ws})y_{bc}}{I_c} = -\frac{(72.35 + 95.72)(12)(35.07)}{364{,}324} = -0.194 \text{ ksi}$$

Live load stress on composite section:

$$f_{LL+I} = -\frac{(M_{LL+I})y_{bc}}{I_c} = -\frac{(841.1)(12)(35.07)}{364{,}324} = -0.972 \text{ ksi}$$

Stress from prestress force:

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}ey_b}{I} = \frac{545.09}{560} + \frac{(545.09)(16)(20.27)}{125{,}390}$$

$$= +2.383 \text{ ksi (compression)}$$

Total tensile stress at service:

$$f_{total} = f_{pe} - (f_{NDL} + f_{CDL}) - f_{LL+I}$$
$$= 2.383 - (1.435 + 0.194) - 0.972 = -0.218 \text{ ksi},$$
$$f_{allow} = -0.424 \text{ ksi}$$
$$R.F._{IN} = f_{allow}/f_{total} = 0.424/0.218 = 1.94$$

18.5.5.2 Design Flexural Strength - Operating Rating

$$M_u = 1.3(M_d + 1.67M_{LL+I}) = 1.3[(907.62) + 1.67(841.1)] = 3{,}005.93 \text{ ft-kips}$$
$$d = 45 + 7.5 - 4.27 = 48.23 \text{ in.}$$

$$f^*_{su} = f'_s\left[1 - \frac{\gamma^*}{\beta_1}\left(\frac{p^*f'_s}{f'_c}\right)\right]$$

$$= 270\left[1 - \left(\frac{0.28}{0.85}\right)\frac{(0.153)(22)}{(98)(48.23)}\left(\frac{270}{3.4}\right)\right] = 265.0 \text{ ksi}$$

$$a = \frac{A^*_s f^*_{su}}{0.85bf'_c} = \frac{(265.0)(22)(0.153)}{(0.85)(98)(3.4)} = 3.15 \text{ in.}$$

$$M_n = A^*_s f^*_{su}\left(d - \frac{a}{2}\right) = (22)(0.153)(265.0)\left(48.23 - \frac{3.15}{2}\right)\left(\frac{1}{12}\right) = 3{,}468.0 \text{ ft-kips}$$

$$R.F._{OP} = \frac{M_n}{M_u} = \frac{3{,}468.0}{3{,}005.93} = 1.15$$

18.5.5.3 Rating Factor

The above calculations show that the design is governed by the lower rating factor for the design flexural strength ($R.F._{OP} = 1.15$). Therefore, the load rating for the bridge is equal to HS(20 x 1.15) = HS23.

18.5.6 Evaluation Guide Specification Rating

Based on values obtained using the *Standard Specifications* in the previous section, the following ratings can be obtained from the *Evaluation Guide Specifications.*

Operating and inventory ratings using the factored load method:

$$R.F._{OP} = \frac{\phi M_n - 1.3M_d}{1.3\ M_{LL+I}} = \frac{(1.0)(3{,}468.0) - (1.3)(907.62)}{(1.3)(841.1)} = 2.09$$

$$R.F._{IN} = \frac{\phi M_n - 1.3M_d}{1.3(1.67)M_{LL+I}} = \frac{(1.0)(3{,}468.0) - (1.3)(907.62)}{(1.3)(1.67)(841.1)} = 1.25$$

Inventory rating using the allowable stress method:

$$R.F._{IN} = \frac{f_{pe} - f_{DL} - f_{allow}}{f_{LL+I}} = \frac{2.383 - (1.435 + 0.194) - (-0.424)}{0.972} = 1.21$$

The inventory load rating is controlled by the service (allowable stress) requirement. Therefore, the inventory rating is equal to 1.21. The inventory rating with the factored load method is 1.25, which shows that the use of this method may result in a higher rating. The final rating factor for prestressed concrete structures should be the lesser of the values obtained by the allowable stress and factored load methods to ensure adherence to the original design assumptions.

18.5.7 Rating using the LRFD Specifications

As noted in Section 18.1, at present there are no established guidelines for load rating using the *LRFD Specifications*. The following rating is based on the guidelines given in the *Standard Specifications* and is intended to illustrate the difference between the two design specifications.

18.5.7.1 Dead Load

The dead loads are essentially the same as previously calculated.

18.5.7.2 Prestress Loss

The prestress loss calculation in the *LRFD Specifications* is almost identical to that of the *Standard Specifications* except for minor changes to the relaxation calculation. The total loss calculated according to the *LRFD Specifications* is 41.62 ksi compared to 40.56 ksi using the *Standard Specifications*. Since these values are fairly close, the effective prestress will be taken as the same value above, computed using the *Standard Specifications*, i.e., 161.94 ksi.

18.5.7.3 Live Load

Rating live load: HL-93

Maximum truck moment per lane: $M_{lane\text{-}HS20(truck)} = 896.0$ ft-kips

Maximum lane moment per lane: $M_{lane\text{-}HS20(lane)} = 338.0$ ft-kips

Maximum tandem moment per lane: $M_{lane\text{-}HS20(tandem)} = 762.5$ ft-kips

Dynamic load allowance: IM = 0.33

AASHTO lane-load distribution factor for cross-section "type k" with two or more lanes loaded:

$$LDF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0L\ t_s^3}\right)^{0.1}$$ [LRFD Table 4.6.2.2.2b-1]

where

$K_g = n(I + Ae_g^2)$ [LRFD Eq. 4.6.2.2.1-1]

$$= \frac{4,287}{3,535}\left(125,390 + 560\left(45 + 7.5 - 20.27 - \frac{7.5}{2}\right)^2\right) = 702,913 \text{ in.}^4$$

$$\text{LDF} = 0.075 + \left(\frac{8.17}{9.5}\right)^{0.6}\left(\frac{8.17}{65}\right)^{0.2}\left(\frac{702,913}{(12.0)(65)(7.5)^3}\right)^{0.1} = 0.726$$

The live load moment is specified as the lane load moment plus the larger of the truck or the tandem moment:

$M_{LL+I} = \text{LDF}[M_{\text{lane–HS20(lane)}} + \max(M_{\text{lane–HS20(truck)}}, M_{\text{lane–HS20(tandem)}})(1 + \text{IM})]$

$= (0.726)[338.0 + 896.0(1.33)] = 1,110.5$ ft-kips

18.5.7.4 Live Load Stress

$$f_{LL+I} = \frac{(M_{LL+I})y_{bc}}{I_c} = \frac{(1,110.5)(12)(35.07)}{364,324} = 1.280 \text{ ksi}$$

It can be seen that the live load stress calculated by the *LRFD Specifications* is much higher than that calculated by the *Standard Specifications* (1.280 ksi versus 0.972 ksi).

18.5.7.5 Strength Calculation

Because the strength calculation in the *LRFD Specifications* is similar to that in the *Standard Specifications*, the detailed calculations are not presented here. For a presentation of the details, consult Section 8.2. To review a sample calculation, refer to Design Example 9.4.

$$M_n = A_{ps}f_{ps}\left(d - \frac{a}{2}\right) = (22)(0.153)(264.21)\left(48.23 - \frac{3.14}{2}\right)\left(\frac{1}{12}\right) = 3,458.0 \text{ ft-kips}$$

This is very close to the value obtained using the *Standard Specifications* (3,468.0 ft-kips). The minor variation in the nominal moment capacity is due to the change in the calculation for "f_{ps}" and sometimes for the calculation for "a" in the *LRFD Specifications*.

18.5.7.6 Inventory Rating

The inventory rating should utilize the same principles that were used in the design. Therefore, the inventory rating, according to Strength I, should be:

$$R.F._{IN} = \frac{\phi M_n - 1.25(M_g + M_s + M_b) - 1.5M_{ws}}{1.75M_{LL+I}}$$

$$= \frac{(1.0)(3,458.0) - (1.25)(308.07 + 431.48 + 72.35) - (1.5)(95.72)}{(1.75)(1,110.5)} = 1.18$$

Although the live load moment by the *LRFD Specifications* analysis is 32 percent larger than that by the *Standard Specifications* (1,110.5 ft-kips versus 841.1 ft-kips), the load factor for live load is considerably less for the *LRFD Specifications* analysis (1.75 versus 2.17). Thus the inventory rating for *LRFD Specifications* Strength I is only 6 percent less than that for the *Standard Specifications* (1.18 versus 1.25).

Inventory rating with *LRFD Specifications* Service III (full dead load plus 80 percent live load):

Allowable tensile stress:

$$R.F._{IN} = \frac{f_{pe} - f_{DL} - f_{allow}}{0.8 f_{LL+I}} = \frac{2.383 - (1.435 + 0.194) - (-0.424)}{(0.8)(1.28)} = 1.15$$

Theoretically, the rating should also be calculated by checking compressive stress. This is not presented here because it is not critical in this example. The final rating should be the lesser of the service and strength check.

The procedures for determining operating rating with the *LRFD Specifications* have not yet been adopted. Therefore, it is recommended that the operating rating be taken as 1.67 times the inventory rating, the same factor used in the *Standard Specifications.*

18.5.8 Rating by Load Testing

18.5.8.1 Test Information

Target inventory rating: HS20

Maximum test load = two test trucks, each with a gross weight of 207 kips

Maximum moment per test truck per lane = 1,868 ft-kips

Solving for the live load stress that will cause the bottom fiber stress to reach the allowable stress:

$f_{LL} = f_{pe} - f_{DL} - f_{allow} = 2.383 - (1.435 + 0.194) - (-0.424) = 1.178$ ksi

The stress, f_{LL}, is theoretically the maximum stress that the bridge can be subjected to without cracking. Since stress cannot be measured directly, the strain corresponding to the stress, f_{LL}, is computed as: $\varepsilon_{LL} = f_{LL}/E_c = 1.178/4{,}287 = 0.000275 = 275 \times 10^{-6}$. This strain value (275 microstrains) is considered the limiting test strain to avoid any permanent damage to the bridge during testing.

18.5.8.2 Test Results

In this example, the bridge was loaded incrementally using the two test trucks and the strains were monitored at all critical locations. The maximum measured strain at the bottom of the beam under the two test vehicles was 182 microstrains, which is 66 percent of the calculated strain limit, ε_{LL}. The bridge showed no signs of distress or cracking at any load level and load-strain relationships were linear throughout the test.

18.5.8.3 Distribution Factor

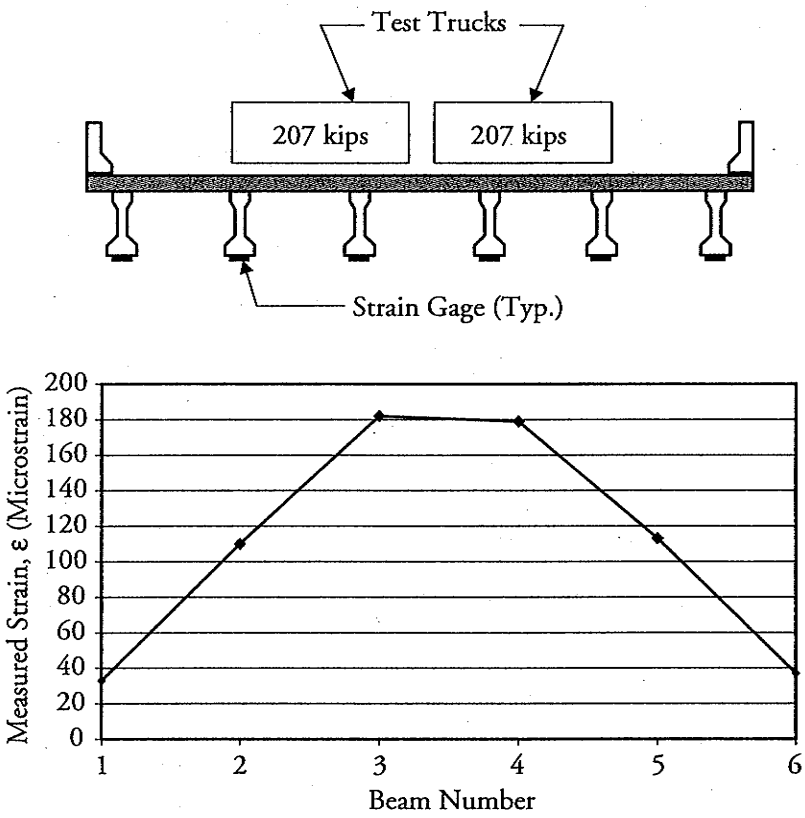
The strain measurements across the bridge under maximum applied live loads are shown in **Fig. 18.5.8.3-1.**

Calculate the measured wheel load distribution factor, WDF:

$$WDF = \frac{N_w \varepsilon_{max}}{\Sigma \varepsilon_i} = \frac{(4)(182)}{33 + 110 + 182 + 179 + 113 + 37} = 1.11$$

where N_w is the number of lines of wheel loads on the tested bridge.

Fig. 18.5.8.3-1
Strain Distribution in Bottom Flanges across Bridge



For comparison, the wheel load distribution factor from the *Standard Specification* is 1.49. The LRFD lane distribution factor calculated in Section 18.5.7.3 must be multiplied by 2 for comparison, which is (2)(0.726) = 1.45. Therefore, the distribution factor determined by the load test is much lower than the values computed by the two specifications.

Because the test vehicles are different from the HS20 truck, the stress from the test for an equivalent HS20 truck plus impact can be calculated from the ratio of test truck moment and HS20 moment as:

$$f_{HS20+I} = \left(\frac{(1+I)M_{WL-HS20}}{M_{test}}\right)\left(-\varepsilon_{measured}\right)\left(E_c\right) = \left(\frac{(2)(1.26)(448)}{1{,}868}\right)\left(-182\times10^{-6}\right)(4{,}287)$$
$$= -0.471 \text{ ksi}$$

where the factor 2 equates the maximum wheel load moment for an HS20 truck to two test trucks.

Because the applied test load moment per lane (1,868 ft-kips/lane) is less than the ultimate design live load moment per lane [2.17(2)(448)(1.26) = 2,450 ft-kips] the load test is considered diagnostic.

18.5.8.4 Test Inventory Rating Factor

The inventory load rating based on the test measurements is shown below.

$$R.F._{IN} = \frac{f_{pe} - f_{DL} - f_{allow}}{f_{HS20+I}} = \frac{2.383 - (1.435 + 0.194) - (-0.424)}{0.471} = 2.50$$

The above test inventory rating, based on test measurements, is more than twice the theoretical AASHTO allowable stress inventory rating obtained in Section 18.5.6. The computed maximum tensile stress from load testing, 0.471 ksi, is significantly less than the theoretically calculated value, 0.972 ksi. This is due to many beneficial factors that are ignored in a theoretical load rating. These factors include:

- slab continuity
- diaphragms
- parapet composite action
- bearing restraint effects
- lower than expected prestress losses
- higher concrete strength
- higher concrete modulus of elasticity

In addition, the AASHTO load distribution factors are generally very conservative resulting in the design of stronger elements than required by actual loading.

18.5.8.5 Test Operating Rating Factor

Maximum live load moment from test for equivalent HS20 plus impact:

$$M_{LL+I} = \left(\frac{I_c}{y_{bc}}\right) f_{HS20+I} = \left(\frac{364,324}{35.07}\right)\left(\frac{0.471}{12}\right) = 407.7 \text{ ft-kips}$$

Operating rating according to the *Standard Specifications*, Load Factor Method:

$$R.F._{OP} = \frac{\phi M_n - 1.3M_d}{1.3M_{LL+I}} = \frac{1.0(3,458.0) - 1.3(907.62)}{1.3(407.7)} = 4.30$$

The inventory rating factor (2.50) and operating rating factor (4.30) above are considered upper bounds due to the nature of diagnostic/linear analysis. Therefore, the final rating should be limited to the original design, i.e., inventory rating of HS20, or operating rating of HS(20 x 1.67) = HS33.

18.6 REFERENCES

Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, American Association of State Highway and Transportation Officials, Washington, DC, 1989

Manual for Condition Evaluation of Bridges, American Association of State Highway and Transportation Officials, Washington, D.C, 1994

AASHTO LRFD Bridge Design Specifications, Second Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1998

Standard Specification for Highway Bridges, 16th Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1996

2008
Bridge Load Rating
Class 101

Workshop Agenda

VIII. Load Rating Example #2
- Simple Span Nail Laminated Timber Deck (without distress)

IX. Class Exercise
- Simple Span Nail Laminated Timber Deck (with distress)

X. Review of worked out examples

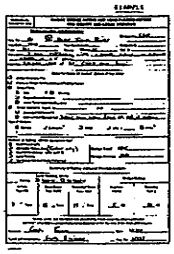
XI. Submittal to MN/DOT and Review of Process

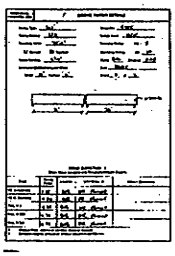
XII. Common mistakes and questions

Submittal and Review Procedure

- We have our rating number, now what?
  - Submitting results to MnDOT
- Quick review of previous material

Submitting results to MnDOT



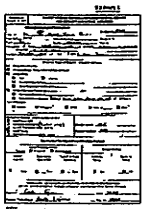


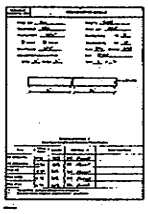
Bridge Management
Attn Jim Pierce
3485 Hadley Ave No
Oakdale MN 55128

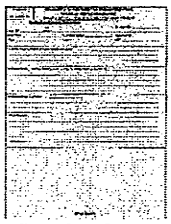
http://www.dot.state.mn.us/bridge/

Submitting results to MnDOT

- If ratings based on engineer assessment include the PIR form with explanation. Describe condition and rationale for estimate.



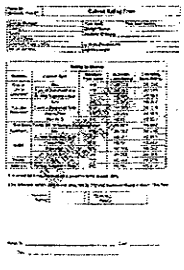




http://www.dot.state.mn.us/bridge/

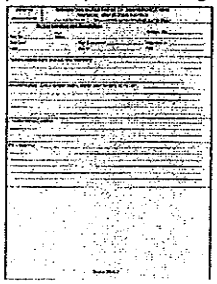
Culverts

Culvert Rating Form



NBI<5 then

Physical Inspection Rating



http://www.dot.state.mn.us/bridge/

Review of Procedure

1. Gather bridge data
2. Do bridge inspection
3. Calculate section properties
   - Incorporate distress
4. Calculate dead loads
5. Calculate live loads (HS20, Posting, Permit)
6. Calculate Inv., Op. and Posting numbers
7. Complete rating forms and send in

2008
Bridge Load Rating
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Workshop Agenda

VIII. Load Rating Example #2
- Simple Span Nail Laminated Timber Deck (without distress)

IX. Class Exercise
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X. Review of worked out examples

XI. Submittal to MN/DOT and Review of Process

XII. Common mistakes and questions

Common Mistakes / Questions

- Live Loads
  - 2 x Wheel loads = lane loads
- Make sure posting sign is out on bridge
  - Same from both directions
- Lack of Enforcement
  - May change posting/closing decision

Common Mistakes / Questions

- For shorter bridges, semi can't fit on bridge. Why do you post for semi then?
  - This is accounted for in calculations. The posting weight is ratioed to axles that can be on bridge
- Different code different answer
  - ASD could be higher for steel
  - LRFR generally lower
- What code is required
  - LRFR for new bridges after 2010
  - LFR acceptable beyond 2010 for existing bridges

Common Mistakes / Questions

- Substructure rating is very subjective
  - There is research into rules of thumb from different states. That should be coming next year.
- Culverts with > 4' fill
  - Major load is not from live load.

Common Mistakes / Questions

- Truckers perspective: Bridge doesn't fall down when I drive a illegal heavy truck over it so it must be OK
  - There are conservatisms in assumptions and calculations. You are reducing your safety factor. There also could be unknown section losses.
  - Reduces service life of bridge

We would like to extend our thanks to:

- Gary Peterson
  - State Construction and Maintenance Bridge Engineer
- Ed Lutgen
  - Bridge Rating Engineer/North Region Bridge Construction Engineer
- Dave Conkel
  - State Aid Bridge Engineer
- Brian Homan
  - Asst State Aid Bridge Engineer (extraordinaire)
- Rick Kjonnes
  - State Aid

Thank you for coming to the:

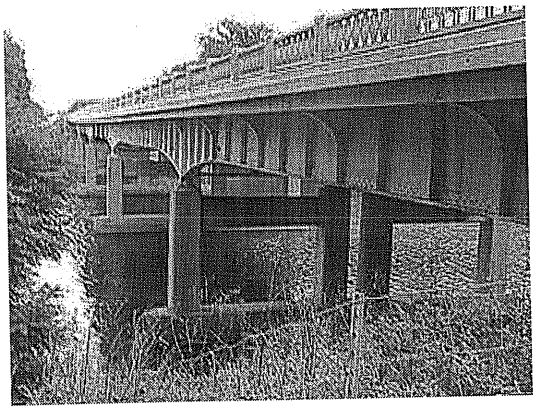
2008 Bridge Load Rating Class 101

Questions/Comments?

Additional Considerations

- Structural steel
- Reinforced concrete
- Concrete culverts
- Prestressed concrete
- Timber
- Truss bridges

Structural Steel



Special Considerations

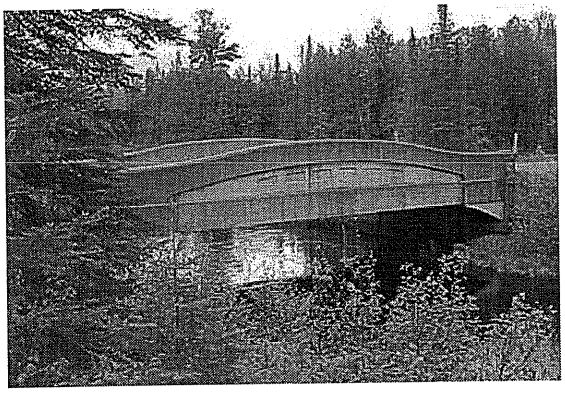
- Use the following table for yield strength:

Year of Construction	Yield Point F_y (ksi)
Before 1905	26
1905-1936	30
1937-1963	33
1964-1989	36
1989-present	50

Special Considerations (Cont.)

- For beams consider overload requirements of AASHTO Stnd. Spec. 10.57 (Serviceability)
- Can use plastic capacity per Std Spec. 10.50 (Z modulus)
- Operating rating of welds, bolts, and rivets use max strength from Table 10.56A AASHTO Stnd. Spec.
- For friction joint fasteners (A325 bolts) use a stress of 21 ksi and 1.0 for A_1 & A_2

Reinforced Concrete



Special Considerations

- Use the following table for reinforcement yield strength:

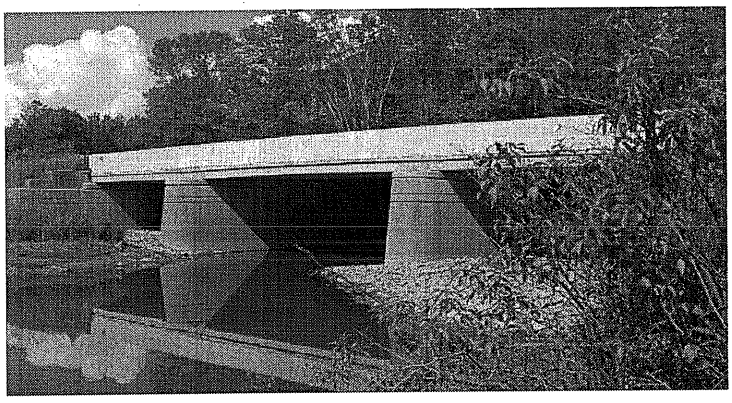
Reinforcing steel	Yield Point F_y (ksi)
Prior to 1940	30
Unknown Steel (prior to 1954)	33
Structural Grade	36
Billet or Intermediate Grade and unknown after 1954 (Grade 40)	40
Rail or Hard Grade (Grade 50)	50
Grade 60	60

Ref: Manual for Condition Evaluation of Bridges
Note some older bridges used square bars not round.

Culvert Considerations

- Culverts > 5' fill have significantly lower live load component
  - By reducing rating/posting the safety not increased
- Watch for distress that could cause failure
  - Settlement
  - Soil pressures
- Hydraulic considerations
  - Higher Velocities
  - Debris buildup
  - Alternate scour paths

Prestressed Concrete



Prestressed Concrete

- Inventory Level (HS-20 Truck)
  - Concrete Tension
    - Almost always controls center beam
  - Concrete Compression (2 equations)
  - Prestress Steel Tension
  - Flexural Strength
  - Shear Strength

Prestressed Concrete

- Operating Level (HS-20 Truck)
  - Flexural Strength
  - Shear Strength (most likely control)
  - Prestressing Steel Tension

Prestressed Concrete

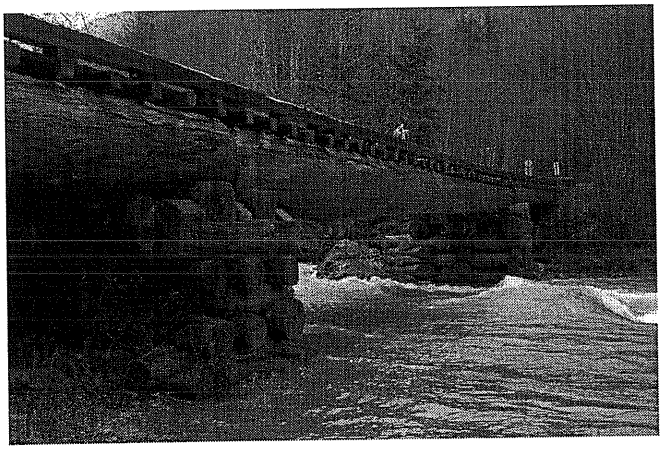
- **Inventory Level (HS-20 Truck)**

$$RF = \frac{6\sqrt{f'_c} - (F_d + F_p + F_s)}{F_l}$$ Concrete Tension

- **Operating Level (HS-20 Truck)**

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

Timber Structures



Timber Structures

- Conduct load rating using ASR or LRFR methods
- ASR
  - Inventory unit stress = allowable stress for stress-grade lumber given in AASHTO Standard Specs.
  - Operating unit stress = 1.33 times allowable stress for stress-grade lumber, then reduced for grade and condition based on inspection

Timber Superstructures

- During inspection:
  - All corrosion/section loss should be recorded and subtracted from the gross cross-section area.
  - Tension Members
    - Find size, number, and location of all bolts/rivets (to determine net area)
    - Check condition of threaded rods at turnbuckles
  - Compression Members
    - Check for misalignments, bends, or kinks
    - Check connections for eccentricities

Timber Decks

- Timber planking
  - Check for moment and shear
- Timber slabs
  - Check spreader beam
- See example calculations provided

Abutment Timber Piling Bent

- Section loss at groundline
- In addition to supporting dead and live loads, abutment piling brace approach backfill.
  - Behavior similar to beam column
- See example calculations provided

Pier Timber Piling Bent

- Section loss at waterline
- Estimate unbraced length
- Pier cap can be critical to carry sufficient load distribution
- Longitudinal and transverse loads (potential for ice impact, wind)
- See example calculations provided

When to Hire a Consultant

- Continuous spans
  - Moment distribution is a function of I
- Excessive distress
  - Possible repair plan
- Complex structures
  - Arches, trusses
- Beyond the comfort level of signing engineer

BRIDGE RATINGS

INTRODUCTION

Bridge rating and load posting analysis (hereafter denoted by one word, rating) is administered and performed by the Bridge Rating Section of the MnDOT Bridge Office. Bridge ratings may also be done by other qualified engineers.

Definition, Rating: The determination of the live load carrying capacity of a new or existing bridge using bridge plans, supplemented by information gathered from a field inspection.

Bridge Ratings are calculated in accordance with the AASHTO Manual for Condition Evaluation of Bridges (MCE). This manual refers the user to the AASHTO Standard Specifications for Highway Bridges for much of the needed information.

FORMS AND DOCUMENTATION

The rating forms can be found on the Bridge Office Web Site at Documents, Downloads, Forms, and Links, in the section titled Bridge Rating and Load Posting Report for [County and Local Agencies].

Fill in the blanks that are applicable to the bridge and the type of rating you are doing. A minimum of two pages are required to document a rating, but additional sheets may be attached if needed. The cover page of the completed rating must be signed by a registered professional engineer.

SUBMITTAL / FILING

The original copy of the rating should be retained in the files of the bridge owner.

For bridges owned by counties and local units, copies of the ratings should be submitted to MnDOT, and will be kept on file by the Bridge Management Unit.

Copies of ratings for Trunk Highway bridges are kept in the files of the Bridge Ratings Unit.

GENERAL

Bridges are rated at two different stress levels, Inventory level and Operating level. The Operating level is used for load posting and for overweight permit checks.

Ratings are calculated and reported in terms of HS - 20. Thus with the HS - 20 truck as the live load in the denominator of the rating equation (AASHTO MCE 9-1a), and if the resulting rating factor is 1.15, then the rating would be reported as HS - 23.

Use a phi factor of 0.91 for prestressed concrete flexure in load factor rating.

All bridges are rated except those that carry pedestrians, recreational traffic, or railroad trains.

Box culverts and other culverts with spans of 10 to 20 feet are rated, but not by a calculated value, They are assigned rating values based on their design load or a physical inspection.

MnDOT rates the bridges on the state system (Interstate, US, and Minnesota). Counties, cities, etc. each rate their own bridges.

Ratings are performed by the Load Factor Method whenever possible. The Allowable Stress Rating and the Load and Resistance Factor Rating methods will sometimes be accepted alternatives.

The use of computer programs is preferred for rating. BARS has been used since about 1973. It is no longer supported by the developer and is being phased out. Virtis is the new program which is replacing BARS.

Bridges entered in Virtis should be in the Girder System Definition whenever the bridge geometric requirements will fit within the limitations of Virtis.

When using the Girder Line Superstructure Definition, rate an interior beam under a vehicle traffic lane.

In almost all cases only the primary load carrying members of the superstructure are rated. Decks or piers may have to be investigated in unusual circumstances such as severe damage or deterioration. Unusually heavy permit loads may also require investigation of the deck and piers.

Temporary bridges are rated, the same as permanent bridges. Also, overweight permit restrictions are regulated in the same manner as permanent bridges.

LOADS

Dead loads are calculated according to AASHTO.

Low slump concrete wearing courses and latex modified wearing courses are considered to be fully composite with the base slab. This might change for an individual bridge if an inspection showed that the bond between them was failing.

The top 0.5 inch of the wearing course or slab is not considered to be effective for composite action or section properties.

Composite section when deck is poured in two steps. This is usually 7 inches base, then a 2 inch low slump wearing course.

DL1 Is defined as noncomposite dead load (stage 1) and DL2 as composite dead load (stage 2). DL1 includes the weight of the beam, diaphragms, and the initial slab pour . The remainder of the dead loads are then in DL2. MnDOT considers the effective composite deck supporting DL2 to be the initial slab pour. The effective composite deck supporting the live load (stage 3) is the full deck including the wearing course minus 0.5 inch. Most computer programs including BARS and Virtis will not accept these two different thicknesses of composite deck for stages 2 and 3. It is then necessary to use the final composite deck thickness as the one that also supports DL2.

Include a dead load for utilities of 2 psf of deck area in rural areas and 3 psf in cities and urban areas. Higher loads may be required if heavier utilities are shown on the plan or have been placed on the bridge.

Use a stool height of 1.5 inches for bridges designed after 1990 and 1 inch for before. Add as uniform dead load an additional weight to account for additional stool, residual camber, slope of the deck, and superelevation.

Distribution of DL2: Railings, sidewalks, utilities and medians may be divided uniformly among all beams if they are located symmetrically on the cross section. Otherwise a different distribution method should be used which is logically sound.

Distribute live loads according to AASHTO. Standard gage width is 6 feet. For overweight permits treat gage widths of up to 7 feet as 6 feet. For gage widths wider than this, an adjustment may be made.

If the design includes an allowance for future dead loads, such as wearing course, these should not be included when calculating ratings.

RATING NEW BRIDGES

New bridges are to be rated anytime after the plan is completed and up to two weeks before the bridge is opened to traffic. The results are then turned in to the Bridge Management Unit for entering in Pontis.

If any changes are made to the bridge during construction that would affect the rating, these changes should be reported to the Bridge Ratings Unit (or the person who did the original rating), and also be recorded on the as built plans, The bridge rating will then be recalculated.

RERATING EXISTING BRIDGES

A new bridge rating should be completed whenever a change occurs that would affect the rating. The most commonly encountered types of changes are:

- a modification that changes the dead load on the bridge (such as an overlay)
- damage or deterioration that alters the structural capacity of the bridge. (Scheduled inspections are usually the source of this information.)
- repairs or remodeling.
- A change in the AASHTO Rating Specification
- An upgrading of the rating software.
- A change in laws regulating truck weights

TIMBER BRIDGES

For decks, use all the provisions of AASHTO Fig. 3.7.7.4 with applicable footnotes. In other words, rate decks with individual axles of 16 k or 17 k or whatever the posting truck has.

Use wet condition for all rated timber members.

The repetitive use factor, C_r, can be used for plank decks, if they are covered by bituminous or perpendicular planks for load distribution. C_r may also be used for laminated decks providing the panels do not show any separation or loss if lamination.

If timber members are in a deteriorated condition, their reduced capacity may be accounted for by either reducing the allowable stress or the section modulus.

When the original plan cannot be found, and the original design stresses cannot be determined from any other source, the following may be used for f_b:

Timber planks	1.5 ksi
Timber beams	1.6 ksi
Transverse Glu-Laminated Decks *	1.5 ksi
Transverse Nail-Laminated Decks **	1.35 ksi
Longitudinal Glu-Laminated Decks ***	1.4 ksi
Longitudinal Nail-Laminated Decks ****	1.2 ksi
Glu-Laminated Beams	2.0 ksi

* Commonly used for decks on temporary bridges. Usual depth = 5 1/8 in.
** Seldom used
*** Used less often
**** Commonly used for county and local bridges.

Assumed species: Douglas Fir-Larch

For the other stress categories such as shear, bearing, etc, refer to the AASHTO tables on the line corresponding to the f_b given above.

These stresses are for the inventory level. Increase them by 33 % for the operating level.

PHYSICAL INSPECTION RATING (PIR)

This method of rating is to be used when there is no bridge plan available and/or the capacity cannot be calculated. Follow AASHTO MCE 7.4.1

The rating is determined by the engineer upon careful consideration of all available information, including bridge condition (corrosion, spalling, damage, distress, cracking, etc.), age, type of construction, redundancy, traffic count (ADT or ADTT), loading (historic to present), etc.

The numbers in the rating should (approximately) follow this ratio: (1.67 x Inventory Rating) = (Operating Rating) = (the posting weight in Tons for the single truck) = (0.625 x the posting weight in Tons for a combination truck).

When a bridge is rated by PIR, it should be posted, at no higher than 26 T, 40 T, 40 T, unless it has a history of satisfactory performance at higher loadings. If not posted, overweight permits should be restricted.

A PIR rating is documented with Form PIR and accompanied by the cover form, RC - TH or RC - CL

The inspection interval should be the normal time, based on bridge condition.

The rating interval should be after each inspection.

FORM PIR Revised Mar 06	MINNESOTA DEPARTMENT OF TRANSPORTATION PHYSICAL INSPECTION RATING (Per AASHTO 7.4.1 - Manual for Condition Evaluation of Bridges)

Bridge Description

Hwy. No. _____ over / under _____ Bridge No. _____

Year Built _____ Year Remodeled _____ Replaces Br. _____

Type _____ County _____ ADT _____

Problem leading to this physical inspection rating: _____

Describe bridge; Spans, lengths, widths, depths, deck, wearing course, etc. _____

Describe Bridge Condition: _____

Other Remarks: _____

Bridge Sketch

Form PW Apr 06	Posting Worksheet	

Bridge No	
Rated by	
Date	

Truck	Wgt, TON	Rating Factor	Location	Limit State	Notes/Comments
T M 3	24				
T M 3S2-40	40				
T M 3S3	40				
T M 3S3 V	40				
SU4	27				
SU5	31				
SU6	34.75				
SU7	38.75				
NRL	40				

FORM RD–CL Revised Feb. 2005	BRIDGE RATING DETAILS

Bridge Type _____ Bridge No. _____

Rating Method _____ Design Load: _____

Roadway Width _____ Inventory Rating: HS - _____

☐ Curved ☐ Tapered Operating Rating: HS - _____

Beam Spacing _____ Rated _____ Checked _____

Live Load Distribution Lane Width Date _____

Single _____ Multiple _____ Sheet _____ of _____

BEAM ELEVATION ②

Show Span Lengths and Structure / Beam Depths

Truck	Rating Factor	Location	Limit State ①	Notes / Comments
HS 20 Inventory				
HS 20 Operating				
Post, M 3				
Post, M 3S2				
Post, M 3-3				

① Choose from: service or ultimate; shear or moment.
② Elevation may be on back or another sheet if it won't fit here.

FORM RC-CL Revised Mar. 2005	Mn/DOT BRIDGE RATING AND LOAD POSTING REPORT FOR COUNTY AND LOCAL AGENCIES

Bridge Location and Description

Bridge No. _____

Hwy. No. _____ over / under _____

Year Built _____ Year Remodeled _____ Replaces Br. _____

Type _____ County _____ Ref. Pt. _____

Description _____

_____

Location _____

Data for Basis of Report (Check all that apply)

- ☐ Bridge Inventory File
- ☐ Previous Bridge Rating and Load Posting Report
- ☐ Bridge Plans
  - ☐ New ☐ Overlay
  - ☐ Repair/Reconstruction _____
  - ☐ Other Dead Load Modifications _____
- ☐ Bridge Inspection Report, by _____ ,date _____
  - ☐ Damaged Component _____
  - ☐ Deteriorated Component _____

Type of Analysis:

☐ Manual ☐ Computer* ☐ BARS ☐ Virtis, V._____ ☐ Other*

* _____

Method of Rating (Check appropriate box)

- ☐ Load Factor (LF)
- ☐ Allowable Stress (AS)
- ☐ Load & Resistance Factor (LRFR)
- ☐ Load Testing
- ☐ No Rating Computations Performed

Design Load _____

Design Method _____

Summary of Rating and Load Posting Analysis

Load Posting Limits Posting: ☐ Required ☐ Not Required			Bridge Rating	
Vehicle Type M3	Semi-Trailer Type M3S2	Truck-Full Trailer Type M3-3	Inventory Rating	Operating Rating
_____ Tons	_____ Tons	_____ Tons	HS _____	HS _____

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

Signature: _____ Date: _____

Typed or Printed Name: _____ Reg. No. _____

VIRTIS GUIDELINES

Resources for Rating Information

Original plan ----- Repair plan ------ As built plan -----

Existing rating ------ Bridge Inventory data ------ Inspection reports

Bridge Description

BID	Assigned by Virtis
Bridge I D	Bridge number (usually 5 digits).
NBI Structure I D	Same as Bridge I D
Bridge Completely Defined	Check this box to indicate whether or not the bridge superstructure is completely defined in this Virtis file.
Name (Bridge) *	Enter the three digit bridge type. (See Bridge LRFD Design Manual, 2 – A.) If there is more than one type in the bridge, enter the type of the main (or lowest rated ?) span and add the suffix, m (for multiple).
Year built	Year of construction
Description	*[Select those that apply from this list, add others if needed.]* This Bridge was copied from it's twin, Br xxxxx Twin to Br xxxxx Number of spans. Other bridge types if multiple types in one bridge. Note if different ratings apply to each side of bridge or direction of traffic or if bridge has one way traffic. Flare or curve, note if present. Skew angle. Describe dead load distribution: Sidewalks, medians, rails, etc Describe in detail if for any reason the computer model deviates from the plan. Does / does not have end blocks.[For prestressed beam bridges] Replaces Bridge xxxx. Initials of person rating and date completed.
Location	City name, or (for example): Baudette, SE 23 mi.
Length	xxxx feet

Facility carried	Facility carried **over** the bridge. Use the highway abbreviation without periods. If the bridge carries one way traffic, add this as a suffix. Examples: I 94, I 35 SB, US 12, US 2 WB, MN 36, CSAH 44, CR 88. If no highway number exists, use the street name/number, etc.
Route number	Often the same as Facility carried. If a minor road crosses over a major highway, enter the major highway number. May be a second highway no.over/under. Omit Prefixes (I, US, MN, CSAH, etc.)
Feature intersected	Highway, river, or feature **under** the bridge.
Mile post	Blank at this time.
Units	English preferred
Recent ADTT	Blank at this time.
District	Select from pull-down menu
County	Select from pull-down menu
Owner	Select from pull-down menu
Maintainer	Select from pull-down menu, Usually same as owner.
Administrative Area *	Select from the pull down menu. (This is used for sorting into folders.)
NHS Indicator	Blank at this time.
Functional Class	Unknown

* This field is being used for an alternate purpose, as defined by the MnDOT Bridge Rating Section

General

Set tolerances, in Configuration Browser, System Defaults, Tolerance tab:

Units	Tolerance
ft.	0.01
in.	0.05
mi.	0.010

Use the girder system method whenever practical.

Let the system define effective widths even though the calculation is liberal.

Use the material strengths as given on plan. If there is no plan and no other source is available, select from the values given in the MCE based on the year of construction.

Under STRUCTURE DEFINITIONS, Load Case Description, the Composite Dead Load (DL2) needs to be separated into two (or more) items (lines). This is because LRFR will use different load factors for each of them:

D, DC includes: original wearing course, railings, median, sidewalk, etc

D, DW includes the future wearing course and utilities.

For new bridges use concrete deck strength of 4 ksi. (Class C)

When selecting concrete reinforcement, check box if epoxy coated.

Deck profile: Total thickness: Enter the depth of the initial deck pour. In the girder system method enter the actual wearing course. In the girder line method, the wearing course is calculated manually and entered as weight per foot in member loads. (The effective width and depth for composite action is entered elsewhere.)

The default setting for transverse distribution of dead load 2 is uniformly to all beams. This is the preferred method when the loading is symmetrical across the bridge width. When the load is not symmetrical, another distribution method may be chosen, or a dummy load may be added to make it symmetrical.

In a Virtis window where the engineer is asked to select a name for a parameter, select a name that is short and descriptive. Some examples are:

PARAMETER FULL NAME	Short name
Bridge Alternatives	Br Alt 1
Stress limits – concrete	PS Conc 7.5
Stirrups for prestressed bm	# 4 Inv U
Prestressed Properties – strands	Strd 0.5 in

Completion of Rating

Compare new bridge ratings calculated with Virtis to any old rating that may exist, and resolve any major discrepancies.

Fill out and sign forms for the new rating. The signature page is to be filled out and signed in ink.

Verify that the bridge rating can be run from the Bridge Explorer.

Prestressed Concrete

In stress limits, select the beam concrete strength. Round down to the nearest 100 psi in the agency library. Adjust the F'ci strength.

Set shear calculation limits to comply with AASHTO section 9.20.2: To change this go to the Member Alternative window. Click on the Engine tab and configure engine properties to BRASS LFD. Click on Properties then go to the Miscellaneous tab and at the bottom of screen are two options. Select the option "no limit on Mcr/Mmax". (The default setting is "limit Mcr/Mmax to 1.0".) (Note that if the shear has been turned off this choice will not be available.) Note that this may have to be set multiple times for each bridge.

> *Explanation: Concrete shear for PCBs is calculated from the lowest of Vci or Vcw, per AASHTO 9.20.2. In the calculation of Vci there is a term Mcr / Mmax. When the AASHTO code was first developed the ratio of this term computed to be less than 1 for the commonly used PCB shapes. In fact Dr Jay Puckett did some research and showed this term should be limited to 1. In BRASS with version 5.3 and before, this term was limited to 1 based on Puckett's research and not the code. With Virtis 5.3.1 we now have the option of allowing this ratio to go above 1.0 and MnDOT is electing to do that.*

If the rating is controlled by shear at the end of the beam, turn off the Virtis set tenth point checks. This is found at the member alternative, engine tab. Then add points of interest at h/2, 1/10, 2/10, 3/10, and 5/10.

If shear still controls the rating, also check and record the rating with shear turned off and record RF on the Bridge Rating form table in the notes column.

Concrete Slab Spans

Enter a strip of slab one unit wide.

Use a slab depth of the full depth minus 0.5 inch. (Thus considering the wearing course as fully composite with the base slab and of the same strength.)

The remaining 0.5 inch is entered in member loads as weight per foot.

Check the ignore shear box.

Steel Beams, Composite

The girder schedule is the preferred method to enter beam sizes.

Items not required to enter: welds, bolts, splices, rebar in the slab (unless specifically designed for this).

If the beam was designed for composite action in the negative moment region, rate it the same way and with the longitudinal slab rebars included.

The contraflexure points must be entered in as a percentage of span. You can get these from either the girder elevation plan sheet or the dead load moments in Virtis.

Follow AASHTO Guide Specification for Curved Steel Bridges, 2003, if beams have curvature.

Be careful to input correct stool height.

Verify diaphragms and stiffener locations on elevation view.

If diaphragms are straight on a skewed bridge make sure to account for two stiffeners and diaphragms close together.

BRASS calculates plastic capacity for simple spans. It does not if the girder is a hybrid.

Check to see that critical locations are not artificially low at counterflexure points. If so then adjust counterflexure points in engine tab so this won't control.

Use elastic rating for continuous steel rating. See equation 10-129c. This is controlled by ***not*** checking compact, located at -Member Alternatives- (member name)-engine-BRASS LFD-properties.

Virtis does not do the 10% redistribution per AASHTO 10.48.1.3

Do not let bearing stiffeners control the final rating.

Virtis 5.3.1 will rate floor beams and stringers.

Timber

Virtis does rate timber decks for moments. Use the provided MathCAD sheet to rate decks for shear. This might control the rating.

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Form 90 Revised: Apr 07	Culvert Rating Form	
Bridge Number:	Year Built:	Year Remodeled:
County:	Bridge Owner:	
Route:	Feature Crossed:	
Culvert Type:		
Structure Type Code:	Culvert Dimensions:	
No. of Barrels:	Barrel Length:	

Rating Guidelines

Material	Culvert Type	Structure Type Code	Inventory Load Rating	Operating Load Rating
Cast-in-place Concrete*	Box	113	HS 22.0	HS 33.0
	Type W Box (1930 era)	113	HS 16.0	HS 24.0
	Footing Supported Arch	112	HS 20.0	HS 30.0
Pre-cast Concrete*	Box	513	HS 24.0	HS 36.0
	Footing Supported Arch	512	HS 20.0	HS 30.0
	Round Pipe	514	HS 24.0	HS 36.0
	Pipe-Arch	515	HS 22.0	HS 33.0
***For Load Factor HS 25 Design Structures use:**			**HS 25.0**	**HS 42.0**
Aluminum	Box	913	HS 14.0	HS 21.0
Metal	Footing Supported Arch	312	HS 12.0	HS 18.0
	Round Pipe	314	HS 16.0	HS 24.0
	Pipe-Arch	315	HS 16.0	HS 24.0
	Elliptical	316	HS 16.0	HS 24.0
Timber	Box	713	HS 14.0	HS 21.0
Masonry	Footing Supported Arch	812	HS 18.0	HS 27.0

The above table may be used as a guideline to the culvert rating.

If the culvert condition code is 4 or less, rate by Physical Inspection Rating instead of this form.

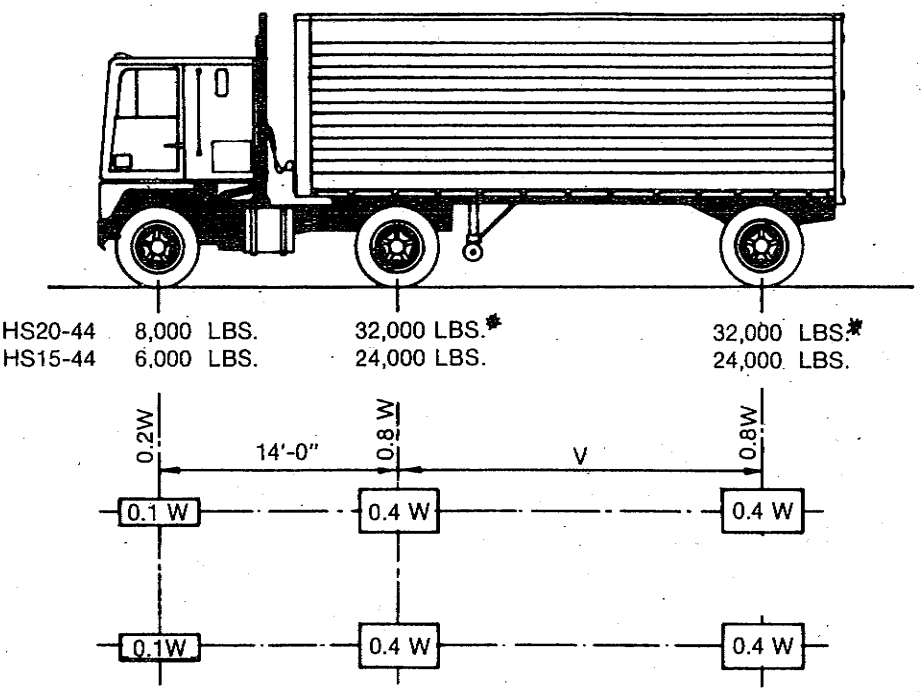
Inventory Rating	

Operating Rating	

Rated by (please print) _____

Title _____ Date: _____

2002 AASHTO STD. SPECS.



W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H TRUCK.

V = VARIABLE SPACING — 14 FEET TO 30 FEET INCLUSIVE. SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES.

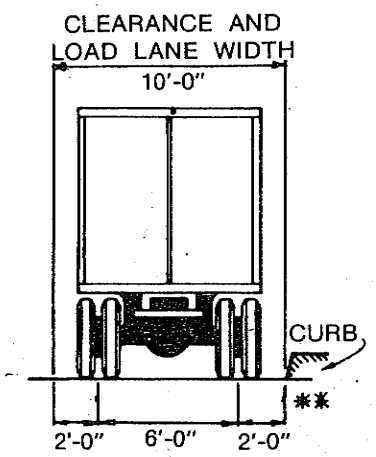


FIGURE 3.7.7A. Standard HS Trucks

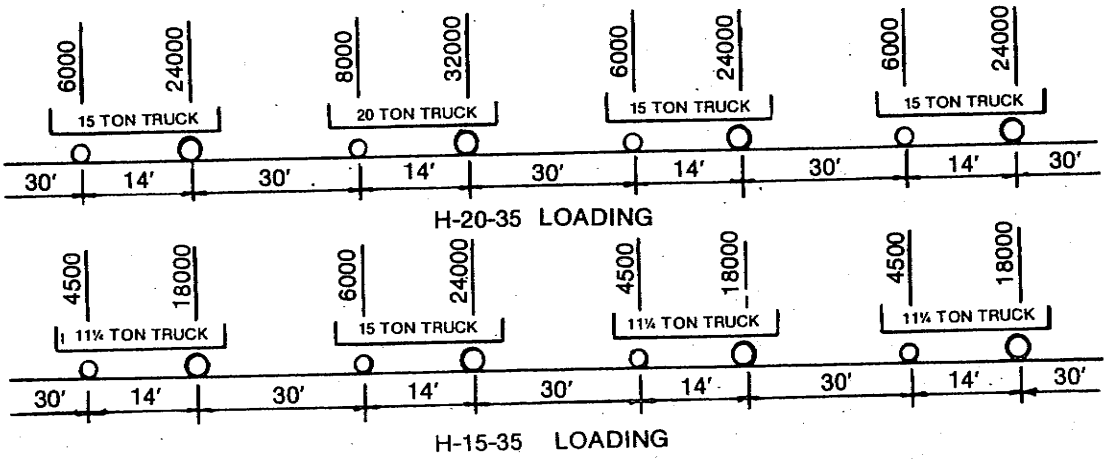
*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H 20 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2.)

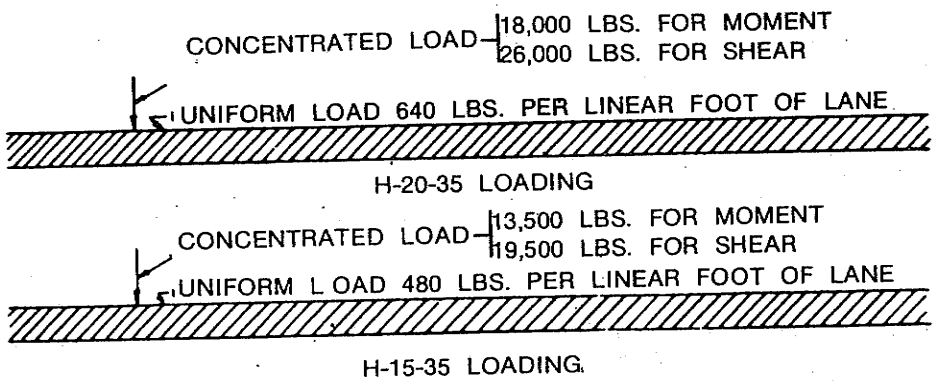
2002 AASHTO STD. SPECS

APPENDIX B

TRUCK TRAIN AND EQUIVALENT LOADINGS — 1935 SPECIFICATIONS AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS



TRUCK TRAIN LOADING



EQUIVALENT LOADING

LANE WIDTH 10 FEET

2002 AASHTO STD. SPECS

LOADING—HS 20-44 (MS18)

TABLE OF MAXIMUM MOMENTS, SHEARS, AND REACTIONS—SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0(b)	32.0(b)	42	485.3(b)	56.0(b)
2	16.0(b)	32.0(b)	44	520.9(b)	56.7(b)
3	24.0(b)	32.0(b)	46	556.5(b)	57.3(b)
4	32.0(b)	32.0(b)	48	592.1(b)	58.0(b)
5	40.0(b)	32.0(b)	50	627.9(b)	58.5(b)
6	48.0(b)	32.0(b)	52	663.6(b)	59.1(b)
7	56.0(b)	32.0(b)	54	699.3(b)	59.6(b)
8	64.0(b)	32.0(b)	56	735.1(b)	60.0(b)
9	72.0(b)	32.0(b)	58	770.8(b)	60.4(b)
10	80.0(b)	32.0(b)	60	806.5(b)	60.8(b)
11	88.0(b)	32.0(b)	62	842.4(b)	61.2(b)
12	96.0(b)	32.0(b)	64	878.1(b)	61.5(b)
13	104.0(b)	32.0(b)	66	914.0(b)	61.9(b)
14	112.0(b)	32.0(b)	68	949.7(b)	62.1(b)
15	120.0(b)	34.1(b)	70	985.6(b)	62.4(b)
16	128.0(b)	36.0(b)	75	1,075.1(b)	63.1(b)
17	136.0(b)	37.7(b)	80	1,164.9(b)	63.6(b)
18	144.0(b)	39.1(b)	85	1,254.7(b)	64.1(b)
19	152.0(b)	40.4(b)	90	1,344.4(b)	64.5(b)
20	160.0(b)	41.6(b)	95	1,434.1(b)	64.9(b)
21	168.0(b)	42.7(b)	100	1,524.0(b)	65.3(b)
22	176.0(b)	43.6(b)	110	1,703.6(b)	65.9(b)
23	184.0(b)	44.5(b)	120	1,883.3(b)	66.4(b)
24	192.7(b)	45.3(b)	130	2,063.1(b)	67.6
25	207.4(b)	46.1(b)	140	2,242.8(b)	70.8
26	222.2(b)	46.8(b)	150	2,475.1	74.0
27	237.0(b)	47.4(b)	160	2,768.0	77.2
28	252.0(b)	48.0(b)	170	3,077.1	80.4
29	267.0(b)	48.8(b)	180	3,402.1	83.6
30	282.1(b)	49.6(b)	190	3,743.1	86.8
31	297.3(b)	50.3(b)	200	4,100.0	90.0
32	312.5(b)	51.0(b)	220	4,862.0	96.4
33	327.8(b)	51.6(b)	240	5,688.0	102.8
34	343.5(b)	52.2(b)	260	6,578.0	109.2
35	361.2(b)	52.8(b)	280	7,532.0	115.6
36	378.9(b)	53.3(b)	300	8,550.0	122.0
37	396.6(b)	53.8(b)			
38	414.3(b)	54.3(b)			
39	432.1(b)	54.8(b)			
40	449.8(b)	55.2(b)			

(a) Concentrated load is considered placed at the support. Loads used are those stipulated for shear.

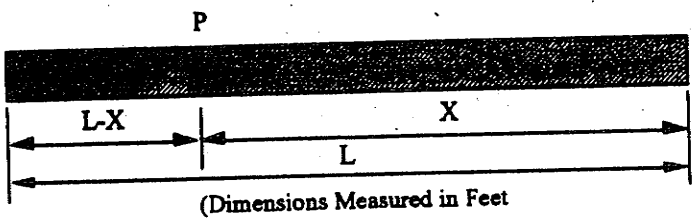
(b) Maximum value determined by Standard Truck Loading. Otherwise the Standard Lane Loading governs.

2003 AASHTO MCE

APPENDIX A7

FORMULAE FOR MAXIMUM SHEAR AT ANY POINT ON SPAN
(No Impact Included)
(Simple Spans Only)

Type Load	L-X / L	Formula for Maximum Shear (1)	Length of Truck	Minimum L-X	Minimum X(2)
3	0-0.500	$V = \frac{25(X-7.44)}{L}$	19'	0	19'
3-S2	0-0.500	$V = \frac{36(X-18.61)}{L}$	41'	0	41'
3-3	0-0.500	$V = \frac{40(X-23.90)}{L}$	54'	0	54'



(Dimensions Measured in Feet

V = Shear at a point "P" which is (L-X) distance from end of span in kips per wheel line.

(1) These formulae are applicable only when dimension "X" exceeds total length of truck.

(2) For spans where dimension "X" is less than the minimum, the maximum shears are to be determined from statics.

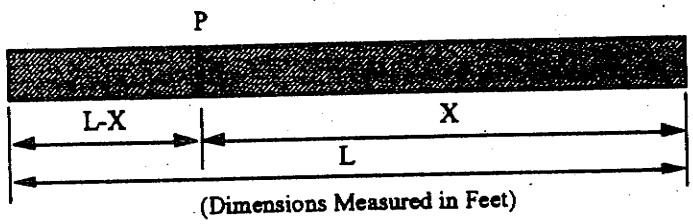
2003 AASHTO MCE

APPENDIX A8

FORMULAE FOR MAXIMUM SHEAR
AT ANY POINT ON SPAN
(No Impact Included)
(Simple Spans Only)

Type Load	$\frac{L-X}{L}$	Use for Girder Lengths	Formula for Maximum Shear (1)	Minimum L-X	Minimum X
HS-20	0-0.500	Under 42'	$V = \frac{36(X-4.67)}{L} - 4$	14	14
		42' to 120'*	$V = \frac{36(X-9.33)}{L}$	0	28
HS-15	0-0.500	Under 42'	$V = \frac{27(X-4.67)}{L} - 3$	14	14
		42' to 120'*	$V = \frac{27(X-9.33)}{L}$	0	28
H-20	0-0.500	to 35'*	$V = \frac{20(X-2.8)}{L}$	0	14
H-15	0-0.500	to 35'*	$V = \frac{15(X-2.8)}{L}$	0	14

(1) All values based on standard truck loadings.
* Truck loading does *not* govern shear beyond the lengths specified. Use lane loading.



(Dimensions Measured in Feet)

V = Shear to Left of point "P" in kips per wheel line.

MINNESOTA GROSS WEIGHT TABLE

Distance in Feet between Axle Centers

Northland College Revision = 10/2003

Number of Consecutive Axles

	2	2	3	4	5	6	7
4	34000	34000					
5	34000	34000					
6	34000	34000					
7	34000	34000	37000*				
8	34000	34000	38500*				
8 plus	34000	38000	42000				
9	35000	39000	43000				
10	36000	40000	43500	49000			
11			44500	49500			
12			45000	50000			
13			46000	51000			
14			46500	51500	57000		
15			47500	52000	57500		
16			48000	53000	58000		
17			49000	53500	59000		
18			49500	54000	59500		
19			50500	55000	60000		
20			51000	55500	60500	66000	72000
21			52000	56000	61500	67000	72500
22			52500	57000	62000	67500	73000
23			53500	57500	62500	68000	73500
24			54000	58000	63000	68500	74000
25			55000	59000	64000	69000	75000
26			55500	59500	64500	70000	75500
27			56500	60000	65000	70500	76000
28			57000	61000	65500	71000	76500
29			58000	61500	66500	71500	77000
30			58500	62000	67000	72000	77500
31			59500	63000	67500	73000	78500
32			60000	63500	68000	73500	79000
33				64000	69000	74000	79500
34				65000	69500	74500	80000
35				65500	70000	75000	
36				66000	70500	76000	
37				67000	71500	76500	
38				67500	72000	77000	
39				68000	72500	77500	
40				69000	73280	78000	
41				69500	74000	79000	
42				70000	74500	79500	
43				71000	75000	80000	
44				71500	75500		
45				72000	76500		
46				72500	77000		
47				73500	77500		
48				74000	78000		
49				74500	79000		
50				75500	79500		
51				76000	80000		

These weights valid only on 10-Ton Routes

8 plus indicates a measurement over 8 feet but less than 9 feet

No vehicle or combination of vehicles shall be operated upon the highways of this state where the total gross weight on any group of two or more consecutive axles exceeds that given in the Table of Weights.

These weights valid only on 10-Ton Routes

Two consecutive tandems may carry a gross weight of 34,000 lbs. each, and a total of 68,000 lbs. together, when the centers of the first and last axle of the group are spaced 36 feet or more.

These weights valid only on 10-Ton Routes

These weights valid only on 10-Ton Routes. Gross Vehicle Weight greater than 73,280 lbs. Allowed on 9-Ton Routes only with 6 or more axles.

* These weights only apply to vehicles manufactured prior to August 1, 1991.

MINNESOTA TIRE WEIGHT CHARTS

10/2003

STEERING AXLES* (Foremost & Rearmost) **600# /in.**		
Tire Width	1 Tire	2 Tires
8"	4,800	9,600
9"	5,400	10,800
10"	6,000	12,000
11"	6,600	13,200
12"	7,200	14,400
13"	7,800.	15,600
14"	8,400	16,800
15"	9,000	18,000
16"	9,600	19,200
17"	10,200	-
18"	10,800	-
19"	11,400	-
20"	12,000	

* ALL AXLES OTHER THAN STEERING AXLES** **500# /in.**		
Tire Width	1 Ti re	2 Tires
8"	4,000	8,000
9"	4,500	9,000
10"	5,000	10,000
11"	5,500	11,000
12"	6,000	12,000
13"	6,500	13,000
14"	7,000	14,000
15"	7,500	15,000
16"	8,000	16,000
17"	8,500	17,000
18"	9,000	18,000
19"	9,500	19,000
20"	10,000	20,000

Approximate Metric Reference Size

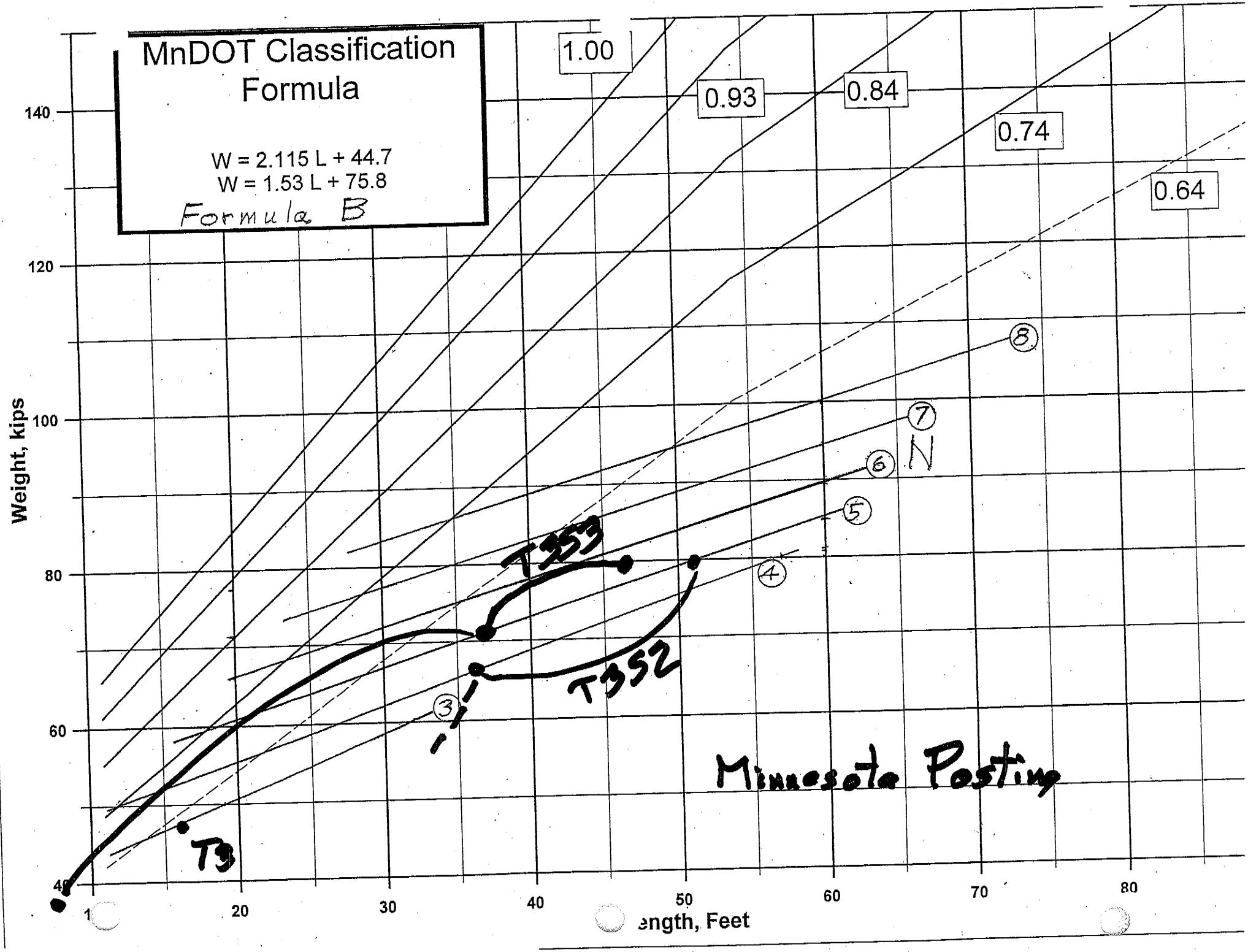
(divide mm by 25.4 for exact conversion)

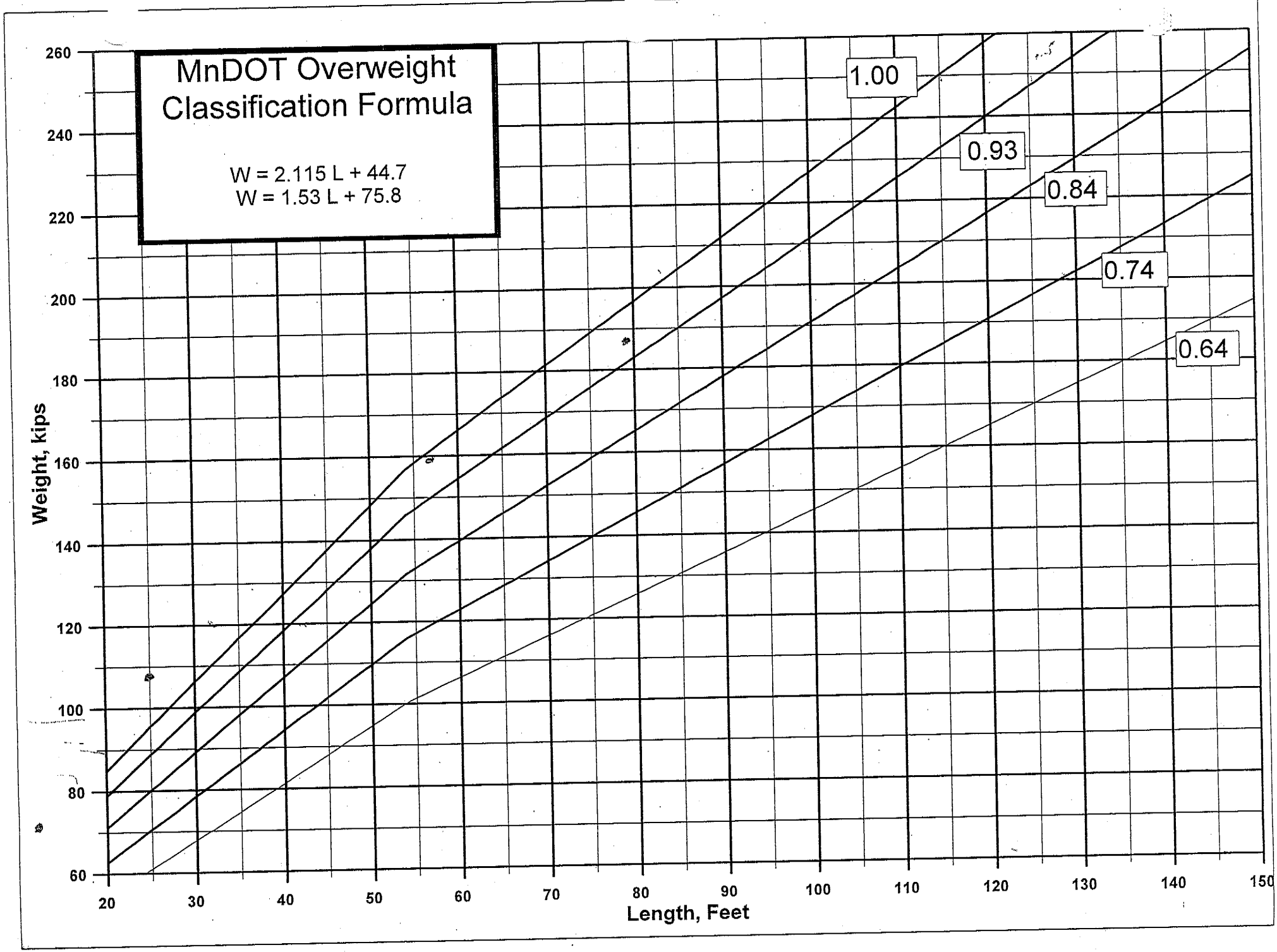
Metric size "rounded" to inches.

245 = 9.6	325 = 12.8	405 = 15.9
255 = 10.	335 = 13.2	415 = 16.3
265 = 10.4	345 = 13.6	425 = 16.7
275 = 10.8	355 = 13.9	435 = 17.1
285 = 11.2	365 = 14.4	445 = 17.5
295 = 11.6	375 = 14.8	455 = 17.9
305 = 12.0	385 = 15.2	465 = 18.3
315 = 12.4	395 = 15.5	475 = 18.7

To convert Kilogram ratings to pounds multiply by 2.2

Kg X 2.2 = lbs.


MnDOT Classification Formula
W = 2.115 L + 44.7
W = 1.53 L + 75.8
Formula B
1.00
0.93
0.84
0.74
0.64
Weight, kips
140
120
100
80
60
40
20
30
40
50
60
70
80
ength, Feet
8
7
6
5
4
3
N
T353
T352
T3
Minnesota Posting


MnDOT Overweight
Classification Formula
W = 2.115 L + 44.7
W = 1.53 L + 75.8
1.00
0.93
0.84
0.74
0.64
Weight, kips
260
240
220
200
180
160
140
120
100
80
60
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Length, Feet

PERMISSIBLE GROSS LOADS FOR VEHICLES IN REGULAR OPERATION [1]

Based on weight formula $W = 500\left[\frac{LN}{N-1} + 12N + 36\right]$

Distance in feet (L) between the extremes of any group of 2 or more consecutive axles	Maximum load in pounds carried on any group of 2 or more consecutive axles							
N =	2 AXLES	3 AXLES	4 AXLES	5 AXLES	6 AXLES	7 AXLES	8 AXLES	9 AXLES
4 (Tandem Axle Weight (see pages 4 & 5))	34,000	----------	----------	----------	----------	----------	----------	----------
5 (Tandem Axle Weight (see pages 4 & 5))	34,000	----------	----------	----------	----------	----------	----------	----------
6 (Tandem Axle Weight (see pages 4 & 5))	34,000	----------	----------	----------	----------	----------	----------	----------
7 (Tandem Axle Weight (see pages 4 & 5))	34,000	----------	----------	----------	----------	----------	----------	----------
8 & less (Tandem Axle Weight (see pages 4 & 5))	34,000	34,000	----------	----------	----------	----------	----------	----------
more than 8	38,000	42,000	----------	----------	----------	----------	----------	----------
9	39,000	42,500	----------	----------	----------	----------	----------	----------
10	40,000	43,500	----------	----------	----------	----------	----------	----------
11	----------	44,000	----------	----------	----------	----------	----------	----------
12	----------	45,000	50,000	----------	----------	----------	----------	----------
13	----------	45,500	50,500	----------	----------	----------	----------	----------
14	----------	46,500	51,500	----------	----------	----------	----------	----------
15	----------	47,000	52,000	----------	----------	----------	----------	----------
16	----------	48,000	52,500	58,000	----------	----------	----------	----------
17	----------	48,500	53,500	58,500	----------	----------	----------	----------
18	----------	49,500	54,000	59,000	----------	----------	----------	----------
19 Example	----------	50,000	54,500	60,000	----------	----------	----------	----------
20 (see page 8)	----------	51,000	55,500	60,500	66,000	----------	----------	----------
21	----------	51,500	56,000	61,000	66,500	----------	----------	----------
22	----------	52,500	56,500	61,500	67,000	----------	----------	----------
23	----------	53,000	57,500	62,500	68,000	----------	----------	----------
24	----------	54,000	58,000	63,000	68,500	74,000	----------	----------
25	----------	54,500	58,500	63,500	69,000	74,500	----------	----------
26	----------	55,500	59,500	64,000	69,500	75,000	----------	----------
27	----------	56,000	60,000	65,000	70,000	75,500	----------	----------
28	----------	57,000	60,500	65,500	71,000	76,500	82,000	----------
29	----------	57,500	61,500	66,000	71,500	77,000	82,500	----------
30	----------	58,500	62,000	66,500	72,000	77,500	83,000	----------
31	----------	59,000	62,500	67,500	72,500	78,000	83,500	----------
32	----------	60,000	63,500	68,000	73,000	78,500	84,500	90,000
33	----------	----------	64,000	68,500	74,000	79,000	85,000	90,500
34	----------	----------	64,500	69,000	74,500	80,000	85,500	91,000
35	----------	----------	65,500	70,000	75,000	80,500	86,000	91,500
36	----------	Exception (see page 10)	66,000	70,500	75,500	81,000	86,500	92,000
37	----------	----------	66,500	71,000	76,000	81,500	87,000	93,000
38	----------	----------	67,500	71,500	77,000	82,000	87,500	93,500
39	----------	----------	68,000	72,500	77,500	82,500	88,500	94,000
40	----------	----------	68,500	73,000	78,000	83,500	89,000	94,500
41	----------	----------	69,500	73,500	78,500	84,000	89,500	95,000
42	----------	----------	70,000	74,000	79,000	84,500	90,000	95,500
43	----------	----------	70,500	75,000	80,000	85,000	90,500	96,000
44	----------	----------	71,500	75,500	80,500	85,500	91,000	96,500
45	----------	----------	72,000	76,000	81,000	86,000	91,500	97,500
46	----------	----------	72,500	76,500	81,500	87,000	92,500	98,000
47	----------	----------	73,500	77,500	82,000	87,500	93,000	98,500
48	----------	----------	74,000	78,000	83,000	88,000	93,500	99,000
49	----------	----------	74,500	78,500	83,500	88,500	94,000	99,500
50	----------	----------	75,500	79,000	84,000	89,000	94,500	100,000
51	----------	----------	76,000	80,000	84,500	89,500	95,000	100,500
52	----------	----------	76,500	80,500	85,000	90,500	95,500	101,000
53	----------	----------	77,500	81,000	86,000	91,000	96,500	102,000
54	----------	----------	78,000	81,500	86,500	91,500	97,000	102,500
55	----------	----------	78,500	82,500	87,000	92,000	97,500	103,000
56	----------	----------	79,500	83,000	87,500	92,500	98,000	103,500
57	----------	Interstate Gross Weight Limit (see page 4)	80,000	83,500	88,000	93,000	98,500	104,000
58	----------	----------	----------	84,000	89,000	94,000	99,000	104,500
59	----------	----------	----------	85,000	89,500	94,500	99,500	105,000
60	----------	----------	----------	85,500	90,000	95,000	100,500	105,500

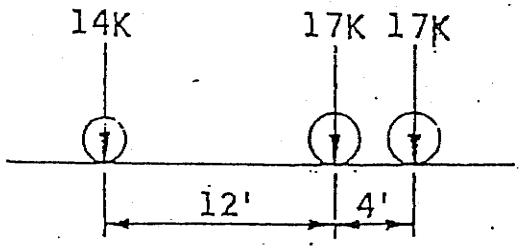
[1] The permissible loads are computed to the nearest 500 pounds as required by statute.

[2] The following loaded vehicles must not operate over H15-44 bridges: 3-S2 (5-axle) with wheelbase less than 38 feet; 2-S1-2 (5-axle) with wheelbase less than 45 feet; 3-3 (6-axle) with wheelbase less than 45 feet; and 7- 8- and 9-axle vehicles regardless of wheelbase.

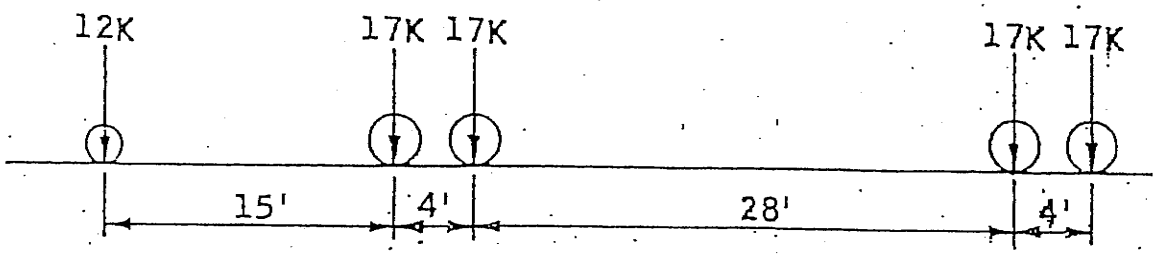
TYPICAL LEGAL LOADS FOR DESIGNATED ROUTES

NOTE: A SINGLE AXLE MAY WEIGHT UP TO 20,000# AND CONTROLS FOR SIMPLE SPANS LESS THEN 9'

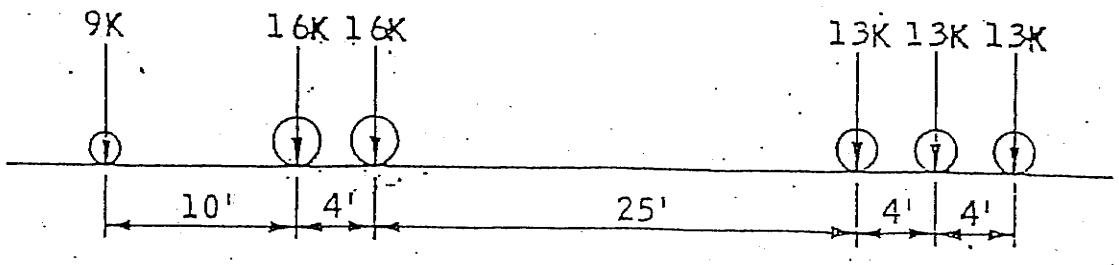
TYPE 3 VEHICLE
W = 48K
L = 16'



TYPE 3S2-40 VEHICLE
W = 80K
L = 51'



TYPE 3S3 VEHICLE
W = 80K
L = 47'

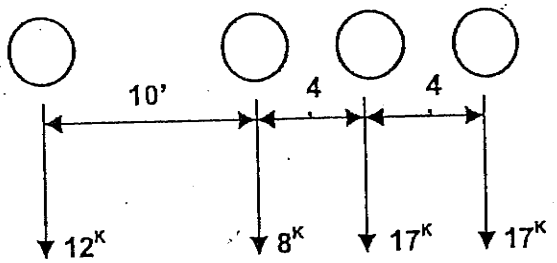


MINNESOTA TRUCK MOMENTS

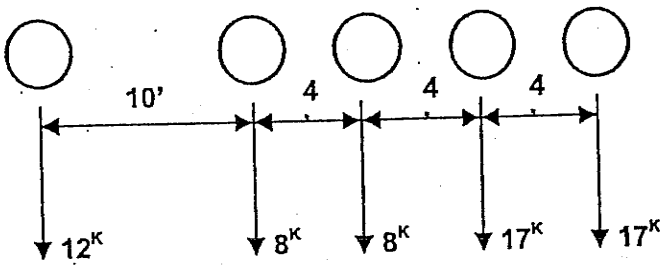
TRUCK MOMENTS PER WHEEL---DESIGNATED ROUTES(FT.KIPS)

WITHOUT IMPACT			SPAN FEET C/C	WITH IMPACT		
TYPE 3	TYPE 3S2-40	TYPE 3S3		TYPE 3	TYPE 3S2-40	TYPE 3S3
27.2	27.2	25.6	10	35.4	35.4	33.3
31.3	31.3	29.5	11	40.7	40.7	38.3
35.4	35.4	33.3	12	46.0	46.0	43.3
39.6	39.6	37.4	13	51.5	51.5	48.6
43.7	43.7	42.2	14	56.8	56.8	54.9
47.9	47.9	47.1	15	62.2	62.2	61.3
52.1	52.1	52.0	16	67.7	67.7	67.6
56.3	56.3	56.9	17	73.1	73.1	73.9
60.4	60.4	61.7	18	78.6	78.6	80.3
64.6	64.6	66.7	19	84.0	84.0	86.6
68.9	68.9	71.5	20	89.5	89.5	93.0
73.1	73.1	76.4	21	95.0	95.0	99.3
77.3	77.3	81.2	22	100.5	100.5	105.7
81.5	81.5	86.1	23	105.9	105.9	112.0
86.0	85.7	91.0	24	111.8	111.4	118.3
92.0	89.9	95.9	25	119.7	116.9	124.7
98.0	94.2	100.7	26	127.4	122.4	131.0
104.0	98.4	105.1	27	135.2	127.9	137.3
110.0	102.6	110.5	28	143.0	133.4	143.[illegible]
116.0	106.8	115.4	29	150.8	138.9	150.[illegible]
122.0	111.6	120.7	30	158.6	145.1	156.4
133.8	123.1	130.0	32	174.0	160.0	169.0
145.5	134.5	139.7	34	189.3	174.8	181.7
157.7	145.9	149.5	36	205.0	189.7	194.3
169.7	157.4	159.2	38	220.6	204.6	207.1
181.7	168.9	169.0	40	236.1	219.5	219.7
193.6	180.3	178.7	42	251.5	234.2	232.4
205.6	191.8	188.5	44	266.4	248.5	244.3
217.6	203.2	198.2	46	281.1	262.6	256.2
229.5	214.7	208.0	48	295.9	276.7	268.1
241.5	226.2	217.7	50	310.6	290.9	279.8
253.5	237.6	227.5	52	325.0	304.9	291.7
265.5	249.1	240.5	54	339.6	318.6	307.6
277.5	260.6	257.7	56	354.0	332.5	328.9
289.4	278.1	274.9	58	368.5	346.3	350.0
301.4	284.6	293.1	60	382.8	361.5	372.2
361.4	382.7	389.7	70	453.9	479.8	489.6
421.3	481.2	487.3	80	524.1	598.7	606.2
481.3	580.0	585.3	90	593.4	715.3	721.2
541.3	679.0	683.8	100	661.4	830.0	835.6
661.2	877.8	881.5	120	796.1	1056.9	1061.4
781.2	1076.8	1080.0	140	929.8	1280.4	1284.0
901.2	1276.1	1278.7	160	1058.9	1499.4	1502.4
1021.1	1475.5	1477.7	180	1188.5	1717.5	172[illegible]
1141.1	1675.1	1676.9	200	1316.9	1933.1	193[illegible]

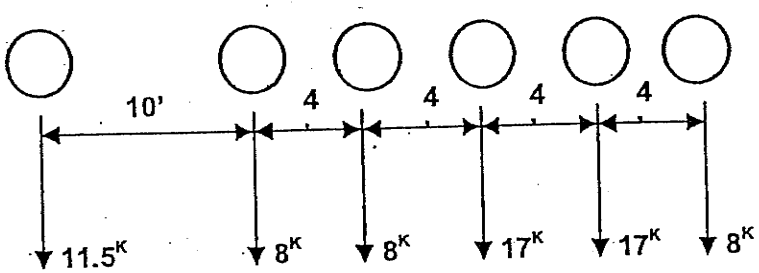
FROM NEW MCE' NOT PUBLISHED YET



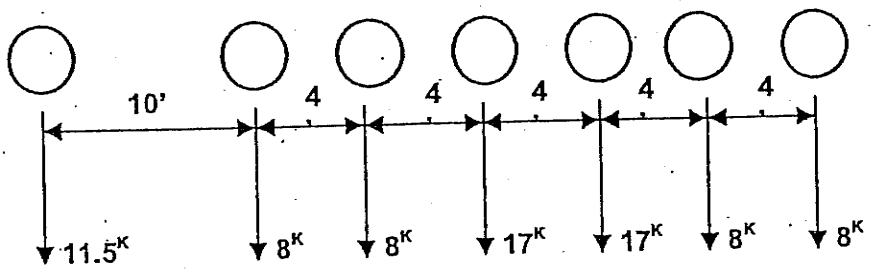
SU4 TRUCK
GVW = 54 KIPS
18'



SU5 TRUCK
GVW = 62 KIPS
22'



SU6 TRUCK
GVW = 69.5 KIPS
26'



SU7 TRUCK
GVW = 77.5 KIPS.
30'
38.75 T

Figure 7.4.3.2 BRIDGE POSTING LOADS FOR SINGLE UNIT TRUCKS THAT MEET FEDERAL BRIDGE FORMULA B.

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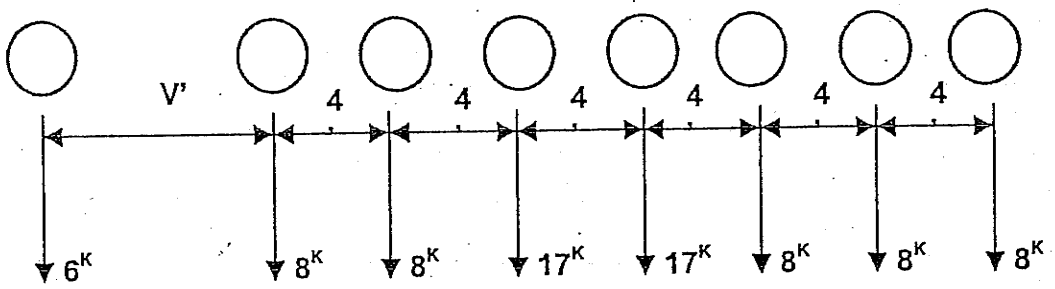


V = VARIABLE DRIVE AXLE SPACING — 6'0" TO 14'-0". SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM LOAD EFFECTS.

AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

MAXIMUM GVW = 80 KIPS

AXLE GAGE WIDTH = 6'-0"

Figure 6.7.2.3 NOTIONAL RATING LOAD "NRL" FOR SINGLE UNIT TRUCKS THAT MEET FEDERAL BRIDGE FORMULA B.

- From NEW "MCE" NOT PUBLISHED YET.

APPENDIX A3-1

LIVE LOAD MOMENTS ON LONGITUDINAL STRINGERS OR GIRDERS

Live Load Moment in Foot-Kips Per Wheel Line

Type of Loading (Without Impact)						Span Feet c/c	Type of Loading (With Impact)					
HS20	NRL	SU4	SU5	SU6	SU7		HS20	NRL	SU4	SU5	SU6	SU7
20.0	10.6	10.6	10.6	10.6	10.6	5	26.0	13.8	13.8	13.8	13.8	13.8
24.0	12.8	12.8	12.8	12.8	12.8	6	31.2	16.6	16.6	16.6	16.6	16.6
28.0	15.2	15.2	15.2	15.2	15.2	7	36.4	19.8	19.8	19.8	19.8	19.8
32.0	19.1	19.1	19.1	19.1	19.1	8	41.6	24.8	24.8	24.8	24.8	24.8
36.0	23.1	23.1	23.1	23.1	23.1	9	46.8	30.0	30.0	30.0	30.0	30.0
40.0	27.9	27.9	27.9	27.9	27.9	10	52.0	36.3	36.3	36.3	36.3	36.3
44.0	33.1	33.1	33.1	33.1	33.1	11	57.2	43.0	43.0	43.0	43.0	43.0
48.0	38.3	38.3	38.3	38.3	38.3	12	62.4	49.8	49.8	49.8	49.8	49.8
52.0	43.5	43.5	43.5	43.5	43.5	13	67.6	56.6	56.6	56.6	56.6	56.6
56.0	48.8	48.8	48.8	48.8	48.8	14	72.8	63.4	63.4	63.4	63.4	63.4
60.0	54.4	54.0	54.0	54.4	54.4	15	78.0	70.7	70.2	70.2	70.7	70.7
64.0	60.6	59.2	59.2	60.6	60.6	16	83.2	78.8	77.0	77.0	78.8	78.8
68.0	66.7	64.5	65.3	66.7	66.7	17	88.4	86.7	83.9	84.9	86.7	86.7
72.0	73.6	69.7	71.5	73.6	73.6	18	93.6	95.7	90.6	93.0	95.7	95.7
76.0	80.8	74.9	77.8	80.8	80.8	19	98.8	105.0	97.4	101.1	105.0	105.0
80.0	88.1	80.2	84.0	88.1	88.1	20	104.0	114.5	104.3	109.2	114.5	114.5
84.0	95.3	85.4	90.3	95.3	95.3	21	109.2	123.9	111.0	117.4	123.9	123.9
88.0	102.6	90.7	96.5	102.6	102.6	22	114.4	133.4	117.9	125.5	133.4	133.4
92.0	110.2	95.9	102.8	109.8	110.2	23	119.6	143.3	124.7	133.6	142.7	143.3
96.3	118.4	101.2	109.0	117.1	118.4	24	125.2	153.9	131.6	141.7	152.2	153.9
103.7	126.6	106.4	115.3	124.3	126.6	25	134.8	164.5	138.3	149.9	161.6	164.5
111.1	135.5	111.6	121.5	131.6	134.8	26	144.4	176.2	145.1	158.0	171.1	175.2
118.5	144.8	116.9	127.8	138.8	143.0	27	154.1	188.2	152.0	166.1	180.4	185.9
126.0	154.0	123.4	134.0	146.1	151.2	28	163.8	200.2	160.4	174.2	189.9	196.6
133.5	163.3	130.1	140.3	153.3	159.4	29	173.6	212.3	169.1	182.4	199.3	207.2
141.0	172.5	136.8	146.5	160.6	167.6	30	183.3	224.3	177.8	190.5	208.7	217.9
156.2	191.0	150.2	159.0	175.1	184.0	32	203.1	248.3	195.3	206.7	227.6	239.2
171.8	209.5	163.6	172.0	189.6	200.5	34	223.3	272.4	212.7	223.6	246.5	260.7
189.4	228.9	177.1	187.3	205.4	216.9	36	246.2	297.6	230.2	243.5	267.0	28
207.1	248.8	190.5	202.7	222.7	235.6	38	269.2	323.4	247.7	263.5	289.5	306
224.9	268.8	204.0	218.0	240.0	255.0	40	292.4	349.4	265.1	283.4	312.0	331.5
242.7	288.8	217.4	233.4	257.3	274.3	42	315.4	375.3	282.5	303.3	334.3	356.4
260.4	308.7	230.9	248.7	274.7	293.7	44	337.4	400.0	299.2	322.3	356.0	380.6
278.3	328.7	244.3	264.1	292.0	313.1	46	359.7	424.8	315.7	341.3	377.4	404.6
296.1	348.7	257.8	279.5	309.3	332.4	48	381.7	449.5	332.3	360.3	398.7	428.5
314.0	368.7	271.3	294.9	326.6	351.8	50	403.7	474.0	348.8	379.2	419.9	452.3
331.8	388.6	284.8	310.3	344.0	371.2	52	425.5	498.4	365.3	398.0	441.2	476.1
349.7	408.6	298.2	325.7	361.3	390.5	54	447.4	522.7	381.5	416.7	462.2	499.6
367.6	428.6	311.7	341.1	378.7	409.9	56	469.1	547.0	397.8	435.3	483.3	523.1
385.4	448.6	325.2	356.6	396.0	429.3	58	490.7	571.2	414.1	454.0	504.2	546.6
403.3	468.5	338.7	372.0	413.3	448.7	60	512.2	595.1	430.2	472.5	525.0	569.9
492.8	568.5	406.1	449.2	500.1	545.5	70	619.2	714.2	510.2	564.4	628.3	685.4
582.5	668.4	473.5	526.5	586.9	642.4	80	724.5	831.4	589.0	654.9	730.0	799.0
672.2	768.4	540.9	603.8	673.7	739.2	90	828.5	947.0	666.7	744.2	830.4	911.1
762.0	868.3	608.4	681.2	760.5	836.1	100	931.3	1061.3	743.6	832.6	929.5	1021.9
941.6	1068.3	743.3	836.0	934.2	1029.8	120	1133.8	1286.3	895.0	1006.6	1124.8	1240.0
1121.4	1268.2	878.3	990.9	1107.9	1223.6	140	1333.0	1507.5	1044.0	1177.8	1316.9	1454.4
1384.0	1468.2	1013.2	1145.8	1281.6	1417.3	160	1626.8	1725.8	1191.0	1346.8	1506.4	1665.9
1701.0	1668.2	1148.2	1300.7	1455.3	1611.1	180	1979.9	1941.7	1336.4	1513.9	1693.9	1875.2
2050.0	1868.2	1283.2	1455.6	1629.0	1804.8	200	2365.4	2155.6	1480.6	1679.5	1879.6	2082.5
3062.5	2368.1	1620.7	1843.0	2063.3	2289.2	250	3470.8	2683.8	1836.8	2088.7	2338.4	2594.4
4275.0	2868.1	1958.1	2230.4	2497.7	2773.5	300	4777.9	3205.5	2188.5	2492.8	2791.5	3099.8

Bold values are based on standard lane loading. All other values based on standard truck loading.

MN/DOT BR. OFFICE, RATINGS UNIT
ED. LUTGEN

Posting Guidelines for County Ratings, 2006

The Minnesota Posting Trucks are:

The "Typical Legal Loads for Designated Routes" are used for bridges on both 9Ton and 10 Ton routes,
Specialized Hauling Vehicles, Figure 7.4.3.2 in MCE revisions: Trucks SU4 thru SU7 and NRL

Examples of the basic Posting Signs are: (Advance warning signs and supplemental signs are found elsewhere.)

Sign R 12–1A is usually used for lower posting weights and shorter bridges.
Sign R 12- 5 is used for most postings at or below 40 Tons. The trucks associated with the three silhouettes are, top to bottom: M 3, M 3S2, and M3S3; Trucks SU4 thru SU7 are associated with the top silhouette, the single truck.
Sign "Permit 45 Ton" is used where higher posting limits are required due to seasonal or permit loads.

Calculate the bridge capacity for each truck according to the AASHTO Manual for Condition Evaluation for Bridges.

Any bridge with an operating rating of less than HS 28 should be checked to see if posting is required. If any rating factor for any vehicle is below the maximum level as shown below, fill in the worksheet completely, and it will then become the third sheet of the rating documents.

At this time (spring 06) we will post for the new posting trucks SU4 thru SU7 only if posting is already required at or below 40 T for vehicles M3S2 or M3S3.

The posting weights that are to be placed on the posting sign are entered on the front page, form RC – CL. The sign type should also be written there. (Until the form is revised, this may be written in the space below the posting tonnages.)

Round the calculated posting tonnages down to the nearest even Ton. (Exception: 3 T or 5 T may be used on sign R 12-1A.)

With sign R 12–5, post the two combination vehicles at the same tonnage, at the lesser of the two calculated tonnages.

With sign R 12–5, all three vehicles must be posted, the maximums are 40 T, 40 T, 40 T.

Truck M 3 (24)

For RF (Rating Factor) < 0.125 the bridge must be closed.
For 0.125 ≤ RF ≤ 1.10 post at indicated tonnage
For 1.10 < RF this model is not valid, defer to SU 4

Truck SU 4 (27)

For 0.89 ≤ RF ≤ 1.10 post at indicated tonnage and use sign R 12-5
For 1.10 < RF this model is not valid, defer to SU 5.

Truck SU 5 (31)

For .87 < RF < 1.10 post at indicated tonnage and use sign R 12-5
For 1.10 < RF this model is not valid, defer to SU 6.

Truck SU 6 (34.75)

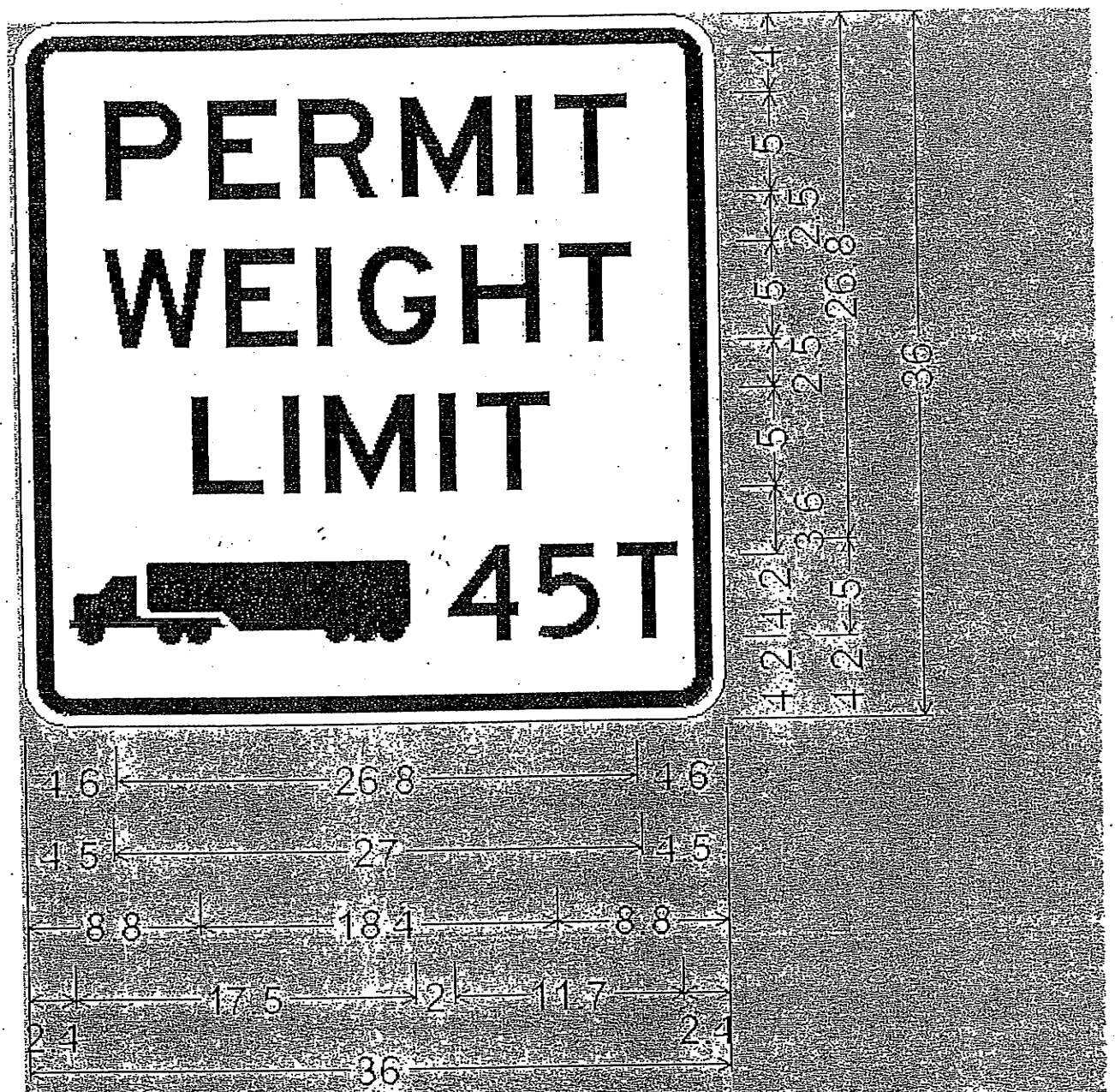
For ..89 < RF < 1.10 post at indicated tonnage and use sign R 12-5
For 1.10 < RF this model is not valid, defer to SU 7.

Truck SU 7 (38.75)

For .89 < RF < 1.025 post at indicated tonnage and use sign R 12-5
For 1.025 < RF < 1.13 post at 40 T
For 1.13 < RF no posting required

Truck M 3S2 – 40, and Truck M 3S3 (40)

For RF < 0.35, post as indicated by truck M3 and use sign R 12-1A
For 0.35 ≤ RF ≤ 1.00 post as indicated with sign R 12-5
For 1.00 < RF ≤ 1.12 post at 40 T with sign R 12-5
For 1.12 < RF ≤ 1.23 post with the 45 T sign, unless truck M 3 requires posting then use sign R 12-5 and use 40 T for these two trucks
For 1.23 < RF no posting required



2.3" Radius, 0.9" Border, 0.6" Indent, Black on White;
"PERMIT" E, "WEIGHT" E,
"LIMIT" E, Permit truck;
"45T" D 90% spacing;

PAGE INTENTIONALLY LEFT BLANK

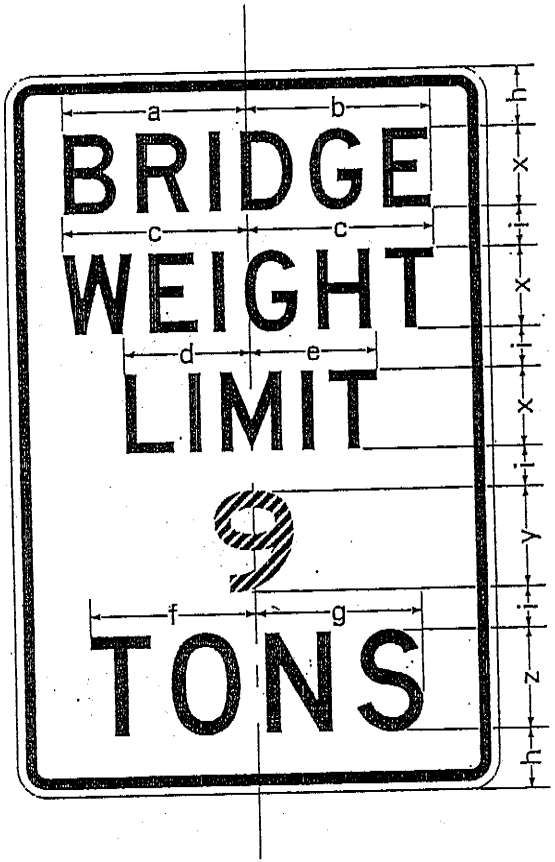
APPROVED 1/1/73

DATE OF REV.

BRIDGE WEIGHT LIMIT ___ TONS

SIGN NUMBER R12-1a

SIZE / DIMENSION	24 X 36	
RADIUS	1.5	
MARGIN	.38	
BORDER	.63	
a	9.1	
b	9.2	
c	9.2	
d	6.2	
e	6.4	
f	8.4	
g	8.2	
h	3	
i	2	
j		
k		
l		
m		
n		
o		
p		
q		
r		
s		
t		
u		
v		
w		
x	4D	
y	5E	
z	5D	



NOTES: 1. All dimensions are in inches.
2. Color - Black legend and border on white reflectorized background.
3. Appropriate tonnage to be specified.

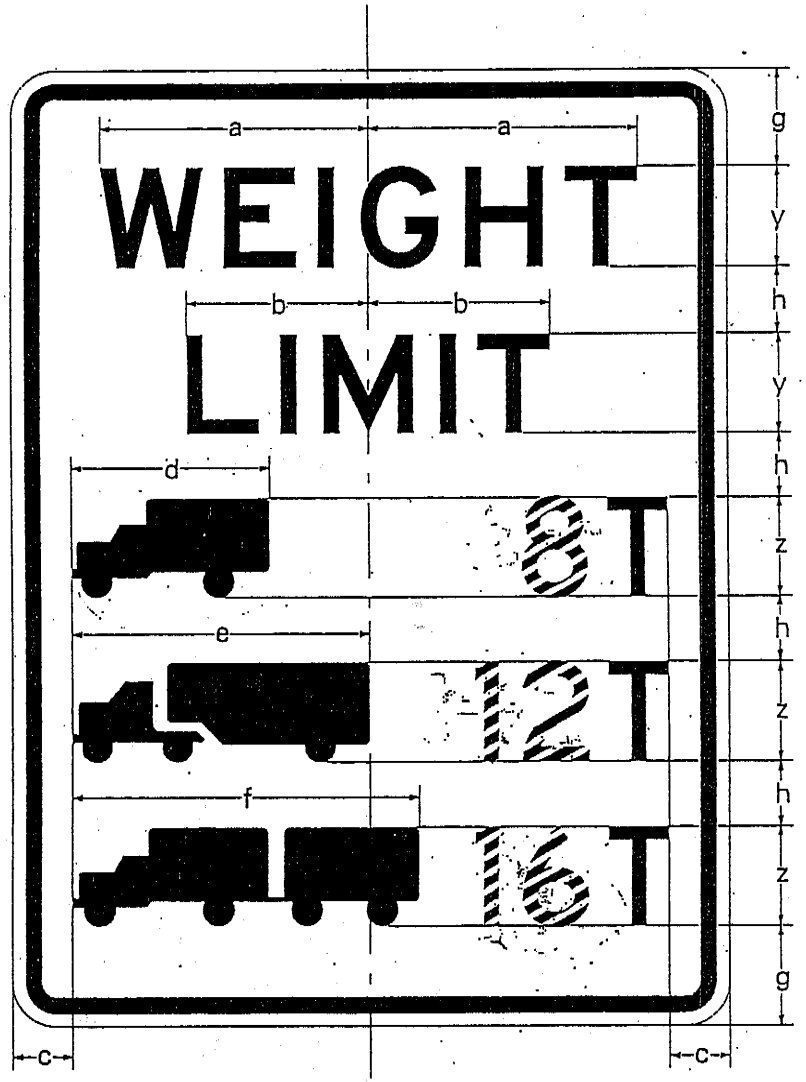
APPROVED 6/11/92

DATE OF REV. 1/1/05

WEIGHT LIMIT

SIGN NUMBER R12-5

SIZE / DIMENSION	24 X 36	30 X 42	36 X 48
RADIUS	1.5	1.88	2.25
MARGIN	.38	.5	.63
BORDER	.63	.75	.88
a	9.5	12.2	13.5
b	6.4	8.2	9.2
c	1.6	1.8	3
d	7	9	10
e	10.5	13.5	15
f	12.2	15.7	17.5
g	3.25	3.75	5
h	3	3	3.25
i			
j			
k			
l			
m			
n			
o			
p			
q			
r			
s			
t			
u			
v			
w			
x			
y	3.5E	4.5E	5E
z	3.5D	4.5D	5D



NOTES: 1. All dimensions are in inches.
2. Color - Black legend and border on white reflectorized background.
3. Appropriate numerals to be specified.

FORM RC–CL
Revised Mar. 2005

Mn/DOT BRIDGE RATING AND LOAD POSTING REPORT FOR COUNTY AND LOCAL AGENCIES

Bridge Location and Description

Bridge No. 88765

Hwy. No. MN 456 over ~~under~~ Fish Creek

Year Built 1949 Year Remodeled — Replaces Br. —

Type 106 County Adams Ref. Pt. —

Description Two simple spans.

Location 10 mi West of Dodge

Data for Basis of Report (Check all that apply)

- ☒ Bridge Inventory File
- ☒ Previous Bridge Rating and Load Posting Report
- ☐ Bridge Plans
  - ☐ New ☐ Overlay
  - ☐ Repair/Reconstruction
  - ☐ Other Dead Load Modifications
- ☒ Bridge Inspection Report, by Jones, date Apr 05
  - ☐ Damaged Component xxx next page
  - ☐ Deteriorated Component yyy " "

Type of Analysis:

☐ Manual ☐ Computer* ☐ BARS ☐ Virtis, V.: ☒ Other*

* Physical Inspection Rating

Method of Rating (Check appropriate box)

- ☐ Load Factor (LF)
- ☐ Allowable Stress (AS)
- ☐ Load & Resistance Factor (LRFR)
- ☐ Load Testing
- ☒ No Rating Computations Performed

Design Load ?

Design Method ASD

Summary of Rating and Load Posting Analysis

Load Posting Limits Posting: ☒ Required ☐ Not Required

Vehicle Type M3	Semi-Trailer Type M3S2	Truck-Full Trailer Type M3-3	Bridge Rating: Inventory Rating	Bridge Rating: Operating Rating
22 Tons (Sign)	36 Tons	36 Tons	HS 14	HS 22

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

Signature: A B Smith Date: xxx

Typed or Printed Name: A B Smith Reg. No. 56789

Example — PIR

FORM PIR Revised Mar 06	MINNESOTA DEPARTMENT OF TRANSPORTATION PHYSICAL INSPECTION RATING (Per AASHTO 7.4.1 - Manual for Condition Evaluation of Bridges)

Bridge Description

Hwy. No. MN 456 over ~~under~~ Fish Creek — Bridge No. ____

Year Built 49 — Year Remodeled — — Replaces Br. —

Type 106 — County Adams — ADT 123

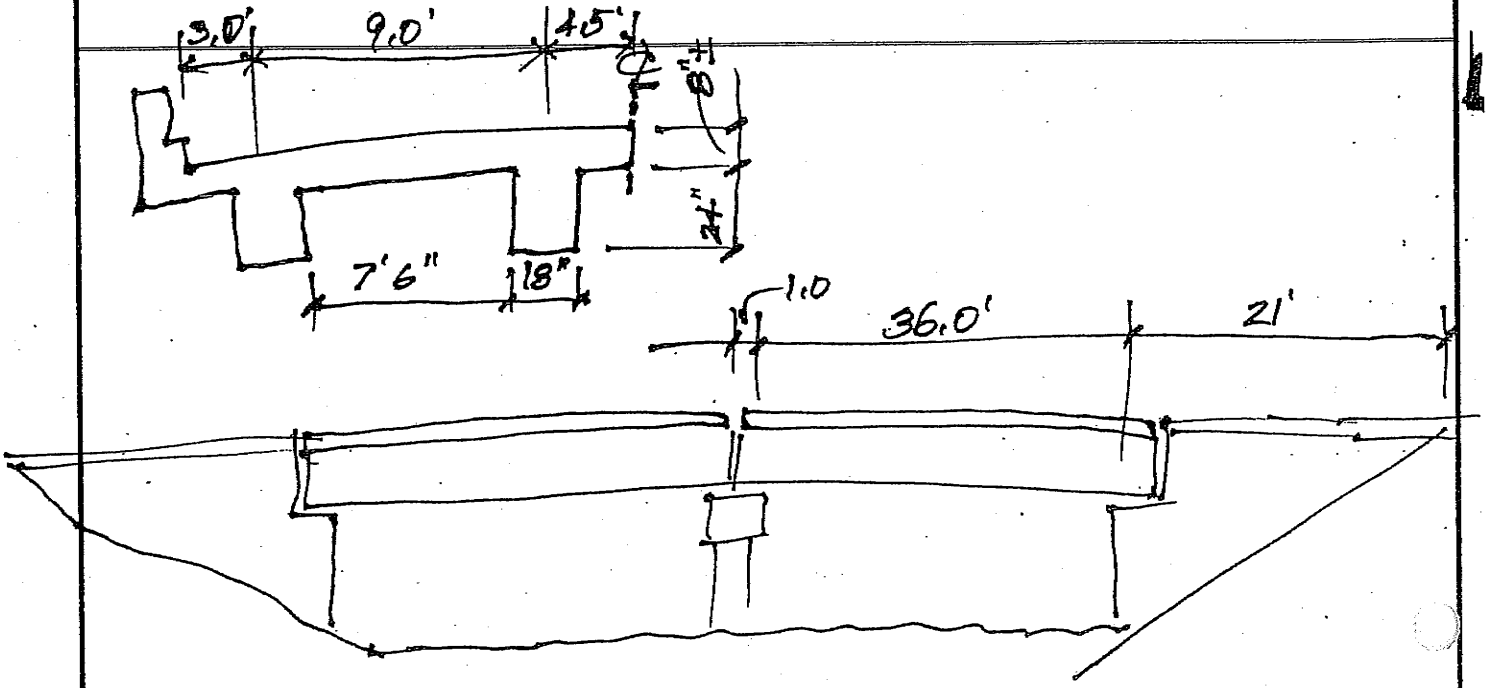
Problem leading to this physical inspection rating: No Plan.
Reinf. Conc. Uncertain of design Live Load

Describe bridge; Spans, lengths, widths, depths, deck, wearing course, etc.
Reinf Conc T Beam

Conc shaft pier -

Describe Bridge Condition:
Bottom of slab spalling, 1 to 1½ deep, approx 400 ft². Spalling near beam ends, 50% of sides & bottom within 2 ft of bearings.

Other Remarks:
Used by school bus & garbage trucks.
Substructures look OK from outside, but hollow abutments are not accessable on inside.



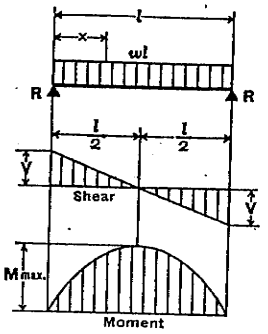
Bridge Sketch

From AISC 11th 9th ED

BEAM DIAGRAMS AND FORMULAS
For various static loading conditions

For meaning of symbols, see page 2 - 293

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . $= wl$

R = V . . . . . $= \frac{wl}{2}$

V_x . . . . . $= w\left(\frac{l}{2} - x\right)$

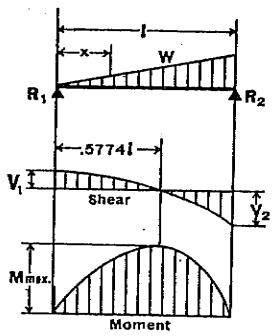
M max. (at center) . . . . $= \frac{wl^2}{8}$

M_x . . . . . $= \frac{wx}{2}(l - x)$

Δmax. (at center) . . . . $= \frac{5\,wl^4}{384\,EI}$

Δ_x . . . . . $= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

2. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO ONE END



Total Equiv. Uniform Load . . . $= \frac{16W}{9\sqrt{3}} = 1.0264W$

$R_1 = V_1$ . . . . . $= \frac{W}{3}$

$R_2 = V_2$ max. . . . . $= \frac{2W}{3}$

V_x . . . . . $= \frac{W}{3} - \frac{Wx^2}{l^2}$

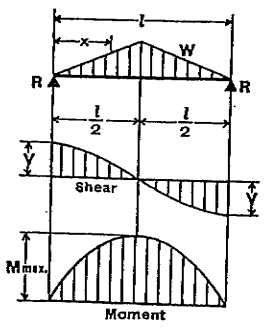
M max. $\left(\text{at } x = \frac{l}{\sqrt{3}} = .5774l\right)$ . . . $= \frac{2Wl}{9\sqrt{3}} = .1283\,Wl$

M_x . . . . . $= \frac{Wx}{3l^2}(l^2 - x^2)$

Δmax. $\left(\text{at } x = l\sqrt{1 - \sqrt{\frac{8}{15}}} = .5193l\right) = .01304\,\frac{Wl^3}{EI}$

Δ_x . . . . . $= \frac{Wx}{180EI\,l^2}(3x^4 - 10l^2x^2 + 7l^4)$

3. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO CENTER



Total Equiv. Uniform Load . . . $= \frac{4W}{3}$

R = V . . . . . $= \frac{W}{2}$

V_x $\left(\text{when } x < \frac{l}{2}\right)$ . . . . $= \frac{W}{2l^2}(l^2 - 4x^2)$

M max. (at center) . . . . $= \frac{Wl}{6}$

M_x $\left(\text{when } x < \frac{l}{2}\right)$ . . . . $= Wx\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right)$

Δmax. (at center) . . . . $= \frac{Wl^3}{60EI}$

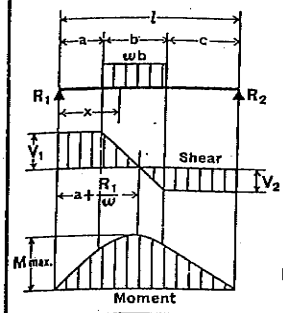
Δ_x $\left(\text{when } x < \frac{l}{2}\right)$ . . . . $= \frac{Wx}{480\,EI\,l^2}(5l^2 - 4x^2)^2$

From AISC 9th ED

BEAM DIAGRAMS AND FORMULAS
For various static loading conditions

For meaning of symbols, see page 2 - 293

4. SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED



$R_1 = V_1$ (max. when $a < c$) . . . $= \frac{wb}{2l}(2c+b)$

$R_2 = V_2$ (max. when $a > c$) . . . $= \frac{wb}{2l}(2a+b)$

V_x (when $x > a$ and $< (a+b)$) . $= R_1 - w(x-a)$

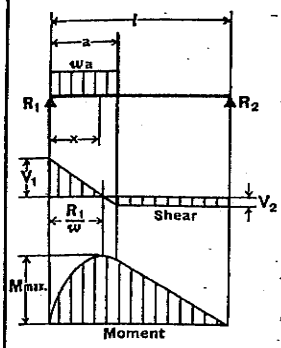
M max. (at $x = a + \frac{R_1}{w}$) . . . . . $= R_1\left(a + \frac{R_1}{2w}\right)$

M_x (when $x < a$) . . . . . $= R_1 x$

M_x (when $x > a$ and $< (a+b)$) . $= R_1 x - \frac{w}{2}(x-a)^2$

M_x (when $x > (a+b)$) . . . . $= R_2(l-x)$

5. SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED AT ONE END



$R_1 = V_1$ max. . . . . $= \frac{wa}{2l}(2l-a)$

$R_2 = V_2$ . . . . . $= \frac{wa^2}{2l}$

V_x (when $x < a$) . . . . . $= R_1 - wx$

M max. (at $x = \frac{R_1}{w}$) . . . . . $= \frac{R_1^2}{2w}$

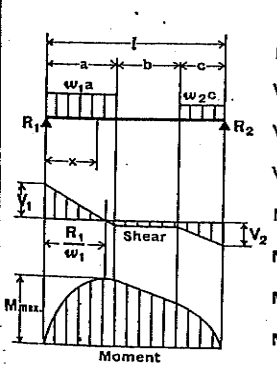
M_x (when $x < a$) . . . . . $= R_1 x - \frac{wx^2}{2}$

M_x (when $x > a$) . . . . . $= R_2(l-x)$

Δ_x (when $x < a$) . . . . . $= \frac{wx}{24EIl}\left(a^2(2l-a)^2 - 2ax^2(2l-a) + lx^3\right)$

Δ_x (when $x > a$) . . . . . $= \frac{wa^2(l-x)}{24EIl}(4xl - 2x^2 - a^2)$

6. SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED AT EACH END



$R_1 = V_1$ . . . . . $= \frac{w_1 a(2l-a) + w_2 c^2}{2l}$

$R_2 = V_2$ . . . . . $= \frac{w_2 c(2l-c) + w_1 a^2}{2l}$

V_x (when $x < a$) . . . . . $= R_1 - w_1 x$

V_x (when $x > a$ and $< (a+b)$) . $= R_1 - w_1 a$

V_x (when $x > (a+b)$) . . . . $= R_2 - w_2(l-x)$

M max. (at $x = \frac{R_1}{w_1}$ when $R_1 < w_1 a$) . $= \frac{R_1^2}{2w_1}$

M max. (at $x = l - \frac{R_2}{w_2}$ when $R_2 < w_2 c$) $= \frac{R_2^2}{2w_2}$

M_x (when $x < a$) . . . . . $= R_1 x - \frac{w_1 x^2}{2}$

M_x (when $x > a$ and $< (a+b)$) . $= R_1 x - \frac{w_1 a}{2}(2x-a)$

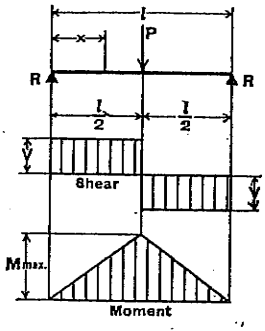
M_x (when $x > (a+b)$) . . . . . $= R_2(l-x) - \frac{w_2(l-x)^2}{2}$

From: AISC 9TH ED.

BEAM DIAGRAMS AND FORMULAS
For various static loading conditions

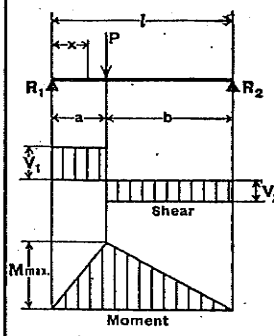
For meaning of symbols, see page 2 - 293

7. SIMPLE BEAM—CONCENTRATED LOAD AT CENTER



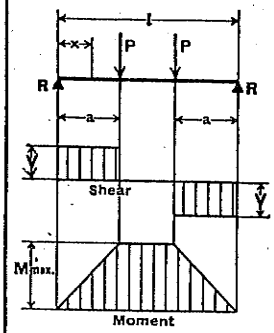
Total Equiv. Uniform Load		$= 2P$
$R = V$		$= \frac{P}{2}$
M max.	(at point of load)	$= \frac{Pl}{4}$
M_x	(when $x < \frac{l}{2}$)	$= \frac{Px}{2}$
Δmax.	(at point of load)	$= \frac{Pl^3}{48EI}$
Δ_x	(when $x < \frac{l}{2}$)	$= \frac{Px}{48EI}(3l^2 - 4x^2)$

8. SIMPLE BEAM—CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load		$= \frac{8\,Pab}{l^2}$
$R_1 = V_1$	(max. when $a < b$)	$= \frac{Pb}{l}$
$R_2 = V_2$	(max. when $a > b$)	$= \frac{Pa}{l}$
M max.	(at point of load)	$= \frac{Pab}{l}$
M_x	(when $x < a$)	$= \frac{Pbx}{l}$
Δmax.	$\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right)$	$= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27\,EI\,l}$
Δ_a	(at point of load)	$= \frac{Pa^2b^2}{3EI\,l}$
Δ_x	(when $x < a$)	$= \frac{Pbx}{6EI\,l}(l^2 - b^2 - x^2)$

9. SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



Total Equiv. Uniform Load		$= \frac{8\,Pa}{l}$
$R = V$		$= P$
M max.	(between loads)	$= Pa$
M_x	(when $x < a$)	$= Px$
Δmax.	(at center)	$= \frac{Pa}{24EI}(3l^2 - 4a^2)$
Δ_x	(when $x < a$)	$= \frac{Px}{6EI}(3la - 3a^2 - x^2)$
Δ_x	(when $x > a$ and $< (l - a)$)	$= \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$

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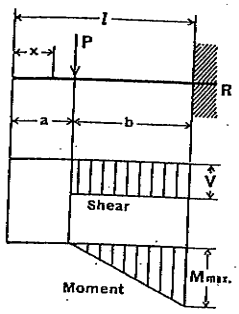
BEAM DIAGRAMS AND FORMULAS
For various static loading conditions

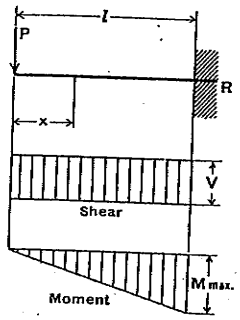
For meaning of symbols, see page 2 - 293

21. CANTILEVER BEAM—CONCENTRATED LOAD AT ANY POINT



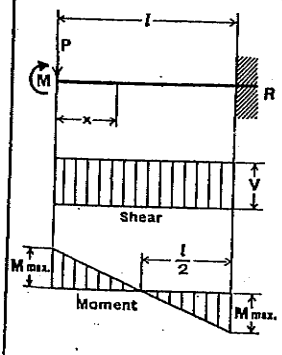
Total Equiv. Uniform Load		$= \frac{8Pb}{l}$
R = V		$= P$
M max.	(at fixed end)	$= Pb$
M_x	(when $x > a$)	$= P(x-a)$
Δmax.	(at free end)	$= \frac{Pb^2}{6EI}(3l-b)$
Δa	(at point of load)	$= \frac{Pb^3}{3EI}$
Δx	(when $x < a$)	$= \frac{Pb^2}{6EI}(3l-3x-b)$
Δx	(when $x > a$)	$= \frac{P(l-x)^2}{6EI}(3b-l+x)$

22. CANTILEVER BEAM—CONCENTRATED LOAD AT FREE END



Total Equiv. Uniform Load		$= 8P$
R = V		$= P$
M max.	(at fixed end)	$= Pl$
M_x		$= Px$
Δmax.	(at free end)	$= \frac{Pl^3}{3EI}$
Δx		$= \frac{P}{6EI}(2l^3-3l^2x+x^3)$

23. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER—CONCENTRATED LOAD AT DEFLECTED END



Total Equiv. Uniform Load		$= 4P$
R = V		$= P$
M max.	(at both ends)	$= \frac{Pl}{2}$
M_x		$= P\left(\frac{l}{2}-x\right)$
Δmax.	(at deflected end)	$= \frac{Pl^3}{12EI}$
Δx		$= \frac{P(l-x)^2}{12EI}(l+2x)$

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BEAM DIAGRAMS AND FORMULAS
For various static loading conditions

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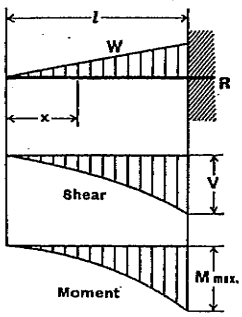
18. CANTILEVER BEAM—LOAD INCREASING UNIFORMLY TO FIXED END



Total Equiv. Uniform Load . . . . $= \frac{8}{3}W$

$R = V$ . . . . . $= W$

V_x . . . . . $= W\frac{x^2}{l^2}$

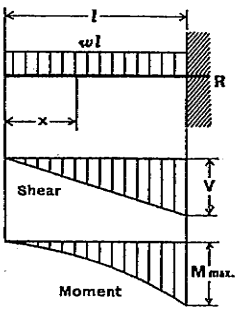
M max. (at fixed end) . . . . . $= \frac{Wl}{3}$

M_x . . . . . $= \frac{Wx^3}{3l^2}$

Δmax. (at free end) . . . . . $= \frac{Wl^3}{15EI}$

Δ_x . . . . . $= \frac{W}{60EIl^2}(x^5 - 5l^4x + 4l^5)$

19. CANTILEVER BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . $= 4wl$

$R = V$ . . . . . $= wl$

V_x . . . . . $= wx$

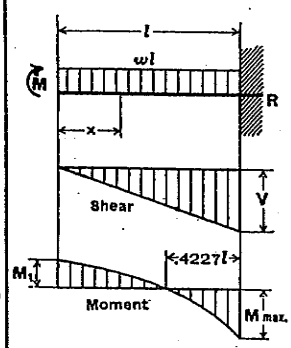
M max. (at fixed end) . . . . . $= \frac{wl^2}{2}$

M_x . . . . . $= \frac{wx^2}{2}$

Δmax. (at free end) . . . . . $= \frac{wl^4}{8EI}$

Δ_x . . . . . $= \frac{w}{24EI}(x^4 - 4l^3x + 3l^4)$

20. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . $= \frac{8}{3}wl$

$R = V$ . . . . . $= wl$

V_x . . . . . $= wx$

M max. (at fixed end) . . . . . $= \frac{wl^2}{3}$

M_1 (at deflected end) . . . . $= \frac{wl^2}{6}$

M_x . . . . . $= \frac{w}{6}(l^2 - 3x^2)$

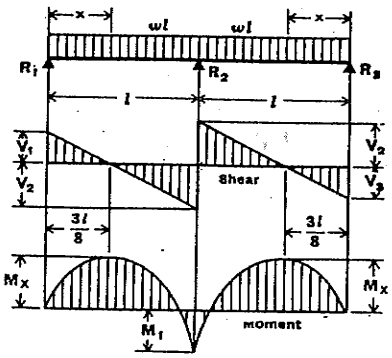
Δmax. (at deflected end) . . . . $= \frac{wl^4}{24EI}$

Δ_x . . . . . $= \frac{w(l^2 - x^2)^2}{24EI}$

AISC 4TH ED.

TABLE 8.13 (*Continued*)

29. CONTINUOUS BEAM—TWO EQUAL SPANS—UNIFORM LOAD ON BOTH SPANS



$R_1 = V_1 = R_3 = V_3 \ldots = \frac{3}{8}\,wl$

$R_2 = 2V_2 \ldots = \frac{10}{8}\,wl$

$V_2 \ldots = \frac{5}{8}\,wl$

$M_x \ldots = R_1x - \frac{wx^2}{2}$

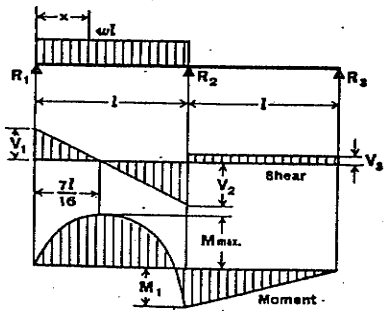
$M_x \left(\text{at } x = \frac{3l}{8}\right) \ldots = \frac{9}{128}\,wl^2$

M_1 (at support R_2) . . $= -\frac{wl^2}{8}$

Δ Max. (0.4215l from R_1 or R_3) = $wl^4/185$

$\Delta_x = \frac{wx}{48EI}\,(l^3 - 3lx^2 + 2x^3)$

30. CONTINUOUS BEAM—TWO EQUAL SPANS—UNIFORM LOAD ON ONE SPAN



$R_1 = V_1 \ldots = \frac{7}{16}\,wl$

$R_2 = V_2 + V_3 \ldots = \frac{5}{8}\,wl$

$R_3 = V_3 \ldots = -\frac{1}{16}\,wl$

$V_2 \ldots = \frac{9}{16}\,wl$

M Max. $\left(\text{at } x = \frac{7}{16}\,l\right) \ldots = \frac{49}{512}\,wl^2$

M_1 (at support R_2) . . $= \frac{1}{16}\,wl^2$

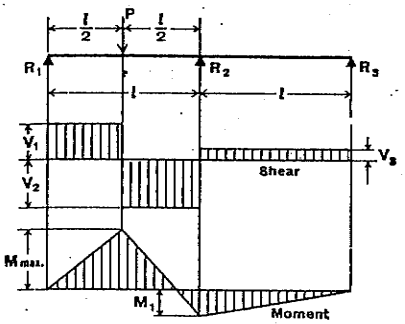
M_x (when $x < l$) . . $= \frac{wx}{16}\,(7l - 8$

Δ Max. (0.472l from R_1) = $wl^4/109EI$

AITC 4TH ED.

TABLE 8.13 (*Continued*)

31. CONTINUOUS BEAM—TWO EQUAL SPANS—CONCENTRATED LOAD AT CENTER OF ONE SPAN



$R_1 = V_1 \ldots = \frac{13}{32} P$

$R_2 = V_2 + V_3 \ldots = \frac{11}{16} P$

$R_3 = V_3 \ldots = -\frac{3}{32} P$

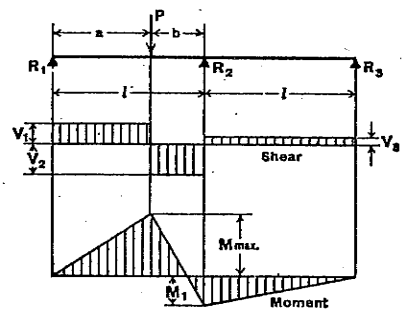
$V_2 \ldots = \frac{19}{32} P$

M Max. (at point of load) . $= \frac{13}{64} Pl$

M_1 (at support R_2) . $= \frac{3}{32} Pl$

Δ Max. (0.480 l from R_1) $= 0.015\ Pl^3/EI$

32. CONTINUOUS BEAM—TWO EQUAL SPANS—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1 \ldots = \frac{Pb}{4l^2}\left(4l^2 - a(l+a)\right)$

$R_2 = V_2 + V_3 \ldots = \frac{Pa}{2l^3}\left(2l^2 + b(l+a)\right)$

$R_3 = V_3 \ldots = -\frac{Pab}{4l^3}(l+a)$

$V_2 \ldots = \frac{Pa}{4l^3}\left(4l^2 + b(l+a)\right)$

M max. (at point of load) . $= \frac{Pab}{4l^3}\left(4l^2 - a(l+a)\right)$

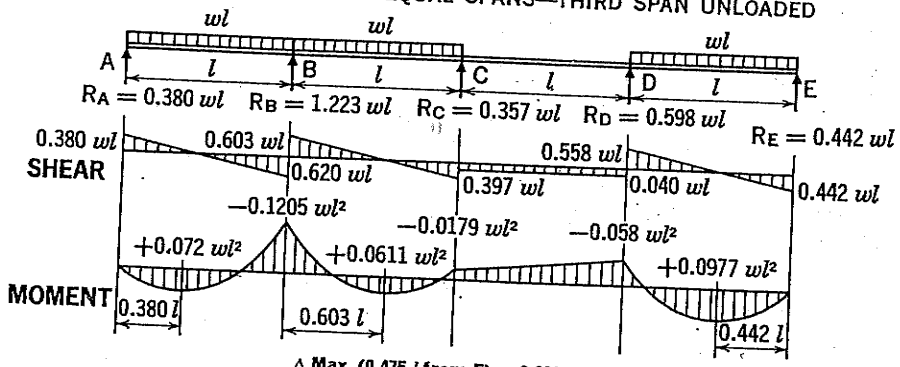
M_1 (at support R_2) . $= \frac{Pab}{4l^2}(l+a)$

AISC 9TH ED.

BEAM DIAGRAMS AND DEFLECTIONS
For various static loading conditions

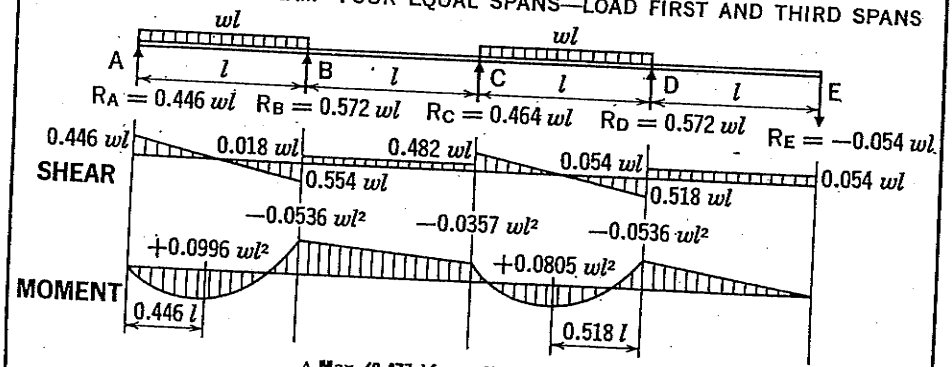
For meaning of symbols, see page 2 - 293

37. CONTINUOUS BEAM—FOUR EQUAL SPANS—THIRD SPAN UNLOADED



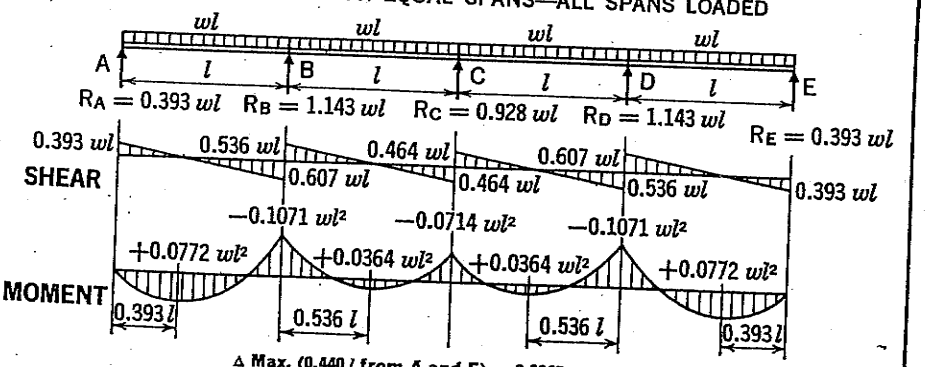
Δ Max. (0.475 *l* from E) = 0.0094 wl^4/EI

38. CONTINUOUS BEAM—FOUR EQUAL SPANS—LOAD FIRST AND THIRD SPANS



Δ Max. (0.477 *l* from A) = 0.0097 wl^4/EI

39. CONTINUOUS BEAM—FOUR EQUAL SPANS—ALL SPANS LOADED

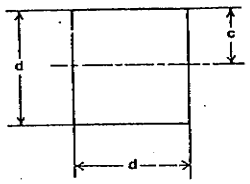


Δ Max. (0.440 *l* from A and E) = 0.0065 wl^4/EI

AISC "ASD" 9TH ED. GREENBOOK

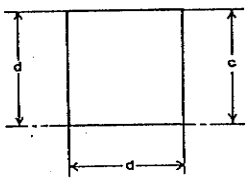
PROPERTIES OF GEOMETRIC SECTIONS

SQUARE
Axis of moments through center



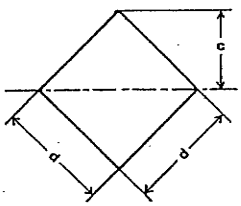
$$A = d^2$$
$$c = \frac{d}{2}$$
$$I = \frac{d^4}{12}$$
$$S = \frac{d^3}{6}$$
$$r = \frac{d}{\sqrt{12}} = .288675\ d$$
$$Z = \frac{d^3}{4}$$

SQUARE
Axis of moments on base



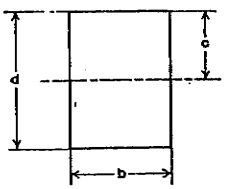
$$A = d^2$$
$$c = d$$
$$I = \frac{d^4}{3}$$
$$S = \frac{d^3}{3}$$
$$r = \frac{d}{\sqrt{3}} = .577350\ d$$

SQUARE
Axis of moments on diagonal



$$A = d^2$$
$$c = \frac{d}{\sqrt{2}} = .707107\ d$$
$$I = \frac{d^4}{12}$$
$$S = \frac{d^3}{6\sqrt{2}} = .117851\ d^3$$
$$r = \frac{d}{\sqrt{12}} = .288675\ d$$
$$Z = \frac{2c^3}{3} = \frac{d^3}{3\sqrt{2}} = .235702d^3$$

RECTANGLE
Axis of moments through center



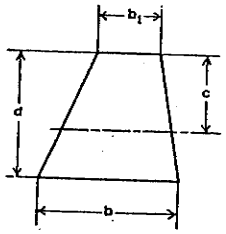
$$A = bd$$
$$c = \frac{d}{2}$$
$$I = \frac{bd^3}{12}$$
$$S = \frac{bd^2}{6}$$
$$r = \frac{d}{\sqrt{12}} = .288675\ d$$
$$Z = \frac{bd^2}{4}$$

AISC 11th ED. 9th ED. GRAYBOOK

PROPERTIES OF GEOMETRIC SECTIONS

TRAPEZOID

Axis of moments through center of gravity



$$A = \frac{d(b + b_1)}{2}$$

$$c = \frac{d(2b + b_1)}{3(b + b_1)}$$

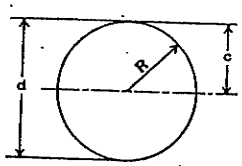
$$I = \frac{d^3 (b^2 + 4bb_1 + b_1^2)}{36 (b + b_1)}$$

$$S = \frac{d^2 (b^2 + 4bb_1 + b_1^2)}{12 (2b + b_1)}$$

$$r = \frac{d}{6(b + b_1)} \sqrt{2 (b^2 + 4bb_1 + b_1^2)}$$

CIRCLE

Axis of moments through center



$$A = \frac{\pi d^2}{4} = \pi R^2 \quad .785398\ d^2 = 3.141593\ R^2$$

$$c = \frac{d}{2} = R$$

$$I = \frac{\pi d^4}{64} = \frac{\pi R^4}{4} = .049087\ d^4 = .785398\ R^4$$

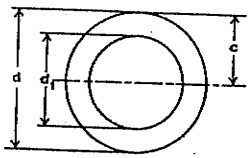
$$S = \frac{\pi d^3}{32} = \frac{\pi R^3}{4} = .098175\ d^3 = .785398\ R^3$$

$$r = \frac{d}{4} = \frac{R}{2}$$

$$Z = \frac{d^3}{6}$$

HOLLOW CIRCLE

Axis of moments through center



$$A = \frac{\pi(d^2 - d_1^2)}{4} = .785398\ (d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087\ (d^4 - d_1^4)$$

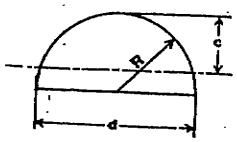
$$S = \frac{\pi(d^4 - d_1^4)}{32d} = .098175\ \frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$

HALF CIRCLE

Axis of moments through center of gravity



$$A = \frac{\pi R^2}{2} = 1.570796\ R^2$$

$$c = R\left(1 - \frac{4}{3\pi}\right) = .575587\ R$$

$$I = R^4\left(\frac{\pi}{8} - \frac{8}{9\pi}\right) = .109757\ R^4$$

$$S = \frac{R^3}{24}\frac{(9\pi^2 - 64)}{(3\pi - 4)} = .190687\ R^3$$

$$r = R\frac{\sqrt{9\pi^2 - 64}}{6\pi} = .264336\ R$$

SECTION MODULUS FORMULA FOR TIMBER SOLID RECTANGULAR & CIRCULAR SHAPES WITH 'CENTER ROT"

Washington State Department of Transportation

Load Capacity Calculation for Timber Stringers

Agency	Calculated By
Bridge Number	Checked By
Bridge Name	Date

Necessary Information

Stringers: ■ Base (Inches) _____ Height (Inches) _____

● Butt (Inches) _____ Tip (Inches) _____

Material: Douglas Fir _____ Cedar _____ Hem Fir _____

Grade: Select Struct. _____ No. 1 _____

No. of Lines of Stringers _____ Typical Spacing Ctr. to Ctr. (Feet) _____

Max Span Length Ctr. to Ctr. of Bearings (Feet) _____

Deck: Type _____ Thickness (Inches) _____ , Roadway Width (Feet) _____

Surfacing: Type _____ Thickness (Inches) _____

Describe Stringer Section Loss: _____

Calculations Per Individual Stringer

Dead Load:

Stringers: ■ H (Inches) _____ x B (Inches) _____ x 0.35

● ↕ Ave. D (Inches) (_____)^2 x 0.27 = _____ lb./ft.

Deck: Timber Thickness _____ (inches) x 4.17

Concrete Thickness _____ (inches) x 12.50

Corrugated Steel: (Factor includes wt. of asphalt to top of corrugations)

Gage	Thickness	Factor
7	.179	21.2
10	.135	18.5
12	.105	16.7

x _____ / Stringer Spacing (ft.) = _____ lb./ft.

Surfacing: (Bituminous) Thickness _____ (inches) x 11.67

(Gravel) Thickness _____ (inches) x 9.17

x _____ / Stringer Spacing (ft.) = _____ lb./ft.

Total DeadLoad _____ lb./ft.

Dead Load Moment: (_____ Total D.L. (lb./ft.)) x (_____ Span Length (ft.))^2 x 0.000125 = _____ K-ft.

Stringer Capacity:

Section Modulus _____ = _____ in.^{3}

Choose Formula From Below
(All dimensions in inches)

B ↔

↕ H Solid Rectangular; $\frac{BH^2}{6}$

↕ H_1 Top rot above Neutral Axis (N.A.); Approx. $\frac{B\left(H-\frac{H_1}{3}\right)^2}{6}$

↕ H_2 Center rot symmetrical about N.A.; $\frac{BH^3 - B_2H_2^3}{6H}$

B_2

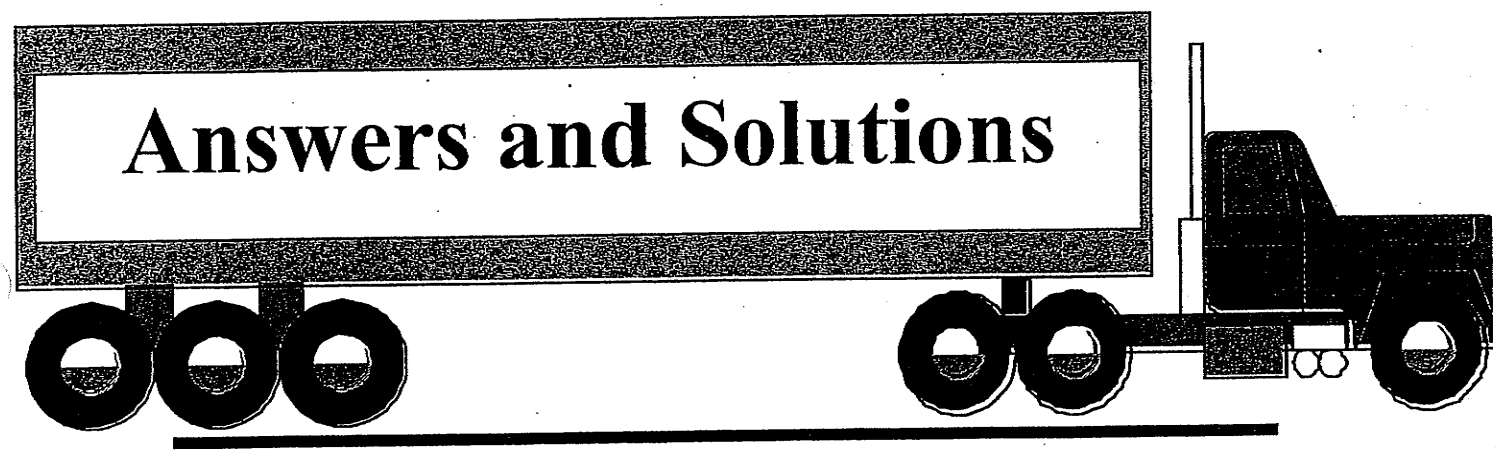
○ ↕ D Solid Circular; Approx. $\frac{D^3}{10}$

● ↕ D_1 Center rot symmetrical about N.A.; Approx. $\frac{D^4 - D_1^4}{10D}$

DOT Form 148-086 EF
Revised 2/98

2005/2006

Minnesota Truck Weight Education Project



Answers and Solutions

An educational class to enhance knowledge and voluntary compliance of Minnesota's truck weight limits.

NORTHLAND COMMUNITY & TECHNICAL COLLEGE

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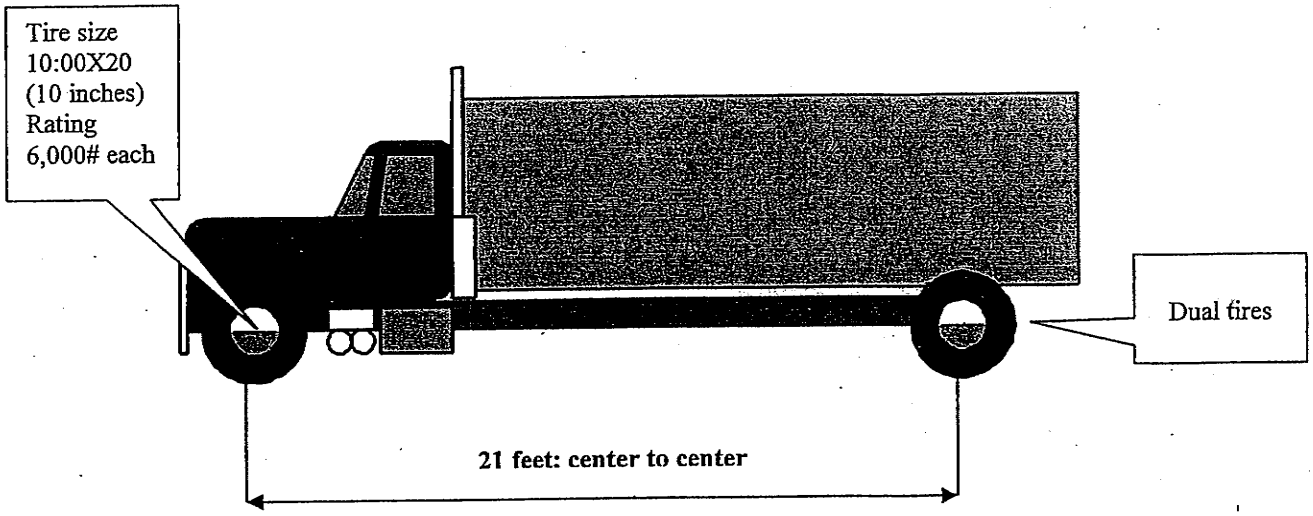

Allowable maximum weight from the gross weight chart? (2 axles spaced 21 feet apart)

9 ton route = **36,000 Pounds**

10 ton route=**40,000 Pounds**

Steering axle weight (both tires)? Weight by size = **12,000 #** Weight by rating = **12,000 #**

Rear single axle weight? (9 ton) **18,000 # (found on chart "H")**

Rear single axle weight? (10 ton) **20,000 # (found on chart "H")**

Actual legal weight? 9 ton =**30,000 #** 10 ton = **32,000 #**
(steering axle weight + rear single axle weight)

Restricted Weights (See Chart "I")

5 ton road? Steering = **10,000 #** Rear = **10,000 #**

7 ton road? Steering = **12,000 #** (can't exceed rating) Rear = **14,000 #**

10% Increase

Steering axle? **12,000 #** (can't exceed rating)

Rear axle (9 ton)? **19,800#** (max gross weight 31,800#)

Rear axle (10 ton)? **22,000#** (max gross 34,000#)

Registration increase?

Need a transportation permit?

A

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Solutions "A"

Truck "A" Two axle

The maximum allowable weight is 40,000 pounds for a 10 ton route and 36,000 pounds for a 9 ton route. This is computed by looking at the gross weight table, going down the left side to the 21 foot measurement and looking across the top, go to the 2 axle column and go down. Note 36,000 (first column) or 40,000 pounds (10 ton column) ends at a 10 foot spacing. Any measurement beyond 10 feet also is 36,000 or 40,000 pounds as it is the maximum two axles can weigh (18,000 + 18,000) or (20,000 + 20,000) depending on 9 or 10 ton route.

The steering axle weight (note chart on the backside of the gross weight table) is 12,000 pounds. This is computed by taking the individual tire rating (6000 pounds) times 2 (2 tires) which equals 12,000 pounds (assuming neither tire exceeds the 6,000 limit.) Also the size (a 10 inch tires) needs to be multiplied by the 600# per inch of tire size. 600 pounds times 10 inches equals 6,000 pounds per tire (see the tire weight chart on the backside of the gross weight chart.) There are two tires on the steering axle, so the maximum there is also 12,000 pounds. NOTE: The legal weight would be the less of the two computations in the event they are not the same. (example a tire rated at 5,000 pounds per tire would limit the steering axle weight to 10,000 pound even though the 600# per inch of tire surface would allow 12,000. The "less" applies.

The single axle weight for a 9 ton route is 18,000 pounds.
The single axle weight for a 10 ton route is 20,000 pounds.

In this example, it is assumed each tire in the set of duels is a minimum 10 inch tire. Since a non-steering tire is only allowed 500# of weight per tire inch, this would allow 5,000 pounds for each tire on this truck. Since there are 4 tires on the axle the maximum weight would be 5,000# X 4 = 20,000 pounds. Tires smaller than 10 inches need to be monitored closely as it would be likely they could reduce the weight on a single axle.

The actual weight of Truck A is 32,000 pounds on a 10 ton route. (12,000 + 20,000 = 32,000 pounds)
The actual weight of Truck A is 30,000 pounds on a 9 ton route. (12,000 + 18,000 = 30,000 pounds)

Note how using only the Gross Weight Table would not give you an accurate "legal" weight as this truck loaded at 40,000 pounds would be overweight somewhere, most likely the rear "drive" axle.

<u>Restricted Weights.</u>

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Looking at Chart "I" a 5 ton single axle is limited to 10,000 pounds. This truck would have to obey this regulation on both axles, Thus the maximum allowable weight could not exceed 10,000 pounds on the steering, and 10,000 pounds on the single axle. A gross weight over 20,000 pounds would indicate an overweight somewhere.
On a 7 ton chart a single axle is limited to 14,000 pounds. The steering axle cannot weigh more than 12,000 pounds due to tire size and rating. It cannot be loaded to the allowed 14,000 pounds on the steering axle. The second axle is limited to 14,000 pounds (7 tons) thus the maximum allowed weight on a 7 ton road would be 26,000 pounds. (12,000 steering, 14,000 drive axle – 26,000 pounds.

<u>10% Increased Weights (Harvest and Winter)</u>

The steering axle can only weigh 12,000 pounds due to the ratings of the two tires. The 10% weight increase would allow another 600 pounds per tire (6600 each) ONLY if it does not exceed the ratings. In this case the ratings would be exceeded so the steering axle will not be allowed to use the increase.

The second axle (drive axle) on a 9 ton route would be exceeded by 10% allowing and additional 1800 pounds making the maximum allowable weight 19,800 pounds. The maximum allowable gross weight would be the 12,000 steering weight added to the increased drive axle weight 19,800 pounds which would allow 31,800 pounds gross weight.
On a 10 ton route, the steering axle would still be allowed 12,000 pounds. The drive axle would be allowed 22,000 pounds providing the tire rating would not be exceeded on the duels. (Note, if the tires are rated at 5,000 pounds each, this truck could not use the 10% at they would have a maximum rating of 20,000 pounds) Assuming the weight rating on the rear axles allow additional weight, the maximum allowable gross weight would be 12,000 + 22,000 = 34,000 Pounds.

A registration increase would only be necessary if the new increased gross weight exceeds the current registered weight by 1,000 pounds or 4% (whichever is greater)

A State permit would be needed in a harvest situation if the truck is operated on a State truck highway (no interstate highways).
A State permit would be needed in a winter weight increase ONLY if the truck is operated on a interstate highway.

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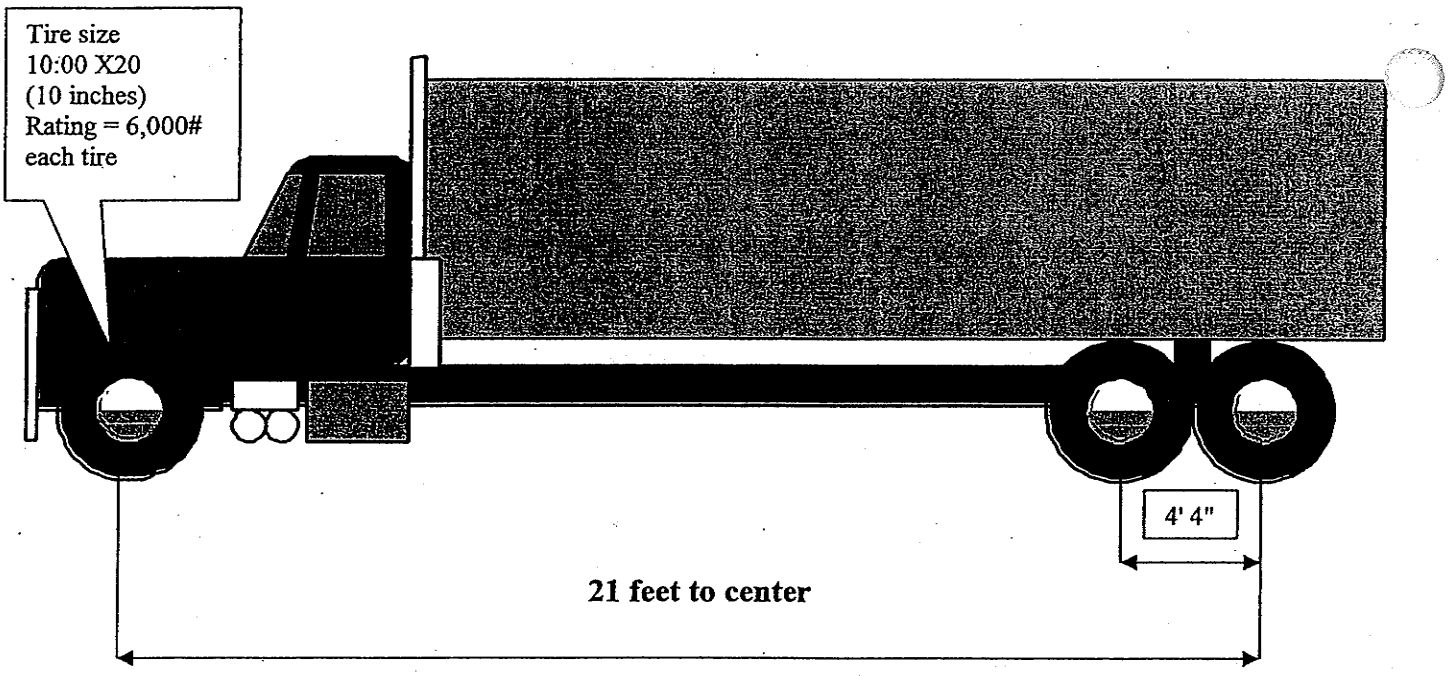

Allowable maximum weight from the gross weight chart = **52,000 #**

Allowable steering axle weight? Weight by size = **12,000#** Weight by rating = **12,000#**

Tandem weight? (9 ton) **34,000 #** Max. axle weight in tandem? **18,000#**

Tandem weight? (10 ton) **34,000#** Max. axle weight in tandem? **20,000#**

Actual legal weight? **46,000#**
(steering weight + tandem weight)

Restricted Weights (See chart "I")

5 ton road? Steering = **10,000#** Tandem = **18,889#** (max gross 28,889#)

7 ton road? Steering = **12,000#** (can't exceed rating) Tandem = **26,444#** (max gross 38,444#)

10% Increase

Steering axle? **12,000#** (can't exceed rating)

Tandem axle (9 ton)? **37,400#** (max gross 49,400#)

Tandem axle (10 ton)? **37,400#** (max gross 49,400#)

Registration increase?

Need a transportation permit?

B

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Solutions "B"

Truck "B" Three Axle

The maximum allowable weight is computed by going to the gross weight chart. Go down the left side to 21 feet and follow across to the column with 3 axles. You should find the weight 52,000. This is the maximum allowable weight for this 3 axle truck.
The steering axle weight is exactly the same as truck "A" or 12,000 pounds. (same computations)
The tandem weight is determined by noting the distance between the last two axles (4 feet 4 inches). Again go to the gross weight chart, on the left note 4 feet, go across to the 2 axle columns (the first one is for 9 ton, the second one (in a red box) is for 10 ton routes. Note that in both cases, two axles spaced 4 feet apart allows 34,000 pounds.
In both 9 and 10 ton transportation routes, the maximum legal weight is 34,000 pounds. The only difference is that on a 9 ton route neither of the individual axles in the tandem can exceed 18,000 pounds, and on a 10 ton route neither can exceed 20,000 pounds. Normally where axles have the same tire size, this is not an issue.

The actual "legal" weight for this truck would be the total of the steering and tandem weights 12,000 + 34,000 = 46,000 pounds. (on both 9 and 10 ton routes)

<u>Restricted Weights</u>.

Looking at chart "I" the steering axle situation is the same as truck "A". The maximum allowed is 12,000 pounds which is more than the 5 ton road limit of 10,000 pounds. 10,000 pounds would be the maximum allowable weight on the steering axle on a 5 ton road. The tandem would be limited by the chart to 18,889 pounds. (found in the "two axles spaced within 8 feet or less column.)
The total weight of this truck on a 5 ton road would be the sum of the allowable steering weight 10,000 plus the allowable tandem axle weight of 18,889. Total being 28,889 pounds. Any gross weight over that amount would indicate an overweight somewhere on the truck.

On a 7 ton route, the maximum allowable weight on the steering axle would be 14,000 pounds which would exceed the tire rating for the steering axle. So, the maximum weight on the steering axle would remain.at the rated 12,000 pounds. The tandem axle would be found on chart "I" as allowing 26,444 pounds. The maximum allowable weight of this truck on a 7 ton road would be the sum of the two being 38,444 pounds. Any weight over that amount would indicate an overweight somewhere on the truck.

<u>10% Weight Increases.</u>

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As with truck 'A' the steering axle is at the maximum legal due to the tire ratings. The steering axle would not be able to be increased 10% because it would exceed the ratings. The tandem, assuming no tire rating issues) could be increased by 10% or 3400 pounds. This would make the maximum allowable tandem weight 37,400 pounds. Add to that the steering axle weight (12,000 pounds) and you have the maximum allowable weight for the 10% increase at 49,400 pounds.

In some rare cases, it would be possible to increase the axle weights more than the table weights on the gross weight table would allow. It's always important to make sure that you don't increase your individual axle weights beyond the 10% of any table weight. In this case the allowable weight is 49,400 pounds, far less than the possible allowable of 57,200 pounds you get when you simple add 10% to the table of 3 axles spaced 21 feet apart (which was the original 52,000 pounds).

<u>Registration Increase?</u>
If this truck was registered for 45,000 pounds ("M" sticker on license plate) you would immediately note the new weight could be 49,400 pounds. The State law allows you either 1,000 pounds, or 4% (whichever is greater) for you to exceed your registration weight without the need to increase. 4% of 45,000 pounds is 1800 pounds which is larger than 1,000. So if your truck exceeds a weight of 46,800 pounds it would be in violation of the registration law. You would need to increase your registration to the next increase and get an "N" sticker on your license plate which would allow 51,000 pounds.

Permit?
Same as truck "A". If you operate it during harvest on a state highway you need a permit to exceed otherwise legal weights. These are not valid on Interstate routes. If you operate during the winter load increase you would not need a permit for state highways ONLY interstate routes.
As always, city, county, and townships have their own processes which must be investigated before using their routes.

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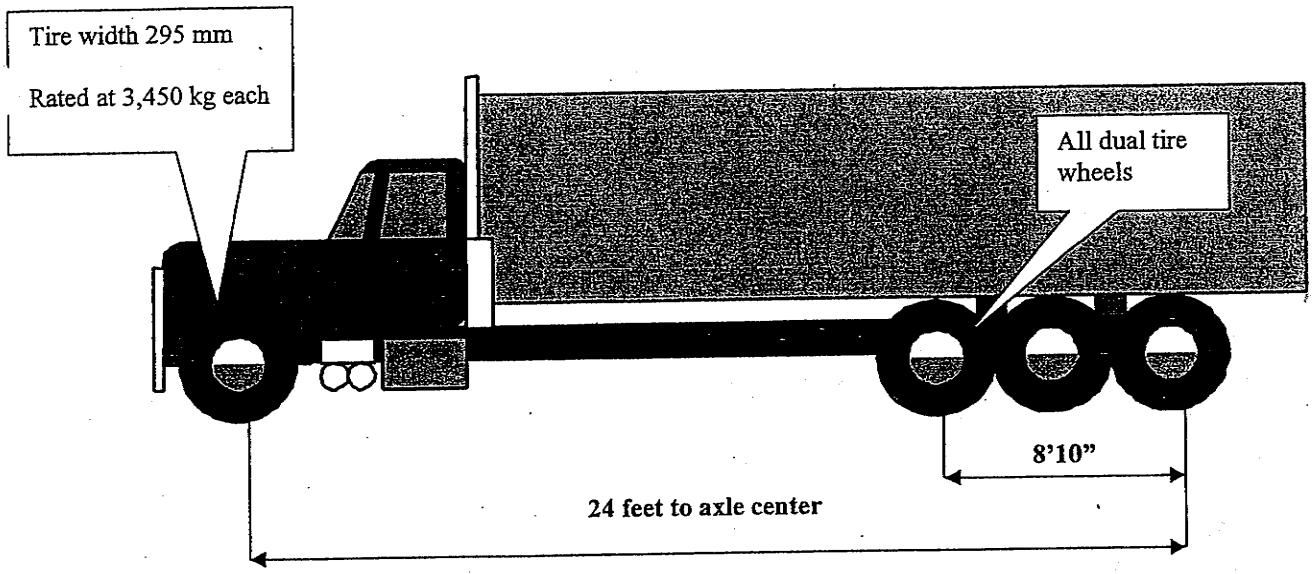

Allowable maximum weight from the gross weight chart? **58,000#** (4 axles spaced 24 feet)

Allowable steering weight? By size = **13,920#** (exact 13,932#) by rating = **15,180#**

Is the rear group a tridem (**Y** or N?)Allowable rear axle group weight from the chart? **42,000#** (this is an 8+ measurement)

What are the allowable maximum axle weights within tridem?
#1= **15,000#** #2= **15,000#** #3= **15,000#** (see notes on the bottom of chart "H")

Actual legal weight? = **55,920#**
(steering + rear axles (tridem)

Restricted Axle Weights

5 ton road? Steering = **10,000#** Tridem = **23,889#** (max gross 33,889#)

7 ton road? Steering = **13,920#** Tridem = **33,444#** (max gross 47,364#)

10% Increase

Steering axle? **15,180# (rating)**...note, 10% of the size went over the rating...

Tridem axle (weight chart w/ increase)? **46,200#**

Allowable (10%) axle weight within tridem?

#1 = **16,500#** #2 = **16,500#** #3 = **16,500#**

Actual 10% increased gross weight for this truck? **61,380#**

Registration increase?

Need a transportation permit?

Note: To convert millimeters (mm) to inches , divide mm by 25.4 = inches

$\frac{\text{mm}}{25.4}$ = inches

example: $\frac{295 \text{ mm}}{25.4} = 11.6$

Also: To convert kilograms (kg) to pounds; multiple by 2.2

example: 3,450 kg X 2.2 = 7,590 lbs

C

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Solutions "C"

Truck "C" Four axle (all duel tire rear axles)

The maximum allowable weight is computed by going to the gross weight chart. Go down the left side to 24 feet (the distance between the first and last axle of the truck), then go across to column 4 (which is the number of axles in the measurement) and you will find 58,000 pounds. This is the maximum allowable weight for this truck if the 58,000 pounds were to be legally distributed within the maximum allowable weights of the axles in the measurement (no overweight axles).

The steering axle weight is noted as "metric". In order to find the maximum allowable weight (noted on the Minnesota tire weight chart on the backside of the gross weight chart), it is necessary to convert metric size to inches. Note the text box on sheet "C". Since the metric tire size is 295mm, that number must be divided by 25.4 to find the measurement in inches. That number is 11.614 inches. (note the backside of the gross weight chart shows this as rounded to 11.6 inches). Since the steering axle is allowed 600 pounds of weight per inch of tire size we must multiply the 11.6 by 600 to get the weight per tire. That weight is 6960 pounds per tire. Assuming both tires are the same size, this weight 6960 multiplied by 2 gives the maximum allowable weight (13,920#) on the steering axle unless the rating is less. The tire rating is 3450 kilograms per tire. This also should be computed to "pounds" to determine if it exceeds the allowable 600 pounds per inch weight. To compute the metric to pounds take the 3450kg times 2.2 which converts the metric 3450kg to 7590 pounds per tire. This weight in pounds (7590) multiplied by 2, gives the maximum allowable "rated" weight as 15,180 pounds.
The tire rating allowable weight is 15,180 pounds.
The tire weight per inch allowable weight is 13,920.
13,920 is the lesser thus is the maximum allowable weight of the steering axle on this truck.

The last three axles on the truck are spaced 8 feet 10 inches apart. This is a tridem (3 axles spaced 9 feet or less). The measurement is over 8 feet yet less than 9 feet so it falls in the distance column as "8 plus".
Go down the left side of the gross weight chart to the "8 plus" and follow over to the right to the 3 axle column (number of axles in this measurement). The legal weight of the tridem is 42,000 pounds. Note, since this is a tridem, no axle within the tridem 3 axle combination can exceed 15,000 pounds. (though the maximum cannot be 45,000 pounds due to the "8 plus" measurement which only allows 42,000 pounds).

The actual legal weight of this truck is the steering axle maximum allowable weight (13,920 pounds) added to the maximum allowable tridem weight (42,000 pounds) which totals 55,920 pounds. The 58,000 pounds which the chart allows for the 24

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foot measurement cannot be attained because of the steering axle tire size, and possible tridem limitations.

Restricted Weights

Look at the restricted gross weight table chart "I".
The steering axle weight on a 5 ton road would be 10,000 pounds. On a 7 ton road it would be 14,000 pounds. Since the maximum legal weight on the steering axle is 13,920 pounds adjustments would have to be made on the 5 ton routes only.
The tridem axle would be restricted to 23,889 on a 5 ton route, and 33,444 on a 7 ton route. Note that the chart addresses the axles in the tridem as a group, not 3 single axles. (3 non-tridem single axles would be allowed 14,000 [@ 7 ton] each equaling 42,000 pounds...this is not a correct way to determine restricted weight in a tridem).
5 ton 10,000 steering added to 23,889 tridem allows 33,889 maximum allowable.
7 ton 13,920 steering added to 33,444 tridem allows 47,364 maximum allowable.
Gross weights in excess of these would indicate an overweight somewhere on the vehicle.

10% Weight Increases

The maximum allowable (non 10% increase) steering axle weight is 13,920 pounds. Add to that an additional 10% (1,392 pounds) computes to 15,312 pounds. The rated weight computed to 15,180 pounds. The 10% increase would be allowed up to the rated weight of 15,180. (only 132 pounds short of the 10% allowed).
The 10% steering axle weight would be the rating of 15,180 pounds.

The 10% tridem axle weight would be 46,200#.
The maximum allowable (non 10% increase) tridem weight is 42,000 pounds. Add to that an additional 10% (4200 pounds) computes to 46,200 pounds. The individual axle weights within the tridem are increased also to 16,500 pounds. (15000 + 1500 = 16,500). Again watch that tire ratings are not exceeded.
15,180 pounds on steering axle added to the 46,200 tridem allows 61,380 possible as a gross weight. (note how you cannot just take normal allowable weights and increase them by 10%).
Registration Increase & permit?

If the registered weight on the cab card is exceeded by more than 1000 pounds or 4% (whichever is greater) the registered weight (license plate weight) must be increased to the weight hauled. This is always true.
A transportation permit is needed during harvest for any state highway but not valid on an Interstate highway.
During the winter weight increase a transportation permit is needed ONLY on an Interstate highway. (I-94, I-35 etc).
Always check with local agencies (city, county and township) prior to increasing any weights on their routes.

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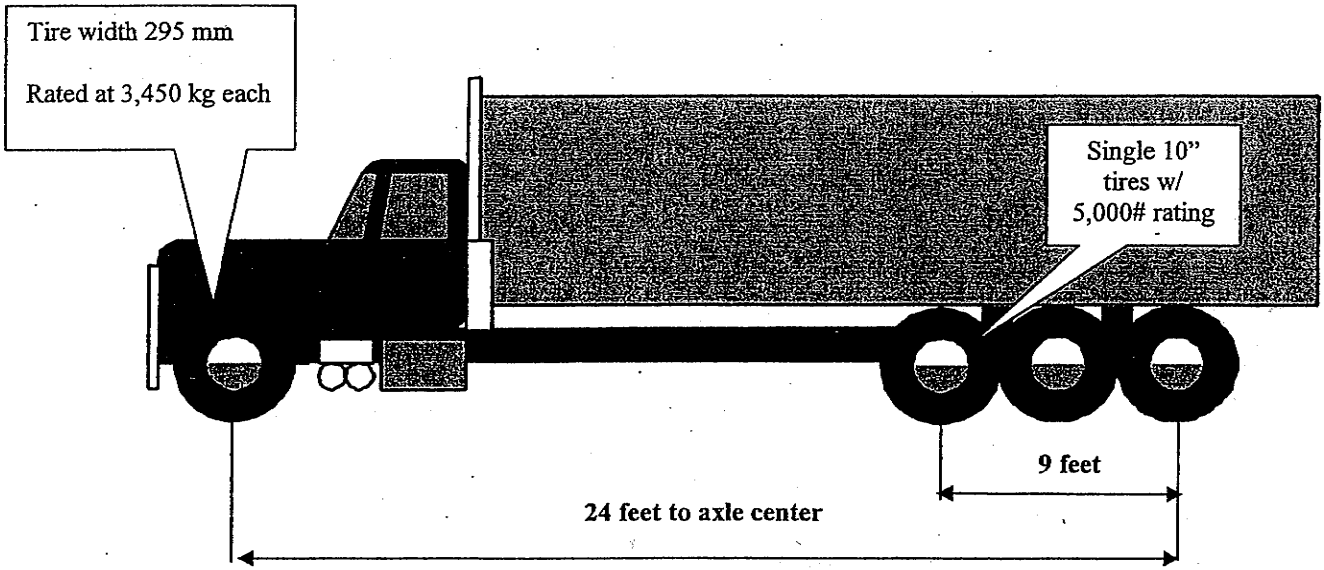

Allowable maximum weight from the gross weight chart? **58,000#**

Allowable steering weight by size = **13,920#** weight by rating = **15,180#**

Is this a tridem (Y or N?) weight from the gross weight chart for the rear group = **43,000#**

Allowable maximum axle weights within tridem?
#1 Weight by size=10,000# #1 Weight by rating =**10,000#** #2 = **15,000#** #3= **15,000#**

Actual weight of rear tridem = **40,000#** (total of 3 axles in rear group as equipped)

Actual legal weight? **53,920#** (13,920# steering + 40,000# tridem)
(steering axle + rear group)

Restricted Weights

5 ton road? Steering = **10,000#** Tridem = **23,889#** (max 33,889#)

7 ton road? Steering = **13,920#** Tridem = **33,444#** (max 47,364#)

10% Increase

Steering axle? **15,180#** (size with 10% increase went over rating)

Tridem axle (**weight chart** w/ increase)? **47,300#**

Allowable (10%) axle weights within tridem?

#1= **10,000#** #2 = **16,500#** #3 = **16,500#** (max of 43,000#)

Actual 10% increased gross weight? **58,180#**

Registration increase?

Note: To convert millimeters (mm) to inches , divide mm by 25.4 = inches

$\frac{\text{mm}}{25.4}$ = inches

example: $\frac{295 \text{ mm}}{25.4} = 11.6$

Also: To convert kilograms (kg) to pounds; multiple by 2.2

example: 3,450 kg X 2.2 = 7,590 lbs

D

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Solutions "D"

Truck "D" Four Axle (single tire on tridem "lift?" axle)

The maximum allowable weight is computed by going to the gross weight chart. Go down the left side to 24 feet (the distance between the first and last axle of the truck), then go across to column 4 (which is the number of axles in the measurement) and you will find 58,000 pounds. This is the maximum allowable weight for this truck if the 58,000 pounds were to be legally distributed within the maximum allowable weights of the axles in the measurement (no overweight axles).

The steering axle weight is noted as "metric". In order to find the maximum allowable weight (noted on the Minnesota tire weight chart on the backside of the gross weight chart), it is necessary to convert metric size to inches. Note the text box on sheet "D". Since the metric tire size is 295mm, that number must be divided by 25.4 to find the measurement in inches. That number is 11.614 inches. (note the backside of the gross weight chart shows this as rounded to 11.6 inches). Since the steering axle is allowed 600 pounds of weight per inch of tire size we must multiply the 11.6 by 600 to get the weight per tire. That weight is 6960 pounds per tire. Assuming both tires are the same size, this weight 6960 multiplied by 2 gives the maximum allowable weight (13,920#) on the steering axle unless the rating is less. The tire rating is 3450 kilograms per tire. This also should be computed to "pounds" to determine if it exceeds the allowable 600 pounds per inch weight. To compute the metric to pounds take the 3450kg times 2.2 which converts the metric 3450kg to 7590 pounds per tire. This weight in pounds (7590) multiplied by 2, gives the maximum allowable "rated" weight as 15,180 pounds.
The tire rating allowable weight is 15,180 pounds.
The tire weight per inch allowable weight is 13,920.
13,920 is the lesser thus is the maximum allowable weight of the steering axle on this truck.

The difference from "C"

The last three axles of this truck are spaced exactly 9 feet apart. This is still defined as a "tridem" as it involves 3 axles spaced 9 feet or less.
Go down the left side of the gross weight table to 9 feet and go across to the 3 axle column (3 axles in the measurement) and you will find the maximum allowable weight in the tridem is 43,000 pounds. Since this is a tridem grouping, no single axle in the tridem group can weigh more than 15,000 pounds. This configuration is common where the first axle of the tridem is a lift axle and sometimes is only a single wheel instead of a duel tire axle. In this example it is a single 10 inch tire. This decision will influence both the maximum tridem weight and the gross weight. Since the lift axle is a single tire, the maximum allowable weight will be governed by the tire size and rating. A 10 inch tire, which is non-steering, is allowed 500 pounds of weight per inch of tire. This limits each tire to 5,000 pounds. In this example the

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rating is 5,000 pounds. The lift axle has a maximum allowable weight of 10,000 pounds. (two single 10 inch tires multiplied by 500 pounds each = 5000 pounds per tire X 2 tires = 10,000 pounds). The decision to use only one tire would mean that the tridem which could have a maximum allowable weight of 43,000 pounds will only be allowed 40,000 pounds. This is because the first (lift) axle can only weigh 10,000 pounds (not 15,000 pounds as it would if it were a duel tire) then add the other two axles maximums at 15,000 pounds each totals 40,000 pounds. (10,000# lift, 15,000# middle duel, 15,000# last axle = 40,000 pounds)
The maximum legal weight of this truck is 13,920 on the steering, and 40,000 pounds on the tridem totaling 53,920. The weight "cost" of only using 1 tire on the lift axle is 3,000 pounds on the tridem, which also lessens the gross weight possibilities. Note that the maximum allowable gross weight of this truck is 2,000 pounds less than truck "C" even though the tridem is allowed (by weight chart) 1,000 pounds more. The additional weight cannot be used because of the lack of tire size. If you wanted to run a maximum allowable weight on this truck using only one tire on each side of the lift axle they would have to be a minimum size of 13 inches each and have adequate ratings to cover 6,500 pounds per tire. The 13,000 pounds is also computed by having two 13 inch tires multiplied by 500 pounds per inch of tire surface (13 X 500# = 6,500 pounds each multiplied by two tires equals 13,000 pounds.) With a 13 inch tire the tridem could weigh 13,000 (lift), 15,000 and 15000 to total the maximum 43,000 pounds allowed on the gross weight table for a 9 foot tridem.

<u>Restricted Weights</u>

Look at the restricted gross weight table chart "I".
The steering axle weight on a 5 ton road would be 10,000 pounds. On a 7 ton road it would be 14,000 pounds. Since the maximum legal weight on the steering axle is 13,920 pounds adjustments would have to be made on the 5 ton routes only.
The tridem axle would be restricted to 23,889 on a 5 ton route, and 33,444 on a 7 ton route. Note that the chart addresses the axles in the tridem as a group, not 3 single axles. (3 non-tridem single axles would be allowed 14,000 [@ 7 ton] each equaling 42,000 pounds...this is not a correct way to determine restricted weight in a tridem).
5 ton 10,000 steering added to 23,889 tridem allows 33,889 maximum allowable.
7 ton 13,920 steering added to 33,444 tridem (Note that the first axle of the tridem on a 7 ton route can only weigh 10,000 pounds due to the rating.) allows 47,364 maximum allowable. Gross weights in excess of these would indicate an overweight somewhere on the vehicle.

<u>10% Weight Increases</u>

The maximum allowable (non 10% increase) steering axle weight is 13,920 pounds. Add to that an additional 10% (1,392 pounds) computes to 15,312 pounds. The rated weight computed to 15,180 pounds. The 10% increase would be allowed <u>up to</u> the rated weight of 15,180. (only 132 pounds short of the 10% possible).

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The maximum allowable (non 10% increase) tridem weight is 43,000 pounds. Add to that an additional 10% (4300 pounds) computes to 47,300 pounds. The individual axle weights within the tridem are increased also to 16,500 pounds. However, because of the single axle on the tridem, all of these potential weights are reduced. Noting the rating on the single lift tridem axle is 5,000 pounds, that axle will receive no benefit of the 10%. If however the axle was rated for 5500 pounds (a 10% increase) it could haul the additional 10% making the maximum allowable weight for that axle still only 11,000 pounds. Assuming the other two axles could maximize their weight at 16,500 pounds each, the total allowable tridem weight would be 44,000 pounds (11,000 + 16,500 + 16,500 =44,000 pounds). However, as shown in the example the tire is rated at 5,000 pounds (same as the allowed 500# per inch) the maximum allowable of the tridem would be 43,000 pounds as there is be 1,000 less pounds allowed on the lift axle. (10,000 + 16,500 + 16,500 = 43000#) The maximum legal weight with 10% increase would be steering = 15,180, tridem = 43,000 pounds totaling a gross weight of 58,180 pounds. This is 3200 pounds less than truck "C". Going to single tires without considering the size and rating could greatly increase your chances of hauling overweight.

Registration Increase & permit?

If the registered weight on the cab card is exceeded by more than 1000 pounds, or 4%, (whichever is greater) the registered weight (license plate weight) must be increased to the weight hauled. This is always true.
A transportation permit is needed during harvest for any state highway but not valid on an Interstate highway.
During the winter weight increase a transportation permit is needed ONLY on an Interstate highway. (I-94, I-35 etc).
Always check with local agencies (city, county and township) prior to increasing any weights on their routes.

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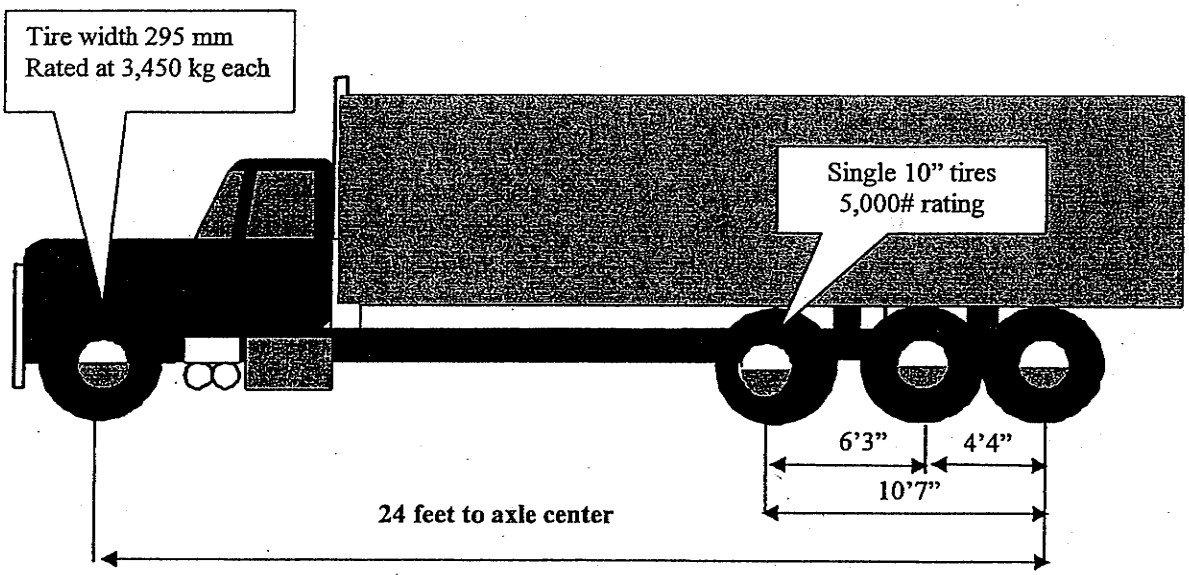

Allowable maximum weight from the gross weight chart? **58,000#**

Allowable steering weight by size = **13,920#** Steering weight by rating = **15,180#**

Is this a tridem (Y / N?) Allowable gross weight from the gross weight chart for the **rear group** = **44,500#**

Axle weights within rear group?
#1 Weight by size =10,000# weight by rating = **10,000#** #2&3= tandem = **34,000#** (max of 44,000#)

Actual gross weight on the rear axle group = ? **44,000#**

Actual legal weight of the vehicle? **57,920#** (only 80# from allowable max.)
(steering + rear axles)

Restricted Weights

5 ton road? Steering = **10,000#** Rear group = **28,889# (10,000#+18,889#)**

7 ton road? Steering = **13,920#** Rear group = **36,444# (10,000#+26,444#)**

10% Increase

Steering axle? **15,180#** (rating)...size + 10% went over the rating.

Rear axle group (**chart** with 10% increase)? **48,950#**

(10%) axle weight increase within group (below)?

#1 Weight by size= 11,000# Rating? = **10,000#** #2&3 = **37,400#**

Actual increased total weight (above) on the rear group? = **47,400#**

Actual 10% increased vehicle gross weight? **62,580#** (weights under 63,800 (58,000 + 5800) are legal)
Registration increase?

Need a transportation permit?

Note: To convert millimeters (mm) to inches , divide mm by 25.4 = inches

mm = inches
25.4

example: 295 mm = 11.6
25.4

Also: To convert kilograms (kg) to pounds; multiple by 2.2

example: 3,450 kg X 2.2 = 7,590 lbs

E

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Solutions "E"

Truck "E" Four Axle (single tire on non-tridem "lift" axle)

The maximum allowable weight is computed by going to the gross weight chart. Go down the left side to 24 feet (the distance between the first and last axle of the truck), then go across to column 4 (which is the number of axles in the measurement) and you will find 58,000 pounds. This is the maximum allowable weight for this truck if the 58,000 pounds were to be legally distributed within the maximum allowable weights of the axles in the measurement (no overweight axles).

The steering axle weight is noted as "metric". In order to find the maximum allowable weight (noted on the Minnesota tire weight chart on the backside of the gross weight chart), it is necessary to convert metric size to inches. Note the text box on sheet "E". Since the metric tire size is 295mm, that number must be divided by 25.4 to find the measurement in inches. That number is 11.614 inches. (note the backside of the gross weight chart shows this as rounded to 11.6 inches). Since the steering axle is allowed 600 pounds of weight per inch of tire size we must multiply the 11.6 by 600 to get the weight per tire. That weight is 6960 pounds per tire. Assuming both tires are the same size, this weight 6960 multiplied by 2 gives the maximum allowable weight (13,920#) on the steering axle unless the rating is less. The tire rating is 3450 kilograms per tire. This also should be computed to "pounds" to determine if it exceeds the allowable 600 pounds per inch weight. To compute the metric to pounds take the 3450kg times 2.2 which converts the metric 3450kg to 7590 pounds per tire. This weight in pounds (7590) multiplied by 2, gives the maximum allowable "rated" weight as 15,180 pounds.
The tire rating allowable weight is 15,180 pounds.
The tire weight per inch allowable weight is 13,920.
13,920 is the lesser thus is the maximum allowable weight of the steering axle on this truck.

The difference from "C" and "D"

The last three axles of this truck are spaced 10 feet 7 inches apart. This is NOT a tridem as the measurement exceeds 9 feet. This means that the axles within this measurement are not limited to a maximum of 15,000 pounds. They are instead limited to a maximum of their tire rating or a maximum weight computed by the tire size multiplied by 500 pounds per inch (whichever is less) up to the maximum weight allowed for the 9 or 10 ton route they are operated on. This means that the maximum axle weights could individually be up to 18,000 (9-ton) or 20,000 (10-ton) as long as the group weight was not exceeded.
The 3 axle group is found by going down the left side of the gross weight chart to the 11 foot measurement (note if the real measurement is 6 inches or greater (10 feet-7 inches) you can use the next foot (11). From the 11 foot measurement move to the right on the chart to the 3 axle column. This shows 44,500 pounds. This is the

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weight that can be hauled on the last three axles as long as the tire ratings or weight per inch (500#) is not exceeded. Because the first axle in the group is a single tire, the tire size is important. These 10 inch tires can haul 5,000 pounds each (it's rating is also 5000 pounds). The way this truck's rear axle group is displayed it can gross a maximum of 44,000 pounds. The first axle (lift) can weigh 10,000 pounds. The other two are a tandem group which can, together, gross 34,000 pounds which totals 44,000 pounds when combined with the first (lift) axle.
Keep in mind that each of the two axles that make up the tandem (last two axles) cannot individually exceed 18,000 pounds on a 9 ton route, or 20,000 pounds on a 10 ton route.
The actual legal weight of this truck is 57,920 pounds. (Steering 13,920, rear group 44,000 pounds = 57,920. (Only 80 pounds from the allowable weight of 58,000)
This example "E" shows the advantage of having the lift axle spaced beyond the "tridem" measurement, thus not limiting the individual axles in the group to 15,000 pounds.

Restricted Weights

Look at the restricted gross weight table chart "I".
The steering axle weight on a 5 ton road would be 10,000 pounds. On a 7 ton road it would be 14,000 pounds. Since the maximum legal weight on the steering axle is 13,920 pounds adjustments would have to be made (to 10,000 pounds) on the 5 ton routes only. The 13,920 pounds will remain the weight for a 7 ton.
The rear group on this truck is not a tridem and the weights used on the weight chart "I" for a tridem would not be valid.
The fist axle of the rear group is limited to 10,000 pounds due to it's rating. That weight is also the legal weight for a 5 ton road. The other two axles can be treated as a tandem. A tandem weight on a 5 ton road is 18,889 pounds. The two added together equal the 5 ton weight for the rear axle group of 28,889 pounds. On a 7 ton road the first axle of the rear group is still locked into 10,000 pounds due to the rating. The second two axles, a tandem, would be allowed by chart "I" 26,444 pounds. The total of all the axles in the rear group would be 36,444 pounds. Because of the single tire in this group, the single tire would not be able to exceed 10,000 pounds on any road, in any group because it is at it's maximum rating.

10% Weight Increases

The maximum allowable (non 10% increase) steering axle weight is 13,920 pounds. Add to that an additional 10% (1,392 pounds) computes to 15,312 pounds. The rated weight computed to 15,180 pounds. The 10% increase would be allowed up to the rated weight of 15,180. (only 132 pounds short of the 10% possible).
The maximum allowable (non 10% increase) weight on the rear 3 axle group is 44,500. Add to that a possible 10% increase of 4450 pounds totals 48,950 pounds. Because the single tire lift axle is a 10 inch tire, rated at 5,000 pounds there can be no increase on this axle. The other two axles, are spaced as a tandem and normally allowed 34,000 pounds. Add 10% to that for a new increased gross of 37,400

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pounds. Now add the lift axle (10,000 pounds) and the rear axle group can haul 47,400 pounds (10,000 + 37,400 = 47,400 pounds.)
The 10% increase on this truck would allow 15,180 on the steering and 47,400 pounds on the rear axle group (as previously distributed) for a total of 62,580. This is very close to the 63,800 pounds the gross weight table allows. (4 axles at 24 feet = 58,000 + 10% (5800) = 63,800 pounds.

Registration Increase & permit?

If the registered weight on the cab card is exceeded by more than 1000 pounds, or 4%, (whichever is greater) the registered weight (license plate weight) must be increased to the weight hauled. This is always true.
A transportation permit is needed during harvest for any state highway but not valid on an Interstate highway.
During the winter weight increase a transportation permit is needed ONLY on an Interstate highway. (I-94, I-35 etc).
Always check with local agencies (city, county and township) prior to increasing any weights on their routes.

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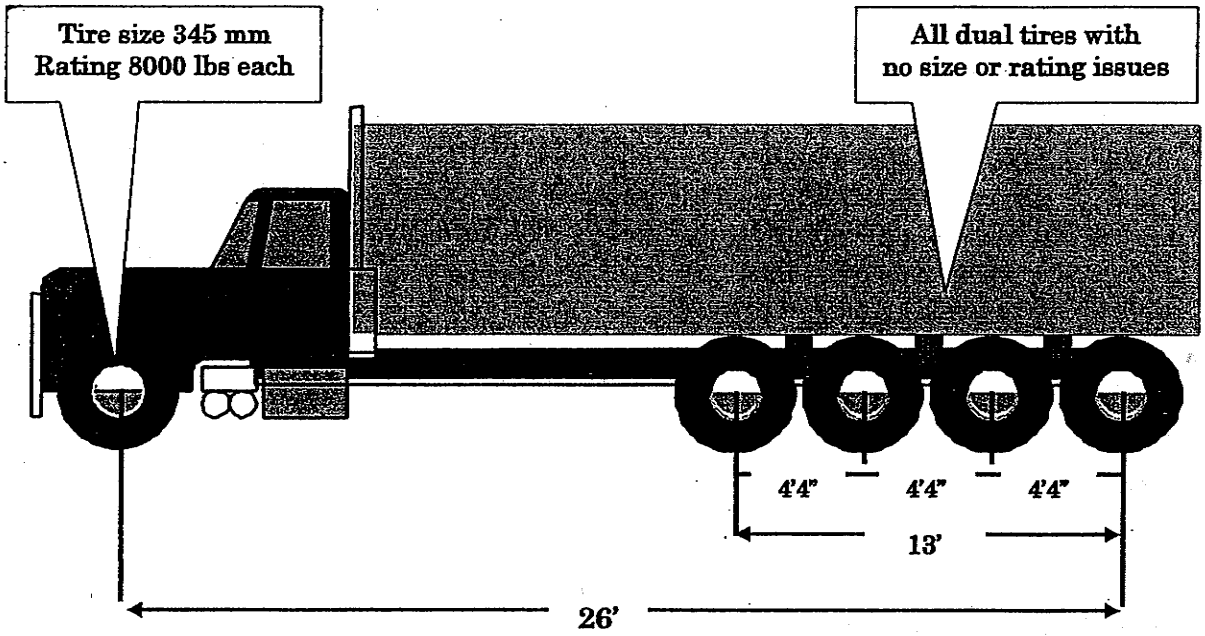

Allowable maximum weight from the gross weight chart = **64,500#**

Allowable steering axle weight by size = **16.320#** by rating = **16,000#**

Any tridems on this truck? **Y** or N

Maximum weight on the quad axle group from the chart = **51,000#**

Maximum weight on any axle within the rear group = **15,000#**

Actual legal weight ? (rear group + steering or chart whichever is less) = **64,500#**

Restricted Weights

5 ton road? Steering **10,000#** Rear axle group **28,611# (From Chart I)**

8 ton road? Steering **16,000#** Rear axle group **45,778# (From Chart I)**

10% Increase

Steering Axle? **16,000# (already at the rated weight)**

Rear Axle Group? **56,100# (51,000 + 5100 = 56,100#)**

Axles within the rear group? **16,500# max per axle not to exceed 56,100 total**

Maximum gross with 10%? **70,950# (cannot exceed chart weight max)**
(the 10% steering and quad [72,100#] exceed the chart 64,500 + 6450 [70,950])

Registration Increase ? (Weight increased from 64,600# to 70.960#)

Transportation Permit? (Interstate during winter, or state highways
During sugar beet, potato or carrot harvest?)

K

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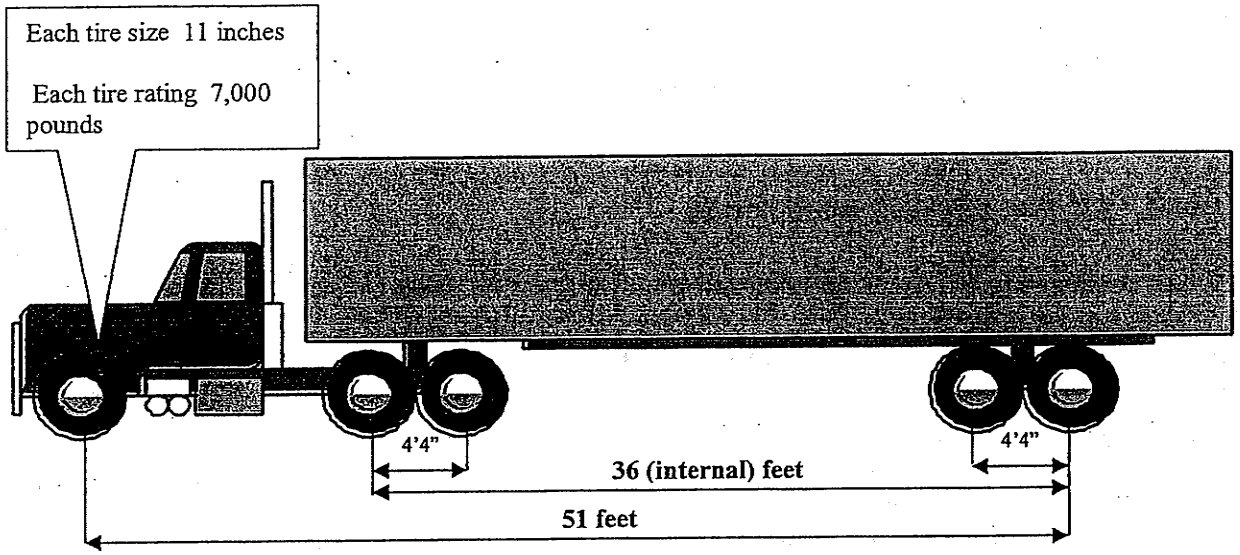

Allowable steering weight by size = **13,200#** .weight by rating = **14,000#**

Each tandem is allowed what weight? **34,000#**

Allowable internal table weight? (4 axles spaced 36 feet)= **68,000#** (these are two consecutive tandems)

Allowable & Actual gross weight? (9 ton) **73,280#**

Allowable & Actual gross weight? (10 ton), **80,000#**

Restricted Weights

5 ton road? Steering = **10,000#** Each tandem = **18,889#** (max gross 47,778#)

7 ton road? Steering = **13,200#** Each tandem= **26,444#** (max gross 66,088#)

10% Increase

Steering axle? **14,000#** (rating)...note that 10% added to the size went over the rating.

Each tandem axle (9 ton)? **37,400# Normal** maximum gross = **80,000#**
(**Normal** means the usual gross weight maximum for 9 ton city and township routes)

Each tandem axle (10 ton)? **37,400#** Max gross = **88,000#***
*(This also applies to 9-ton Winter Load Increases (WLI) on <u>County</u> roads)

Registration increase?

Need a transportation permit?

F

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Solutions "F"

Truck "F" Five axle semi combination

The steering axle weight is determined by taking the tire size (11 inches) times 600 pounds per inch of tire surface. 11 X 600 = 6600 pounds per tire. (Note the chart on the backside of the gross weight table for quick reference). Also we must consider the manufacturers weight rating on the tire and take whichever is less. In this case, the manufacturers rating is 7,000 pounds per tire which is more than the 600 pound per inch of tire surface law allows. So we must use the 6600 pound weight per tire to determine the steering axle weight. 6600 X 2 (two tires on the steering axle) = 13,200 pounds and is the maximum allowable steering axle weight.

Both the drive axles and the trailer axles are spaced as legal "tandem" weights which allow 34,000 pounds per tandem on either a 9 or 10 ton route.

The maximum allowable "internal table weight" is determined by the measurement between the first axle of the drive axle and the last axle of the trailer. This is shown on truck "F" as 36 feet. To determine the maximum allowable weight for that spacing you must go to the gross weight table, down the left side to the 36 foot line and go right to the 4 axle (number of axles in this measurement) column. The blue box notes that two consecutive tandems with a 36 foot measurement can carry a gross weight of 34,000 pounds each with a total of 68,000 pounds combined. The allowable internal table weight is 68,000 pounds.

The maximum allowable weight on a 9 ton route is 73,280 pounds. Truck "F" will probably not use all the allowable weight because of being on a 9 ton route. The maximum axle weights are: steering = 13,200, drivers = 34,000, trailer 34,000 pounds, however because the maximum legal weight for a 9 ton is 73,280 pounds there will be no way to use all the "axle" maximums allowed.

The maximum allowable weight on a 10 ton route is 80,000 pounds. (Note looking at the gross weight table, 5 axles spaced 51 feet is the minimum distance needed for 80,000 pounds on the table. Any distance less than 51 feet would take away from the maximum allowable gross weight. (example: 49 feet would change the maximum to 79,000 pounds...note, the red box areas on the gross weight table are valid only on 10 ton routes).
The steering, drivers, and trailer axles are the same as 9 ton. In the 10 ton example, a common weight distribution could be steering 12,000, drivers 34,000, trailer 34,000 for a maximum gross of 80,000 pounds. Note that if the distance on the "internal table weight" was less than 36 feet, it would reduce the weight on the trailer. (example a measurement of 34 feet on the last 4 axles would maximize that weight at 65,000 pounds...that weight could be distributed up to 34,000 pounds per tandem, but not both tandems, as that would exceed the 65,000 pounds allowed.)

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Restricted Weights

Look at chart "I" the Minnesota restricted gross weight table. The steering axle on a 5 ton route is limited to 10,000 pounds. Note each tandem (drive axle and trailer axle) is limited to 18,889 pounds. Though there is no gross weight restriction in law, the maximum allowable is limited to what the axles can carry. In the 5 ton example it would be steering = 10,000, drivers 18,889, trailer 18,889 which would mean a gross weight in excess of 47,778 would indicate an overweight somewhere on the vehicle.
On a 7 ton route the steering would be limited by size to 13,200 pounds (less than the 14,000 pounds a single axle can carry on a 7 ton route.) Again looking at the chart "I" tandems are allowed 26,444 each. The maximum weight without an overweight somewhere on the truck would be steering = 13,200, drivers 26,444, trailer 26,444 for a possible total of 66,088. This weight may not be possible to attain as usually a truck cannot get 13,200 pounds of weight on the steering axle of a semi without exceeding the 26,444 pounds on the drive axles.

10% Weight Increases

The non 10% allowable steering axle weight is 13,200 pounds. An additional 10% (1320 pounds) would make the steering axle 14,520 pounds which would, in this case, exceed the manufacturers rating of 14,000 pounds (7,000 pounds per tire). So the 10% increase on the steering would end at 14,000 pounds (the rated and correct steering axle weight.)
Each tandem is legal at 34,000 pounds, thus would be allowed an additional 3400 pounds each maximizing the tandem weights at 37,400 pounds each. Note that the internal table weight (68,000 pounds) allows an additional 6800 pounds making that weight allowable to 74,800 pounds. This is exactly what the new tandem weights will allow (37,400 + 37,400 = 74,800). If the internal table weight would have been less than 36 feet, the total weights of the two tandems could not exceed that internal table weight.
On a 9 ton route, the maximum allowable weight during the 10% increase on a 5 axle truck is 80,000 pounds*. (instead of 73,280.) The maximum this truck could haul on a 9 ton route is 80,000 pounds*. The maximum axle weights used to distribute that weight would be: steering 14,000 pounds, drivers 37,400 pounds, and trailer 37,400 pounds.
On a 10 ton route, the maximum allowable weight during the 10% increase on a 5 axle truck is 88,000 pounds. The maximum axle weights used to transport that weight would be steering 14,000 pounds, drivers 37,400, trailer 37,400 pounds. Note when you add the maximums they equal 88,800 pounds. This would be excessive weight and not be legal to haul, as 88,000 is the maximum on a 10 ton route, and 80,000 on a 9 ton route*.

* The 2003 legislature changed the maximum gross weight on a COUNTY 9 ton route to be the same as a 10 ton route during the WLI (winter load increases [not harvest]). This could mean up to 88,000# gross on properly spaced axles.

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Registration Increase & permit?

If the registered weight on the cab card is exceeded by more than 1000 pounds, or 4%, (whichever is greater) the registered weight (license plate weight) must be increased to the weight hauled. This is always true.
A transportation permit is needed during harvest for any state highway but not valid on an Interstate highway.
During the winter weight increase a transportation permit is needed ONLY on an Interstate highway. (I-94, I-35 etc).
Always check with local agencies (city, county and township) prior to increasing any weights on their routes.

Important!

It's very important to make sure you consider the "internal" table weight. The length (demonstrated in truck "F" as 36 feet) must be considered. Without enough length between axles, there could be violations of "combined" weight where no individual axle is, by itself, in violation.

As noted in the example, IF the internal measurement was 34 feet, (not 36 as shown) and IF each tandem was loaded to it's individual maximum (34,000 pounds) there would be a weight violation of 3,000 pounds. Where? Remember that 4 axles spaced 34 feet apart are allowed a combined weight of 65,000 pounds. (see gross weight table, 4 axles spaced 34 feet). But if you loaded each tandem to it's legal limit (34,000 pounds) they would total 68,000 pounds which would exceed the consecutive (table) axle weight allowed (65,000 pounds) by 3,000 pounds and could result in a citation and fine. Remember to measure.

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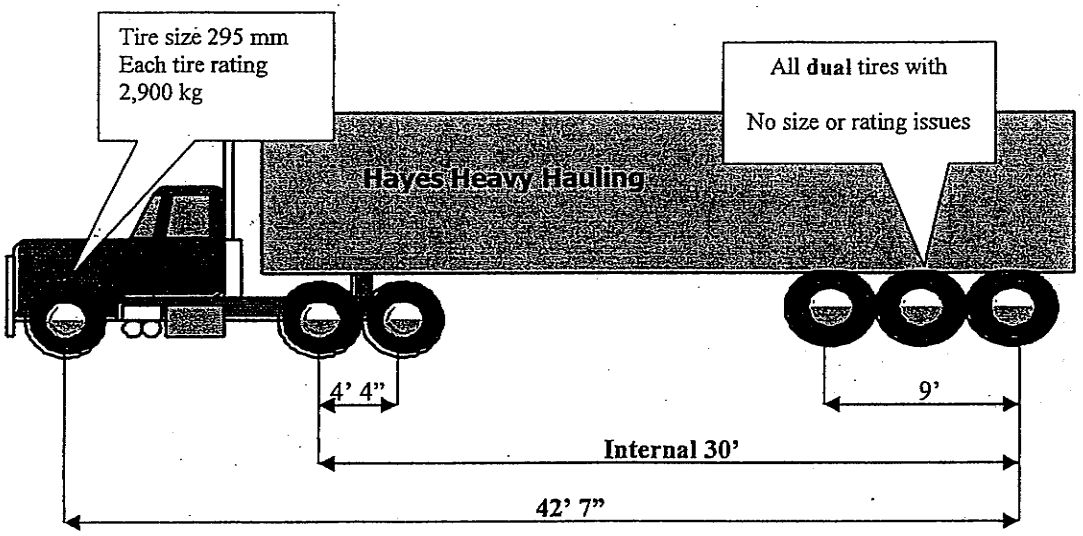

Allowable maximum weight from the gross weight chart? 9 ton = **80,000#** 10 ton = **80,000#**

Allowable steering axle weight? Weight by size = **13,920#** Weight by rating = **12,760 #**

Tandem weight from gross weight chart? **34,000#**

Tridem weight from gross weight chart? **43,000#**

Allowable internal table weight from the chart (5 axles spaced 30 feet) ? **67,000#** (note this is less than the tandem + tridem)

Actual legal weight? (internal weight plus steering) **79,760#**

Note: The internal weight is often less than the weight totaled by adding the tandem and tridem. Always use the lesser weight. The tandem cannot exceed 34,000# and the tridem cannot exceed 43,000# but the total of the 5 axles cannot exceed the internal table weight for 30 feet (in this example).

Restricted Weights (See chart "I")

5 ton road posting? Steering = **10,000#** Tandem = **18,889#** Tridem = **23,889#**

7 ton road posting? Steering = **12,760#** Tandem = **26,444#** Tridem = **33,444#**

10% Increase

Steering axle? **12,760#**

Increased 10 % tandem axle group? **37,400#**

Increased 10% tridem axle group ? **47,300#**

Increased 10% internal table weight? **73,700#**

Increased 10% **Normal** maximum gross weight? 9 ton = **80,000#**
Normal means the maximum weight allowed for city and township routes.

Increased 10% gross weight on 10 ton = **86,460***
*(This also applies to 9-ton Winter Load Increases (WLI) on County roads)

Registration increase?

Need a transportation permit?

G

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Solutions "G"

Truck "G" Six axle semi combination

The maximum (table) weight is 80,000 pounds (either 9 or 10 ton). This is determined by going to the gross weight table, and going down the left side to the 43 foot distance. Since the total distance from axle one to axle six is 42 feet 7 inches, this measurement is rounded to 43 feet. At the 43 foot measurement on the chart go to the right to the 6 axle column which shows 80,000 pounds. Note: This is only one measurement and all axle groups must be measured to determine if the 80,000 pounds can, in fact, be used as the actual maximum weight.

To determine the maximum allowable steering axle weight we must compare the rating to the pounds per inch and use the lesser of the two weights. This is a metric tire size 295mm. As done with trucks C, D, and E this has to be converted to inches by dividing the metric by 25.4. Note the quick reference chart on the backside of the gross weight chart. 295 is converted to 11.6 inches. Now you multiply the inches by the 600 pounds per inch allowed for a steering axle, which would allow 6960 pounds per tire or 13,920 pounds for the axle. We must also consider the tire rating. Truck G has steering axle tires rated at 2900kg. This needs to be converted to pounds by multiplying the 2900kg by 2.2 which equals 6380 pounds per tire. 6380 multiplied by two (two tires) gives a converted rating in pounds to 12,760. In this case the rating is less than the pounds per inch (13,920) so the rating weight must be used. The maximum allowable steering axle weight is 12,760 pounds.

The maximum allowable gross weight on a 9 ton road is 80,000 pounds. When you look at the gross weight table you can see that a 6 axle semi, with properly spaced axles (a minimum of 43 feet) can gross 80,000 pounds on a 9 ton route. This is the maximum weight allowed for a vehicle on a 9 ton route.
The maximum allowable gross weight on a 10 ton road is also 80,000 pounds.

The tandem weight, on the drive axle is 34,000 pounds. Both the 9 and 10 ton columns on the gross weight table allow that weight.

The tridem weight is 43,000 pounds. The gross weight chart shows 3 axles spaced exactly 9 feet apart can haul 43,000 pounds on both a 9 and 10 ton route. Note, this is a tridem (3 axles 9 feet or less) so it is limited to the 15,000 pounds per axle maximum in the tridem group. Since all tires are duels with adequate ratings, we don't have the concerns of having a single tire axle to consider. This truck has enough ratings and tire size for 20,000 pounds per axle which is more than is needed.

The allowable internal table weight is 67,000 pounds. This is computed by going down the left side of the gross weight table to the 30 foot mark (the distance between

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the first drive axle and the last trailer axle) and go to the right to the 5 axle column (the number of axles in the measurement) this will show 67,000 pounds.
NOTE: Even though, in this measurement the tandem can weigh a maximum of 34,000 pounds and the tridem can weigh a maximum of 43,000 pounds you cannot add the two together and assume it can haul that totaled amount of weight (77,000 pounds.) The 30 foot internal table distance only allows 67,000 pounds.

The actual legal weight of truck "G" is 79,760 pounds. This is computed by taking the allowed weight on the steering axle (12,760) and add the maximum allowed on the last 5 axles (67,000) which totals 79,760 pounds. The most probable reason this truck cannot haul the allowed maximum of 80,000 pounds is the short distance in the internal axle table weight.

Restricted Weights

On a 5 ton route, the steering axle would have to be reduced to 10,000 pounds maximum weight. (See chart "I") the tandem weight would be allowed a maximum of 18,889, and the tridem would be allowed a maximum of 23,889 pounds.
On a 7 ton route, the steering axle would remain at the normal maximum legal weight of 12,760 as there is not enough tire "rated weight" to increase the load to 14,000 pounds (if it were possible to do so). The tandem weight according to chart "I" would be allowed 26,444 and the tridem would be allowed a maximum of 33,444 pounds.

10% Weight Increases

The steering axle is already at it's maximum weight of 12,760 as it is at it's maximum weight rating. No increase would be allowed on this axle.
The tandem axle would be allowed an increase from it's legal weight (34,000 pounds) by adding 10% (3400 pounds) to a maximum of 37,400 pounds.
The tridem axle would be allowed an increase from it's legal weight (43,000 pounds) by adding 10% (4300 pounds) to a maximum of 47,300 pounds. However, one cannot simply load to those weights without considering the internal table weight on the last 5 axles. This truck was previously legal at 67,000 pounds on the last 5 axles, so an increase of 10% (6700 pounds) would set a maximum of 73,700 pounds. This is less than the total of the two increased axle groups (tandem and tridem increased weights would be 84,700 pounds [37,400 + 47,300]).

On a 9 ton route the maximum this truck could haul with a 10% increase is 80,000 pounds. (See note*)
On a 10 ton route the maximum this truck could haul with a 10% increase is 86,460.
This is calculated by taking the steering axle weight of 12,760 and adding to it the

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maximum allowable weight for the internal table weight on the last 5 axles which was increased from 67,000 pounds to 73,700 pounds after the 10% increase. 12,760 + 73,700 = 86,460 pounds.
*** The 2003 legislature changed the maximum gross weight on a COUNTY 9 ton route to be the same as a 10 ton route during the WLI (winter load increases [not harvest]). This could mean up to 88,000# gross on properly spaced axles. In this example it would be the same as the 10-ton example 86,460. This legislative increase does not apply to harvest 10% increases in gross weight.**

Registration Increase & permit?

If the registered weight on the cab card is exceeded by more than 1000 pounds, or 4%, (whichever is greater) the registered weight (license plate weight) must be increased to the weight hauled. This is always true.
A transportation permit is needed during harvest for any state highway but not valid on an Interstate highway.
During the winter weight increase a transportation permit is needed ONLY on an Interstate highway. (I-94, I-35 etc).
Always check with local agencies (city, county and township) prior to increasing any weights on their routes.

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MINNESOTA WEIGHT LIMITS	9-Ton Route	10-Ton Route
Single Axle Weight (not on chart)	18,000 pounds	20,000 pounds
Single Tire weight	9,000 pounds	10,000 pounds
Tandem Axles(2 axles spaced more than 40 inches, but not more than 96 inches [8 feet] apart.)	34,000 pounds	34,000 pounds
Tire—Steering Axle	**600 pounds** per inch of *tire width or* the manufacturer's rated load, whichever is less. (Also, shall not exceed single wheel weight limit.)	**600 pounds** per inch of *tire width* or the manufacturer's rated load, whichever is less. (Also, shall not exceed single wheel weight limit.)
Tire-Other than Steering Axle	**500 pounds** per inch of *tire width* or the manufacturers rated load, whichever is less. (Also, shall not exceed single wheel weight limit.)	**500 pounds** per inch of *tire width* or the manufacturer's rated load, whichever is less. (Also, shall not exceed single wheel weight limit.)
Two or more consecutive axles	*Refer to Gross Weight Chart*	*Refer to Gross Weight Chart*
Gross Vehicle Weight	73,280 pounds with five or fewer axles; 80,000 pounds with six or more axles.	80,000 pounds

Tridem Axles are defined as 3 axles spaced 9 feet or less. When a group is defined as a tridem, each axle in the tridem has a maximum potential of weighing up to 15,000 pounds. There must be enough tire size and rating to allow 15,000 pounds. Since the maximum weight for a 9 foot tridem is 43,000 pounds (chart), it would not be possible for each axle to weigh the 15,000 pounds as the group would then weigh 45,000 pounds or be 2,000 pounds over the 43,000 shown on the chart.

NOTE:
Most all State Highway are 10 ton routes. County and City road authorities may "Designate" a local route as 10 ton by following the rules set in state statutes.

H

Restricted Axle Weight Limits in Pounds

	POSTED AXLE LIMITS					
	9 TON	8 TON	7 TON	6 TON	5 TON	4 TON
Single axle	18,000	16,000	14,000	12,000	10,000	8,000
Two axles spaced within 8 feet or less (Tandem)	34,000	30,222	26,444	22,667	18,889	15,111
Three axles spaced within 9 feet or less (Tridem)	*	38,222	33,444	28,667	23,889	19,111
Four axles spaced within 14 feet or less	*	45,778	40,056	34,333	28,611	22,889

NOTES: *1. For 9 ton restricted routes see the 9 ton GROSS WEIGHT TABLE for maximum weights allowed on groups of 3 and 4 axles spaced less than 9 and 14 feet respectively.

2. No combination of axle weights shall exceed those weights specified for non-designated (9 ton) routes.

Make sure to consider the individual tire "size" and "rating" in allowing the weights on this chart.

I

2003, 2ND ED AASHTO MCE

had an effect. The location of any cracks and their size should be carefully noted for future reference and comparison. Evidences of rust at cracks can mean possible damage to prestressing steel.

Pretensioned box sections should be checked during the passage of heavy loads to see whether any unit is acting independently of the others. Such independent action would indicate spreading of the girders or failure of the longitudinal key between girders.

On bridges with underpassing traffic the exterior faces and the soffits of all types of prestressed girders should be examined. Spalling, cracking or damage to prestressing steel should be noted.

Inspections of earthquake restrainer mechanisms and for earthquake damage should be conducted as outlined in Article 3.8.3.2.

3.8.3.4 Timber Systems

Examine timber stringers for splitting, cracking, and excessive deflection. Look for crushing and evidence of decay where they bear on the bent caps or abutment seats and at their top edge where the floor is supported. Stringers should be kept clear of dirt accumulations to help prevent decay from starting and to help prevent its acceleration once it has started.

The bridging between the timber stringers should be checked to see that it is tight and functioning properly. Timber connections should be checked for loose or missing fasteners.

In order to evaluate the capacity of existing timber structures, the following information should be recorded:

(a) The beam size, spacing and span length;
(b) The type of beam: rough sawn, dressed, nail-laminated, or glue-laminated;
(c) Horizontal shear capacity is controlled by beam depth. Have beams been cut or notched at the bearing and to what extent?
(d) Age of timber should be estimated;
(e) The moisture content of the timber should be estimated or measured;
(f) The species and grade of the lumber should be identified. Original and repair construction records should be checked for material delivery slips. Where no information is available, the inspector must use judgment based upon local experience, visual appearance, odor, cross grain, etc. Where more exact information is required, obtain a sample for testing by a laboratory.

The age, moisture content, species and grade of timber are used in establishing values for the allowable timber stresses to be used in the load rating computations. Field grading and/or estimates of allowable stresses may be necessary.

3.8.3.5 Floor Systems

Truss and deck girder structures are constructed with a system of stringers, floor beams and, if present, brackets to transmit the live load from the deck to the main load-carrying members (girders or trusses). The transverse floor beams and/or brackets can be Fracture Critical Members depending on the framing used. A U-bolt floor beam connection to the truss may be an example of a fracture critical detail. The bridge record should clearly indicate whether or not the floor system contains FCMs.

Inspect stringers, floor beams and overhang brackets for cracks and losses due to rust. Floor beams and connections located below deck relief joints frequently show severe rust due to leakage through the deck joint. Floor beam overhanging tie plates should be carefully examined for evidence of cracking or section loss.

Stringer systems are usually provided with simple expansion devices such as slotted holes at the floor beam connections. These expansion devices should be checked for freedom of movement, uplift or other evidence that the floor system is not functioning as designed.

The floor beams are frequently subjected to out-of-plane bending due to restraints imposed by stringer, girder and bracing connections. Check for evidence of fatigue cracks adjacent to the various connection points.

On those bridges where the deck does not bear directly on the main longitudinal members, there is a tendency for the deck and main longitudinal members not to respond to dynamic loading in synchronization which can cause twisting and out-of-plane bending in the floor beams. Check for evidence of fatigue cracks adjacent to the floor beam/girder connections.

3.8.3.6 Trusses

The examination of any truss will normally begin with sighting along the roadway rail or curb and along the truss chord members to determine any misalignment either vertical or horizontal. Check alignment of trusses carefully for any sag which may

is usually attached to the stringers with plug welds which are not directly observable. Vertical movement of the deck under the passage of live load may indicate weld failure. The fill material of the wearing surface should be examined for cracks or depressions. Open cracks in the wearing surface will allow rust through of the deck elements to occur at an accelerated rate.

Orthotropic steel plate decks consist of a flat steel plate with a series of stiffening web elements. A wearing surface is bonded to the top of the steel plate. On some structures the steel plate is itself a flange element of a box girder section. The inspector should check for debonding of the overlay, rust through or cracks in the steel plate, and for the development of fatigue cracks in the web elements or connecting welds. The connection between the orthotropic plate deck and supporting members should be checked, where visible, and any evidence of live load movement noted.

3.8.4.4 Timber Decks

Timber decks should be examined for decay especially at their contact surfaces where they bear on the stringers and between layers of planking or laminated pieces. Note any looseness which may have developed from inadequate nailing or bolting, or where the spikes have worked loose. Observation under passing traffic will reveal looseness or excessive deflection in the members.

3.8.4.5 Expansion Joints

Expansion joints provide for thermal expansion of the deck and superstructure. They should be checked for freedom of expansion. The clear opening of the joint should provide for adequate expansion of the adjacent superstructure elements considering the span lengths and temperature at the time of inspection. The inspector should measure expansion joint openings and ambient temperature at easily identifiable locations, so that future inspections can establish a record of joint movement over time. Inspect for solid objects (noncompressibles) which can become wedged in the joint and prevent joint contraction.

On joints without armoring, inspect for proper joint alignment, the presence and condition of any joint sealant material, and for evidence of spalls or "D" cracking in the slab edges which would prevent proper sealing of the joint.

Armored joints, without sealant material, such as sliding plate dams or finger joints should be inspected both above and below deck for the condition of the supports. Any horizontal or vertical misalignment of the joint elements should be recorded and checked at future inspections. Where drainage troughs are provided, check for a build-up of debris that prevents proper drainage and causes spill over onto the superstructure and substructure components, or impedes joint movement.

Sealed armored joints such as strip seals or compression seals should be checked for the presence of defects such as tears, separations, sagging, protrusions or embedment of foreign material. Ultraviolet degradation of the seal material is evidenced by hardening and brittleness of the surface and by the appearance of pattern cracking. The underside of all sealed deck joints should be checked for evidence of active joint leakage shown by water staining of the underlying structural elements. Areas of water staining should be clearly marked on drawings or in the field notes, so that future inspections can more accurately assess the extent of active leakage.

Reinforced elastomeric joints are composed of various proprietary combinations of steel supports and sealant material. Inspect for missing anchor bolt covers, separation of joint elements and audible or visual evidence of loose joint panels under traffic. Loose joint panels should be repaired immediately because the bolt failure is progressive and may result in one of the joint panels breaking loose under traffic.

Modular joints are composed of single or multiple support systems working together to accommodate large bridge movements. Inspect for surface damage to seals and separation beams. Examine underside for evidence of leakage and also for unusual noise which may indicate fractured welds or bolts.

3.8.4.6 Railings, Sidewalks and Curbs

3.8.4.6.1 Railings

Bridge railing and parapets, if present, should be evaluated as to condition and as to adequacy of geometry and structural capacity. The inspector should be familiar with the railing requirements of the Bridge Owner. On through-truss bridges, the structural elements, especially fracture critical members such as eyebars, hangers, etc., should be separated from traffic by an adequate vehicular railing system to prevent vehicle impact from causing major structural damage

penetrant seeps from the crack where it is trapped and stains the developer. For this reason bright-colored (often red) penetrants are used. The red penetrant stains on the white chalky developer indicate the presence of a crack or other defect when visually inspected by the examiner. Modifications of the system include penetrants of different viscosity to detect different size cracks, wet rather than dry developers, and penetrants that fluoresce under ultraviolet light. These penetrants, used in conjunction with ultraviolet light examination, make smaller defects visible.

The principal advantages of the method are the ease with which the tests are conducted, the minimal skills required, and the low cost. Tests are not time consuming and may be made frequently during other operations (for example, to determine if a defect being removed by grinding is completely eliminated). It must be considered the most portable of all methods.

The principal disadvantage is that only surface defects can be detected. This places a limitation on the usefulness of the method for the defect depth determination and "code" approval of most structures. However, from the practical shop viewpoint, many defects that occur during construction (for example, weld cracks) are detectable if dye penetrant is used at intermediate stages in the construction. Thus, defects that are later buried can be detected and repaired before they are hidden from view. Use of dye penetrant during fabrication may prevent later rejection when ultrasonic or X-ray examination is used. The more sophisticated dye penetrant methods, using ultraviolet light, are rarely used in field applications.

4.2.2.5 Ultrasonic Examination

Ultrasonic testing relies on the wave properties of sound in materials to detect internal flaws. High-frequency sound waves in the form of mechanical vibrations are applied to the part to be tested and the waves, passing through the material, strike either a defect or, eventually, an external surface. The sound vibrations are then reflected and the nature of the return signal indicates the location and type of reflecting surface. Normal instrumentation includes a sound wave generator and pick-up device (usually combined in one unit) and a display screen on which the initial and reflected pulse is displayed. Display instrumentation permits an estimation of the position (in depth) of the defect, the nature of the defect and, by moving the detection portion of the unit (called the search unit) along the part to be examined, the size of the defect. The test sensitivity is influenced by a great number of testing variables, such as sound frequency, design of the search unit, instrumentation, electronic processing of the return signal, and the skill of the operator. Normal results of the examination are a form prepared by the operator based on his observations of the display screen.

The major advantages of this system of nondestructive examination are its portability, sensitivity and ability to detect the location of cracks or defects in depth. On the other hand, the major fault of the system is that, until very recent times, no permanent record of the defect was produced. It is now possible to make photographic records of the display, and equipment is now available to permit the storage of field data in a format suitable for subsequent computer processing and reporting. Another characteristic of the system often cited as a difficulty is the sensitivity of the method. It is possible to see too much; i.e., grain size in metals and minor defects not observable by other methods. The system cannot detect surface defects very well. The dependency of the method on operator skill must also be considered an unfavorable factor.

More research has been undertaken to modify this method and make it more widely applicable than most of the others, so advances in technology are more likely in this field.

4.2.3 Timber Field Tests

Typical field test procedures for detecting defects and deterioration in timber bridge components are described below.

A summary of the capabilities of each of the test methods for detecting defects and deterioration in timber components is given in Table 4.2.3. This table should be used as a guide in selecting an appropriate field test procedure for timber components.

4.2.3.1 Penetration Methods

Any probe, such as a knife, ice pick, nail, or brace and bit, can be used to test for internal decay or vermin infestation. The ease with which a member can be penetrated is then a measure of its soundness. Only a qualitative assessment is obtained because the pressure on the instrument is neither controlled nor measured. Although the procedure is rather crude, it is rapid and an overall assessment of the condition

Table 4.2.3 Capability of Investigative Techniques for Detecting Defects in Timber Structures in Field Use

Method Based on	Capability of Defect Detection[a]				
	Surface Decay and Rot	Internal Decay and Voids	Weathering	Chemical Attack	Abrasion and Wear
Penetration	G	G	F	F	N
Electrical	F	F	N	N	N
Ultrasonics	N	G	G	N	N

[a]G = Good; F = Fair; P = Poor; N = Not suitable.

of a structure can be obtained quickly. The use of a probe is much more satisfactory than attempting to identify a hollow member by sounding because the load on the member affects the response and may lead to erroneous conclusions.

An increment borer, which consists of a sharpened hollow tube, usually about 1/4-in. (6-mm) internal diameter, can also be used to penetrate the wood. The borer is superior to a nail or ice pick because it gives a more accurate record of the depth of decay or infestation. It also allows samples to be removed from the interior of the member for detailed examination or testing for such items as moisture content and preservative penetration, or to be cultured for positive evidence of decay fungi.

4.2.3.2 Electrical Methods

The main application of electrical methods is to measure the moisture content of timber. There are several electrical techniques available for measuring moisture content.

Resistance meters are based on a direct current measurement of electrical resistance between point or blade electrodes pushed into the timber. The resistance is related to the moisture content, which is displayed on a calibrated scale. The results are affected by the species of timber and correction factors must be applied. Resistance moisture meters are light, compact, and inexpensive but the major disadvantage is that they measure the moisture content of the surface layers unless special deep probes are used. Readings over 30 percent moisture content are not reliable and contamination by some chemicals, such as salt, affects the readings.

Capacitance meters are based on an alternating current measure of the dielectric constant of wood, which is proportional to its moisture content. The results are a function of the relative density of the wood and correction factors must be applied. The meters measure primarily surface moisture content, and, on lumber thicker than 2 in. (50 mm), do not respond to internal moisture adequately. Capacitance meters have a wider range (0 to at least 35 percent moisture content) than resistance meters and are less affected by the presence of chemicals.

Radio frequency power-loss meters operate in the frequency range 0 to 25 MHz and are based on an alternating current measurement of the impedance (combined effect of resistance) and capacitance of timber. They need to be calibrated for wood species and density. The meters use plate-type electrodes and the field penetrates about 3/4 in. (20 mm) but the surface layers have the predominant effect. The cost of the meters is similar to that of capacity-type meters, being higher than that of simple resistance types.

Electrical resistance measurements are also the basis of an instrument designed to detect internal rot. The device consists of a resistance probe, which is inserted to various depths in a hole 3/32 in. (2.4 mm) in diameter. A marked change in electrical resistance is an indication of decay. Although the device effectively detects rot, it is susceptible to false indications of decay in apparently sound wood.

4.2.3.3 Ultrasonic Techniques

The same ultrasonic pulse-velocity equipment and techniques described in Article 4.2.1.3 for application to concrete members can also be used for the in-situ testing of timber structures, both above and below the water surface.

Pulse-velocity measurements relate to the elastic properties of the wood and are therefore sensitive to the direction of the grain. However, pulse-velocity measurements have been found to follow similar trends to strength changes caused by fluctuations in density and local defects. Consequently, the strength and stiffness properties of the timber can be assessed. The ultrasonic method can also be used to identify internal decay and hollow areas, as well as internal knots, checks, and shakes. Because a discontinuity, such as a crack or a hollow area caused by decay, reflects part of the sound wave and changes the veloc-

ity of the transmitted wave, the technique is most sensitive to detecting defects that are oriented perpendicularly to the pulse. For this reason, the direct transmission mode with transducers on opposite faces of the member is generally the most useful configuration. However, in some situations, it may be necessary to investigate other relative positions of the transducers in order to produce a maximum response. To simplify interpretation of the results it is common practice to compare the pulse velocity from a suspected area of deterioration with that from an area known to be sound (measured using the same transducer configuration), thereby eliminating the need to measure the density of the timber. In all cases, a good contact between the transducer and the surface of the timber is essential. A light grease or glycerol are suitable for the coupling medium. Bentonite paste has also been found satisfactory.

4.3 MATERIAL SAMPLING

Tests which require the removal of material from the structure should be used only when a particular piece of information is desired, and only when the results can provide something useful in the overall evaluation of the bridge.

A few common material sampling standards are shown in Table 4.3-1. Samples should be removed from those areas of a bridge subjected to low stress levels as determined by the Engineer. An adequate number of samples should be obtained to provide results representative of the entire structure being evaluated. Normally, a minimum of three samples would be required.

The removal of material from a structure will leave a hole or void in one or more members. Repairs can be readily made to concrete, masonry and timber members. Repairs to steel members may be much more complex, particularly if welding is used, and should be carried out by experienced personnel. Care should be taken to minimize any residual stress resulting from the repair.

Table 4.3-1 Standard ASTM and AASHTO Methods for Material Sampling

Designation	Title
C 42/T 24	Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
T 260	Sampling and Testing for Total Chloride Ion in Concrete Raw Materials
C 823	Standard Practice for Examination and Sampling of Hardened Concrete in Constructions
A 610	Sampling Ferroalloys for Size (Before or After Shipment)
A 673	Sampling Procedures for Impact Testing of Structural Steel (Charpy Test)
A 370	Standard Test Methods and Definitions for Mechanical Testing of Steel Products

TABLE 4.4-1 Standard ASTM and AASHTO Test Methods for Concrete for Use in the Laboratory

Designation[a]	Title
C 39/T 22	Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 85/T 178	Test Method for Cement Content of Hardened Portland Cement Concrete
C 174/T 148	Method of Measuring Length of Drilled Concrete Cores
C 457	Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
C 469	Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C 496	Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C 617/T 231	Method of Capping Cylindrical Concrete Specimens
C 642	Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete
C 666/T 161	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C 856	Recommended Practice for Petrographic Examination of Hardened Concrete
T 259	Method of Test for Resistance of Concrete to Chloride Ion Penetration[b]
T 260	Method of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials
T 277	Interim Method of Test for Rapid Determination of the Chloride Permeability of Concrete

[a]ASTM test methods are designated C. AASHTO test methods are designated T.
[b]Corrosion threshold is about 1.3 to 2.0 pounds of chloride per cubic yard.

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6. LOAD RATING

6.1 GENERAL

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

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

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

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

6.1.1 Assumptions

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

6.1.2 Substructure Consideration

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

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

6.1.3 Safety Criteria

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

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

6.1.4 Application of Standard Design Specifications

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

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)} \qquad (6\text{-}1a)$$

where:

RF = the rating factor for the live-load carrying capacity. The rating factor multiplied by the rating vehicle in tons gives the rating of the structure (see equation 6-1b)

C = the capacity of the member (see Article 6.6)

D = the dead load effect on the member (see Article 6.7.1). For composite members, the dead load effect on the noncomposite section and the dead load effect on the composite section need to be evaluated when the Allowable Stress method is used

L = the live load effect on the member (see Article 6.7.2)

I = the impact factor to be used with the live load effect (see Article 6.7.4)

A_1 = factor for dead loads (see Articles 6.5.2 and 6.5.3)

A_2 = factor for live load (see Articles 6.5.2 and 6.5.3)

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

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

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

where:

RT = bridge member rating in tons

W = weight (tons) of nominal truck used in determining the live load effect (L)

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

6.5.2 Allowable Stress

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

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

6.5.3 Load Factor

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

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

6.5.4 Condition of Bridge Members

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

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

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

6.6.2.7 Timber

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

(1) Inventory Stress

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

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

(2) Operating Stress

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

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

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

in which

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

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

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

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

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

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

6.6.3 Load Factor Method

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

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

6.6.3.1 Structural Steel

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

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

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

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

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Section 3
LOADS

Part A
TYPES OF LOADS

3.1 NOTATIONS

A = maximum expected acceleration of bedrock at the site
a = length of short span of slab (Article 3.24.6)
B = buoyancy (Article 3.22)
b = width of pier or diameter of pile (Article 3.18.2.2.4)
b = length of long span of slab (Article 3.24.6)
C = combined response coefficient
C = stiffness parameter = K(W/L) (Article 3.23.4.3)
C = centrifugal force in percent of live load (Article 3.10.1)
CF = centrifugal force (Article 3.22)
C_n = coefficient for nose inclination (Article 3.18.2.2.1)
C_M = steel bending stress coefficient (Article 3.25.1.5)
C_R = steel shear stress coefficient (Article 3.25.1.5)
D = parameter used in determination of load fraction of wheel load (Article 3.23.4.3)
D = degree of curve (Article 3.10.1)
D = dead load (Article 3.22)
D.F. = fraction of wheel load applied to beam (Article 3.28.1)
DL = contributing dead load
E = width of slab over which a wheel load is distributed (Article 3.24.3)
E = earth pressure (Article 3.22)
EQ = equivalent static horizontal force applied at the center of gravity of the structure
E_c = modulus of elasticity of concrete (Article 3.26.3)
E_s = modulus of elasticity of steel (Article 3.26.3)
E_w = modulus of elasticity of wood (Article 3.26.3)
F = horizontal ice force on pier (Article 3.18.2.2.1)
F = framing factor (Article 3.21.1.1)
F_b = allowable bending stress (Article 3.25.1.3)
F_v = allowable shear stress (Article 3.25.1.3)
g = 32.2 ft./sec.^{2}
I = impact fraction (Article 3.8.2)
I = gross flexural moment of inertia of the precast member (Article 3.23.4.3)
ICE = ice pressure (Article 3.22)
J = gross Saint-Venant torsional constant of the precast member (Article 3.23.4.3)
K = stream flow force constant (Article 3.18.1)
K = stiffness constant (Article 3.23.4)
K = wheel load distribution constant for timber flooring (Article 3.25.1.3)
k = live load distribution constant for spread box girders (Article 3.28.1)
L = loaded length of span (Article 3.8.2)
L = loaded length of sidewalk (Article 3.14.1.1)

L = live load (Article 3.22)
L = span length (Article 3.23.4)
LF = longitudinal force from live load (Article 3.22)
M_D = moment capacity of dowel (Article 3.25.1.4)
$\underline{M_x}$ = primary bending moment (Article 3.25.1.3)
$\overline{M_y}$ = total transferred secondary moment (Article 3.25.1.4)
N_B = number of beams (Article 3.28.1)
N_L = number of traffic lanes (Article 3.23.4)
n = number of dowels (Article 3.25.1.4)
P = live load on sidewalk (Article 3.14.1.1)
P = stream flow pressure (Article 3.18.1)
P = total uniform force required to cause unit horizontal deflection of whole structure
P = load on one rear wheel of truck (Article 3.24.3)
P = wheel load (Article 3.24.5)
P = design wheel load (Article 3.25.1.3)
P_{15} = 12,000 pounds (Article 3.24.3)
P_{20} = 16,000 pounds (Article 3.24.3)
p = effective ice strength (Article 3.18.2.2.1)
p = proportion of load carried by short span (Article 3.24.6.1)
R = radius of curve (Article 3.10.1)
R = normalized rock response
R = rib shortening (Article 3.22)
R_D = shear capacity of dowel (Article 3.25.1.4)
$\underline{R_x}$ = primary shear (Article 3.25.1.3)
$\overline{R_y}$ = total secondary shear transferred (Article 3.25.1.4)
S = design speed (Article 3.10.1)
S = soil amplification spectral ratio
S = shrinkage (Article 3.22)
S = average stringer spacing (Article 3.23.2.3.1)
S = spacing of beams (Article 3.23.3)
S = width of precast member (Article 3.23.4.3)
S = effective span length (Article 3.24.1)
S = span length (Article 3.24.8.2)
S = beam spacing (Article 3.28.1)
s = effective deck span (Article 3.25.1.3)
SF = stream flow (Article 3.22)
T = period of vibration
T = temperature (Article 3.22)
t = thickness of ice (Article 3.18.2.2.4)
t = deck thickness (Article 3.25.1.3)
V = variable spacing of truck axles (Figure 3.7.7A)
V = velocity of water (Article 3.18.1)
W = combined weight on the first two axles of a standard HS Truck (Figure 3.7.7A)
W = width of sidewalk (Article 3.14.1.1)
W = wind load on structure (Article 3.22)
W = total dead weight of the structure
W_e = width of exterior girder (Article 3.23.2.3.2)
W = overall width of bridge (Article 3.23.4.3)
W = roadway width between curbs (Article 3.28.1)
WL = wind load on live load (Article 3.22)
w = width of pier or diameter of circular-shaft pier at the level of ice action (Article 3.18.2.2.1)
X = distance from load to point of support (Article 3.24.5.1)
x = subscript denoting direction perpendicular to longitudinal stringers (Article 3.25.1.3)

3.3.5 Where the abrasion of concrete is not expected, the traffic may bear directly on the concrete slab. If considered desirable, ¼ inch or more may be added to the slab for a wearing surface.

3.3.6 The following weights are to be used in computing the dead load:

	#/cu.ft.
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete, plain or reinforced	150
Compacted sand, earth, gravel, or ballast	120
Loose sand, earth, and gravel	100
Macadam or gravel, rolled	140
Cinder filling	60
Pavement, other than wood block	150
Railway rails, guardrails, and fastenings (per linear foot of track)	200
Stone masonry	170
Asphalt plank, 1 in. thick	9 lb. sq. ft.

3.4 LIVE LOAD

The live load shall consist of the weight of the applied moving load of vehicles, cars, and pedestrians.

3.5 OVERLOAD PROVISIONS

3.5.1 For all loadings less than H 20, provision shall be made for an infrequent heavy load by applying Loading Combination IA (see Article 3.22), with the live load assumed to be H or HS truck and to occupy a single lane without concurrent loading in any other lane. The overload shall apply to all parts of the structure affected, except the roadway deck, or roadway deck plates and stiffening ribs in the case of orthotropic bridge superstructures.

3.5.2 Structures may be analyzed for an overload that is selected by the operating agency in accordance with Loading Combination Group IB in Article 3.22.

3.6 TRAFFIC LANES

3.6.1 The lane loading or standard truck shall be assumed to occupy a width of 10 feet.

3.6.2 These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width measured between curbs.

3.6.3 Fractional parts of design lanes shall not be used, but roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

3.6.4 The traffic lanes shall be placed in such numbers and positions on the roadway, and the loads shall be placed in such positions within their individual traffic lanes, so as to produce the maximum stress in the member under consideration.

3.7 HIGHWAY LOADS

3.7.1 Standard Truck and Lane Loads*

3.7.1.1 The highway live loadings on the roadways of bridges or incidental structures shall consist of standard trucks or lane loads that are equivalent to truck trains. Two systems of loading are provided, the H loadings and the HS loadings—the HS loadings being heavier than the corresponding H loadings.

3.7.1.2 Each lane load shall consist of a uniform load per linear foot of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans—see Article 3.11.3), so placed on the span as to produce maximum stress. The concentrated load and uniform load shall be considered as uniformly distributed over a 10-foot width on a line normal to the center line of the lane.

3.7.1.3 For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 3.7.6B. The lighter concentrated loads shall be used when the stresses are primarily bending stresses, and the heavier concentrated loads shall be used when the stresses are primarily shearing stresses.

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*Note: The system of lane loads defined here (and illustrated in Figure 3.7.6.B) was developed in order to give a simpler method of calculating moments and shears than that based on wheel loads of the truck.

Appendix B shows the truck train loadings of the 1935 Specifications of AASHO and the corresponding lane loadings.

In 1944, the HS series of trucks was developed. These approximate the effect of the corresponding 1935 truck preceded and followed by a train of trucks weighing three-fourths as much as the basic truck.

3.7.2 Classes of Loading

There are four standard classes of highway loading: H 20, H 15, HS 20, and HS 15. Loading H 15 is 75 percent of loading H 20. Loading HS 15 is 75 percent of Loading HS 20. If loadings other than those designated are desired, they shall be obtained by proportionately changing the weights shown for both the standard truck and the corresponding lane loads.

3.7.3 Designation of Loadings

The policy of affixing the year to loadings to identify them was instituted with the publication of the 1944 Edition in the following manner:

H 15 Loading, 1944 Edition shall be designated	H 15-44
H 20 Loading, 1944 Edition shall be designated	H 20-44
H 15-S 12 Loading, 1944 Edition shall be designated	HS 15-44
H 20-S 16 Loading, 1944 Edition shall be designated	HS 20-44

The affix shall remain unchanged until such time as the loading specification is revised. The same policy for identification shall be applied, for future reference, to loadings previously adopted by the American Association of State Highway and Transportation Officials.

3.7.4 Minimum Loading

Bridges supporting Interstate highways or other highways which carry, or which may carry, heavy truck traffic, shall be designed for HS20-44 Loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000 pounds, whichever produces the greatest stress.

3.7.5 H Loading

The H loadings consist of a two-axle truck or the corresponding lane loading as illustrated in Figures 3.7.6A and 3.7.6B. The H loadings are designated H followed by a number indicating the gross weight in tons of the standard truck.

3.7.6 HS Loading

The HS loadings consist of a tractor truck with semitrailer or the corresponding lane load as illustrated in Figures 3.7.7A and 3.7.6B. The HS loadings are designated by the letters HS followed by a number indicating the gross weight in tons of the tractor truck. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans, in that heavy axle loads may be so placed on adjoining spans as to produce maximum negative moments.

3.8 IMPACT

3.8.1 Application

Highway Live Loads shall be increased for those structural elements in Group A, below, to allow for dynamic, vibratory and impact effects. Impact allowances shall not be applied to items in Group B. It is intended that impact be included as part of the loads transferred from superstructure to substructure, but shall not be included in loads transferred to footings nor to those parts of piles or columns that are below ground.

3.8.1.1 Group A—Impact shall be included.

(1) Superstructure, including legs of rigid frames.
(2) Piers, (with or without bearings regardless of type) excluding footings and those portions below the ground line.
(3) The portions above the ground line of concrete or steel piles that support the superstructure.

3.8.1.2 Group B—Impact shall not be included.

(1) Abutments, retaining walls, piles except as specified in 3.8.1.1 (3).
(2) Foundation pressures and footings.
(3) Timber structures.
(4) Sidewalk loads.
(5) Culverts and structures having 3 feet or more cover.

3.8.2. Impact Formula

3.8.2.1 The amount of the impact allowance or increment is expressed as a fraction of the live load stress, and shall be determined by the formula:

$$I = \frac{50}{L+125} \qquad (3\text{-}1)$$

in which,

I = impact fraction (maximum 30 percent);

3.10 CENTRIFUGAL FORCES

3.10.1 Structures on curves shall be designed for a horizontal radial force equal to the following percentage of the live load, without impact, in all traffic lanes:

$$C = 0.00117S^2D = \frac{6.68S^2}{R} \qquad (3\text{-}2)$$

where,

C = the centrifugal force in percent of the live load, without impact;
S = the design speed in miles per hour;
D = the degree of curve;
R = the radius of the curve in feet.

3.10.2 The effects of superelevation shall be taken into account.

3.10.3 The centrifugal force shall be applied 6 feet above the roadway surface, measured along the center line of the roadway. The design speed shall be determined with regard to the amount of superelevation provided in the roadway. The traffic lanes shall be loaded in accordance with the provisions of Article 3.7 with one standard truck on each design traffic lane placed in position for maximum loading.

3.10.4 Lane loads shall not be used in the computation of centrifugal forces.

3.10.5 When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the centrifugal forces acting on the live load.

3.11 APPLICATION OF LIVE LOAD

3.11.1 Traffic Lane Units

In computing stresses, each 10-foot lane load or single standard truck shall be considered as a unit, and fractions of load lane widths or trucks shall not be used.

3.11.2 Number and Position of Traffic Lane Units

The number and position of the lane load or truck loads shall be as specified in Article 3.7 and, whether lane or truck loads, shall be such as to produce maximum stress, subject to the reduction specified in Article 3.12.

3.11.3 Lane Loads on Continuous Spans

For the determination of maximum negative moment in the design of continuous spans, the lane load shown in Figure 3.7.6B shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect. For maximum positive moment, only one concentrated load shall be used per lane, combined with as many spans loaded uniformly as are required to produce maximum moment.

3.11.4 Loading for Maximum Stress

3.11.4.1 On both simple and continuous spans, the type of loading, whether lane load or truck load, to be used shall be the loading which produces the maximum stress. The moment and shear tables given in Appendix A show which types of loading controls for simple spans.

3.11.4.2 For continuous spans, the lane loading shall be continuous or discontinuous; only one standard H or HS truck per lane shall be considered on the structure.

3.12 REDUCTION IN LOAD INTENSITY

3.12.1 Where maximum stresses are produced in any member by loading a number of traffic lanes simultaneously, the following percentages of the live loads may be used in view of the improbability of coincident maximum loading:

	Percent
One or two lanes	100
Three lanes	90
Four lanes or more	75

3.12.2 The reduction in load intensity specified in Article 3.12.1 shall not be applicable when distribution factors from Table 3.23.1 are used to determine moments in longitudinal beams.

3.12.3 The reduction in intensity of loads on transverse members such as floor beams shall be determined as in the case of main trusses or girders, using the number of traffic lanes across the width of roadway that must be loaded to produce maximum stresses in the floor beam.

TABLE 3.23.1 Distribution of Wheel Loads in Longitudinal Beams

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or more Traffic Lanes
Timber:[a]		
Plank[b]	S/4.0	S/3.75
Nail laminated[c] 4" thick or multiple layer[d] floors over 5" thick	S/4.5	S/4.0
Nail laminated[c] 6" or more thick	S/5.0 If S exceeds 5' use footnote f.	S/4.25 If S exceeds 6.5' use footnote f.
Glued laminated[e] Panels on glued laminated stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 7.5' use footnote f.
On steel stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/5.25 If S exceeds 5.5' use footnote f.	S/4.5 If S exceeds 7' use footnote f.
Concrete:		
On steel I-Beam stringers[g] and prestressed concrete girders	S/7.0 If S exceeds 10' use footnote f.	S/5.5 If S exceeds 14' use footnote f.
On concrete T-Beams	S/6.5 If S exceeds 6' use footnote f.	S/6.0 If S exceeds 10' use footnote f.
On timber stringers	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 10' use footnote f.
Concrete box girders[h]	S/8.0 If S exceeds 12' use footnote f.	S/7.0 If S exceeds 16' use footnote f.
On steel box girders	See Article 10.39.2.	
On prestressed concrete spread box Beams	See Article 3.28.	
Steel grid:		
(Less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 10.5' use footnote f.
Steel bridge Corrugated plank[i]		
(2" min. depth)	S/5.5	S/4.5

S = average stringer spacing in feet.

[a]Timber dimensions shown are for nominal thickness.

[b]Plank floors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports (see Article 16.3.11—Division II).

[c]Nail laminated floors consist of pieces of lumber laid face to face with the narrow edges bearing on the supports, each piece being nailed to the preceding piece (see Article 16.3.12—Division II).

[d]Multiple layer floors consist of two or more layers of planks, each layer being laid at an angle to the other (see Article 16.3.11—Division II).

[e]Glued laminated panel floors consist of vertically glued laminated members with the narrow edges of the laminations bearing on the supports (see Article 16.3.13—Division II).

[f]In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam.

[g]"Design of I-Beam Bridges" by N. M. Newmark—Proceedings, ASCE, March 1948.

[h]The sidewalk live load (see Article 3.14) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.

[i]Distribution factors for Steel Bridge Corrugated Plank set forth above are based substantially on the following reference:

Journal of Washington Academy of Sciences, Vol. 67, No. 2, 1977 "Wheel Load Distribution of Steel Bridge Plank," by Conrad P. Heins, Professor of Civil Engineering, University of Maryland.

These distribution factors were developed based on studies using 6" × 2" steel corrugated plank. The factors should yield safe results for other corrugated configurations provided primary bending stiffness is the same as or greater than the 6" × 2" corrugated plank used in the studies.

3.22.4 When long span structures are being designed by load factor design, the gamma and beta factors specified for Load Factor Design represent general conditions and should be increased if, in the Engineer's judgment, expected loads, service conditions, or materials of construction are different from those anticipated by the specifications.

3.22.5 Structures may be analyzed for an overload that is selected by the operating agency. Size and configuration of the overload, loading combinations, and load distribution will be consistent with procedures defined in permit policy of that agency. The load shall be applied in Group IB as defined in Table 3.22.1A. For all loadings less than H 20, Group IA loading combination shall be used (see Article 3.5).

Part C
DISTRIBUTION OF LOADS

3.23 DISTRIBUTION OF LOADS TO STRINGERS, LONGITUDINAL BEAMS, AND FLOOR BEAMS*

3.23.1 Position of Loads for Shear

3.23.1.1 In calculating end shears and end reactions in transverse floor beams and longitudinal beams and stringers, no longitudinal distribution of the wheel load shall be assumed for the wheel or axle load adjacent to the transverse floor beam or the end of the longitudinal beam or stringer at which the stress is being determined.

3.23.1.2 Lateral distribution of the wheel loads at ends of the beams or stringers shall be that produced by

*Provisions in this Article shall not apply to orthotropic deck bridges.

assuming the flooring to act as a simple span between stringers or beams. For wheels or axles in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment, except that the calculations of horizontal shear in rectangular timber beams shall be in accordance with Article 13.3.

3.23.2 Bending Moments in Stringers and Longitudinal Beams**

3.23.2.1 General

In calculating bending moments in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows.

3.23.2.2 Interior Stringers and Beams

The live load bending moment for each interior stringer shall be determined by applying to the stringer the fraction of a wheel load (both front and rear) determined in Table 3.23.1.

3.23.2.3 Outside Roadway Stringers and Beams

3.23.2.3.1 Steel-Timber-Concrete T-Beams

3.23.2.3.1.1 The dead load supported by the outside roadway stringer or beam shall be that portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surface, if placed after the slab has cured, may be distributed equally to all roadway stringers or beams.

3.23.2.3.1.2 The live load bending moment for outside roadway stringers or beams shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams.

3.23.2.3.1.3 When the outside roadway beam or stringer supports the sidewalk live load as well as traffic live load and impact and the structure is to be designed by the service load method, the allowable stress in the beam or stringer may be increased by 25 percent for the combination of dead load, sidewalk live load, traffic live load, and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalks. When the combination of sidewalk live load and traffic live load plus impact governs the design and the structure is to be designed by the load factor method, 1.25 may be used as the beta factor in place of 1.67.

**In view of the complexity of the theoretical analysis involved in the distribution of wheel loads to stringers, the empirical method herein described is authorized for the design of normal highway bridges.

3.23.2.3.1.4 In no case shall an exterior stringer have less carrying capacity than an interior stringer.

3.23.2.3.1.5 In the case of a span with concrete floor supported by 4 or more steel stringers, the fraction of the wheel load shall not be less than:

$$\frac{S}{5.5}$$

where, S = 6 feet or less and is the distance in feet between outside and adjacent interior stringers, and

$$\frac{S}{4.0 + 0.25S}$$

where, S is more than 6 feet and less than 14 feet. When S is 14 feet or more, use footnote f, Table 3.23.1.

3.23.2.3.2 Concrete Box Girders

3.23.2.3.2.1 The dead load supported by the exterior girder shall be determined in the same manner as for steel, timber, or concrete T-beams, as given in Article 3.23.2.3.1.

3.23.2.3.2.2 The factor for the wheel load distribution to the exterior girder shall be $W_e/7$, where W_e is the width of exterior girder which shall be taken as the top slab width, measured from the midpoint between girders to the outside edge of the slab. The cantilever dimension of any slab extending beyond the exterior girder shall preferably not exceed half the girder spacing.

3.23.2.3.3 Total Capacity of Stringers and Beams

The combined design load capacity of all the beams and stringers in a span shall not be less than required to support the total live and dead load in the span.

3.23.3 Bending Moments in Floor Beams (Transverse)

3.23.3.1 In calculating bending moments in floor beams, no transverse distribution of the wheel loads shall be assumed.

3.23.3.2 If longitudinal stringers are omitted and the floor is supported directly on floor beams, the beams shall be designed for loads determined in accordance with Table 3.23.3.1.

where,

p = proportion of load carried by short span;
a = length of short span of slab;
b = length of long span of slab.

3.24.6.2 Where the length of the slab exceeds 1½ times its width, the entire load shall be carried by the transverse reinforcement.

3.24.6.3 The distribution width, E, for the load taken by either span shall be determined as provided for other slabs. The moments obtained shall be used in designing the center half of the short and long slabs. The reinforcement steel in the outer quarters of both short and long spans may be reduced by 50 percent. In the design of the supporting beams, consideration shall be given to the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams.

3.24.7 Median Slabs

Raised median slabs shall be designed in accordance with the provisions of this article with truck loadings so placed as to produce maximum stresses. Combined dead, live, and impact stresses shall not be greater than 150 percent of the allowable stresses. Flush median slabs shall be designed without overstress.

3.24.8 Longitudinal Edge Beams

3.24.8.1 Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb.

3.24.8.2 The edge beam of a simple span shall be designed to resist a live load moment of 0.10 PS, where,

P = wheel load in pounds P_{15} or P_{20};
S = span length in feet.

3.24.8.3 For continuous spans, the moment may be reduced by 20 percent unless a greater reduction results from a more exact analysis.

3.24.9 Unsupported Transverse Edges

The design assumptions of this article do not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges shall be supported by diaphragms or other suitable means. The diaphragms shall be designed to resist the full moment and shear produced by the wheel loads which can come on them.

3.24.10 Distribution Reinforcement

3.24.10.1 To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottoms of all slabs except culvert or bridge slabs where the depth of fill over the slab exceeds 2 feet.

3.24.10.2 The amount of distribution reinforcement shall be the percentage of the main reinforcement steel required for positive moment as given by the following formulas:

For main reinforcement parallel to traffic,

$$\text{Percentage} = \frac{100}{\sqrt{S}} \text{ Maximum } 50\% \qquad (3\text{-}21)$$

For main reinforcement perpendicular to traffic,

$$\text{Percentage} = \frac{220}{\sqrt{S}} \text{ Maximum } 67\% \qquad (3\text{-}22)$$

where, S = the effective span length in feet.

3.24.10.3 For main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement shall be used in the middle half of the slab span, and not less than 50 percent of the specified amount shall be used in the outer quarters of the slab span.

3.25 DISTRIBUTION OF WHEEL LOADS ON TIMBER FLOORING

For the calculation of bending moments in timber flooring each wheel load shall be distributed as follows.

3.25.1 Transverse Flooring

3.25.1.1 In the direction of flooring span, the wheel load shall be distributed over the width of tire as given in Article 3.30.

Normal to the direction of flooring span, the wheel load shall be distributed as follows:

Plank floor: the width of plank, but not less than 10 inches.

Non-interconnected* nail laminated panel floor: 15 inches, but not to exceed panel width.

Non-interconnected glued laminated panel floor: 15 inches plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated panel floor, with adequate shear transfer between panels**: 15 inches plus thickness of floor, but not to exceed panel width.

Interconnected* glued laminated panel floor, with adequate shear transfer between panels**, not less than 6 inches thick: 15 inches plus twice thickness of floor, but not to exceed panel width.

3.25.1.2 For transverse flooring the span shall be taken as the clear distance between stringers plus one-half the width of one stringer, but shall not exceed the clear span plus the floor thickness.

3.25.1.3 One design method for interconnected glued laminated panel floors is as follows: For glued laminated panel decks using vertically laminated lumber with the panel placed in a transverse direction to the stringers and with panels interconnected using steel dowels, the determination of the deck thickness shall be based on the following equations for maximum unit primary moment and shear.† The maximum shear is for a wheel position assumed to be 15 inches or less from the center line of the support. The maximum moment is for a wheel position assumed to be centered between the supports.

$$M_x = P(.51 \log_{10} s - K) \qquad (3\text{-}23)$$

$$R_x = .034P \qquad (3\text{-}24)$$

Thus,
$$t = \sqrt{\frac{6M_x}{F_b}} \qquad (3\text{-}25)$$

or,

$$t = \frac{3R_x}{2F_v} \textit{ whichever is greater} \qquad (3\text{-}26)$$

where,

M_x = primary bending moment in inch-pounds per inch;

R_x = primary shear in pounds per inch;

x = denotes direction perpendicular to longitudinal stringers;

P = design wheel load in pounds;

s = effective deck span in inches;

t = deck thickness, in inches, based on moment or shear, whichever controls;

K = design constant depending on design load as follows:

H 15 K = 0.47

H 20 K = 0.51

F_b = allowable bending stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B);

F_v = allowable shear stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B).

3.25.1.4 The determination of the minimum size and spacing required of the steel dowels required to transfer the load between panels shall be based on the following equation:

$$n = \frac{1,000}{\sigma_{PL}} \times \left[\frac{\overline{R}_y}{R_D} + \frac{\overline{M}_y}{M_D}\right] \qquad (3\text{-}27)$$

where,

n = number of steel dowels required for the given spans;

σ_{PL} = proportional limit stress perpendicular to grain (for Douglas Fir or Southern pine, use 1,000 psi);

$\overline{R_y}$ = total secondary shear transferred, in pounds, determined by the relationship:

$$\overline{R_y} = 6Ps/1,000 \text{ for } s \le 50 \text{ inches} \qquad (3\text{-}28)$$

or,

$$\overline{R_y} = \frac{P}{2s}(s-20) \text{ for } s > 50 \text{ inches} \qquad (3\text{-}29)$$

$\overline{M_y}$ = total secondary moment transferred, in inch-pound, determined by the relationship,

$$\overline{M_y} = \frac{Ps}{1,600}(s-10) \text{ for } s \le 50 \text{ inches} \qquad (3\text{-}30)$$

$$\overline{M_y} = \frac{Ps}{20}\frac{(s-30)}{(s-10)} \text{ for } s > 50 \text{ inches} \qquad (3\text{-}31)$$

R_D and M_D = shear and moment capacities, respectively, as given in the following table:

*The terms interconnected and non-interconnected refer to the joints between the individual nail laminated or glued laminated panels.

**This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or other suitable means.

†The equations are developed for deck panel spans equal to or greater than the width of the tire (as specified in Article 3.30), but not greater than 200 inches.

Diameter of Dowel	Shear Capacity R_D	Moment Capacity M_D	Steel Stress Coefficients C_R	C_M	Total Dowel Length Required
in.	lb.	in.-lb.	l/in.^{2}	l/in.^{3}	in.
0.5	600	850	36.9	81.5	8.50
.625	800	1,340	22.3	41.7	10.00
.75	1,020	1,960	14.8	24.1	11.50
.875	1,260	2,720	10.5	15.2	13.00
1.0	1,520	3,630	7.75	10.2	14.50
1.125	1,790	4,680	5.94	7.15	15.50
1.25	2,100	5,950	4.69	5.22	17.00
1.375	2,420	7,360	3.78	3.92	18.00
1.5	2,770	8,990	3.11	3.02	19.50

3.25.1.5 In addition, the dowels shall be checked to ensure that the allowable stress of the steel is not exceeded using the following equation:

$$\sigma = \frac{1}{n}(C_R \overline{R_y} + C_M \overline{M_y}) \qquad (3\text{-}32)$$

where,

σ = minimum yield point of steel pins in pounds per square inch (see Table 10.32.1A);
$n, \overline{R_y}, \overline{M_y}$ = as previously defined;
C_R, C_M = steel stress coefficients as given in preceding table.

3.25.2 Plank and Nail Laminated Longitudinal Flooring

3.25.2.1 In the direction of the span, the wheel load shall be distributed over 10 inches.

3.25.2.2 Normal to the direction of the span the wheel load shall be distributed as follows:

Plank floor: 20 inches;
Non-interconnected nail laminated floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated floor, with adequate shear transfer between panels*, not less than 6 inches thick: width of tire plus twice thickness of floor.

3.25.2.3 For longitudinal flooring the span shall be taken as the clear distance between floor beams plus one-half the width of one beam but shall not exceed the clear span plus the floor thickness.

*This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or spreader beams located at intervals along the panels or other suitable means.

3.25.3 Longitudinal Glued Laminated Timber Decks

3.25.3.1 Bending Moment

In calculating bending moments in glued laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

TWO OR MORE TRAFFIC LANES

$$\text{Load Fraction} = \frac{W_p}{3.75 + \frac{L}{28}} \text{ or } \frac{W_p}{5.00}, \text{ whichever is}$$

greater.

ONE TRAFFIC LANE

$$\text{Load Fraction} = \frac{W_p}{4.25 + \frac{L}{28}} \text{ or } \frac{W_p}{5.50}, \text{ whichever is}$$

greater.

where, W_p = Width of Panel; in feet ($3.5 \le W_p \le 4.5$)

L = Length of span for simple span bridges and the length of the shortest span for continuous bridges in feet.

3.25.3.2 Shear (Glue Laminated)

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel

$$= \frac{W_p}{4.00} \text{ but not less than 1.}$$

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.2 Shear

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel

$$= \frac{W_p}{4.00} \text{ but not less than 1.}$$

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.3 Deflections

The maximum deflection may be calculated by applying to the panel the wheel load fraction determined by the method prescribed for moment.

3.25.3.4 Stiffener Arrangement

The transverse stiffeners shall be adequately attached to each panel, at points near the panel edges, with either steel plates, thru-bolts, C-clips or aluminum brackets. The stiffener spacing required will depend upon the spacing needed in order to prevent differential panel movement; however, a stiffener shall be placed at mid-span with additional stiffeners placed at intervals not to exceed 10 feet. The stiffness factor EI of the stiffener shall not be less than 80,000 kip-in^2.

3.25.4 Continuous Flooring

If the flooring is continuous over more than two spans, the maximum bending moment shall be assumed as being 80% of that obtained for a simple span.

3.26 DISTRIBUTION OF WHEEL LOADS AND DESIGN OF COMPOSITE WOOD-CONCRETE MEMBERS

3.26.1 Distribution of Concentrated Loads for Bending Moment and Shear

3.26.1.1 For freely supported or continuous slab spans of composite wood-concrete construction, as described in Article 16.3.14, Division II, the wheel loads shall be distributed over a transverse width of 5 feet for bending moment and a width of 4 feet for shear.

3.26.1.2 For composite T-beams of wood and concrete, as described in Article 20.19.2, Division II, the effective flange width shall not exceed that given in Article 10.38.3. Shear connectors shall be capable of resisting both vertical and horizontal movement.

3.26.2 Distribution of Bending Moments in Continuous Spans

3.26.2.1 Both positive and negative moments shall be distributed in accordance with the following table:

Maximum Bending Moments—Percent of Simple Span Moment

	Maximum Uniform Dead Load Moments				Maximum Live Load Moments			
	Wood Subdeck		Composite Slab		Concentrated Load		Uniform Load	
Span	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
Interior	50	50	55	45	75	25	75	55
End	70	60	70	60	85	30	85	65
2-Span[a]	65	70	60	75	85	30	80	75

[a] Continuous beam of 2 equal spans.

3.26.2.2 Impact should be considered in computing stresses for concrete and steel, but neglected for wood.

3.26.3 Design

The analysis and design of composite wood-concrete members shall be based on assumptions that account for the different mechanical properties of the components. A suitable procedure may be based on the elastic properties of the materials as follows:

$\frac{E_c}{E_w}$ = 1 for slab in which the net concrete thickness is less than half the overall depth of the composite section

$\frac{E_c}{E_w}$ = 2 for slab in which the net concrete thickness is at least half the overall depth of the composite section

$\frac{E_s}{E_w}$ = 18.75 (for Douglas fir and Southern pine)

in which,

E_c = modulus of elasticity of concrete;
E_w = modulus of elasticity of wood;
E_s = modulus of elasticity of steel.

3.27 DISTRIBUTION OF WHEEL LOADS ON STEEL GRID FLOORS*

3.27.1 General

3.27.1.1 The grid floor shall be designed as continuous, but simple span moments may be used and reduced as provided in Article 3.24.

3.27.1.2 The following rules for distribution of loads assume that the grid floor is composed of main elements that span between girders, stringers, or cross beams, and secondary elements that are capable of transferring load between the main elements.

3.27.1.3 Reinforcement for secondary elements shall consist of bars or shapes welded to the main steel.

3.27.2 Floors Filled with Concrete

3.27.2.1 The distribution and bending moment shall be as specified for concrete slabs, Article 3.24. The following items specified in that article shall also apply to concrete filled steel grid floors:

Longitudinal edge beams
Unsupported transverse edges
Span lengths

3.27.2.2 The strength of the composite steel and concrete slab shall be determined by means of the "transformed area" method. The allowable stresses shall be as set forth in Articles 8.15.2, 8.16.1, and 10.32.

3.27.3 Open Floors

3.27.3.1 A wheel load shall be distributed, normal to the main elements, over a width equal to 1¼ inches per ton of axle load plus twice the distance center to center of main elements. The portion of the load assigned to each main element shall be applied uniformly over a length equal to the rear tire width (20 inches for H 20, 15 inches for H 15).

3.27.3.2 The strength of the section shall be determined by the moment of inertia method. The allowable stresses shall be as set forth in Article 10.32.

3.27.3.3 Edges of open grid steel floors shall be supported by suitable means as required. These supports may be longitudinal or transverse, or both, as may be required to support all edges properly.

3.27.3.4 When investigating for fatigue, the minimum cycles of maximum stress shall be used.

3.28 DISTRIBUTION OF LOADS FOR BENDING MOMENT IN SPREAD BOX GIRDERS**

3.28.1 Interior Beams

The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel load (both front and rear) determined by the following equation:

$$D.F. = \frac{2N_L}{N_B} + k\frac{S}{L} \qquad (3\text{-}33)$$

where,

N_L = number of design traffic lanes (Article 3.6);
N_B = number of beams ($4 \le N_B \le 10$);
S = beam spacing in feet ($6.57 \le S \le 11.00$);
L = span length in feet;
k = $0.07\,W - N_L\,(0.10N_L - 0.26) - 0.20N_B - 0.12$; (3-34)
W = numeric value of the roadway width between curbs expressed in feet ($32 \le W \le 66$).

3.28.2 Exterior Beams

The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than $2N_L/N_B$.

3.29 MOMENTS, SHEARS, AND REACTIONS

Maximum moments, shears, and reactions are given in tables, Appendix A, for H 15, H 20, HS 15, and HS 20 loadings. They are calculated for the standard truck or the lane loading applied to a single lane on freely supported spans. It is indicated in the table whether the standard truck or the lane loadings produces the maximum stress.

*Provisions in this article shall not apply to orthotropic bridge superstructures.

**The provisions of Article 3.12, Reduction in Load Intensity, were not applied in the development of the provisions presented in Articles 3.28.1 and 3.28.2.

3.30 TIRE CONTACT AREA

The tire contact area for the Alternate Military Loading or HS 20-44 shall be assumed as a rectangle with a length in the direction of traffic of 10 inches, and a width of tire of 20 inches. For other design vehicles, the tire contact should be determined by the engineer.

The following items specified in that article shall also apply to concrete filled steel grid floors:

Longitudinal edge beams
Unsupported transverse edges
Span lengths

3.27.2.2 The strength of the composite steel and concrete slab shall be determined by means of the "transformed area" method. The allowable stresses shall be as set forth in Articles 8.15.2, 8.16.1, and 10.32.

3.27.3 Open Floors

3.27.3.1 A wheel load shall be distributed, normal to the main elements, over a width equal to 1 1/4 inches per ton of axle load plus twice the distance center to center of main elements. The portion of the load assigned to each main element shall be applied uniformly over a length equal to the rear tire width (20 inches for H20, 15 inches for H15).

3.27.3.2 The strength of the section shall be determined by the moment of inertia method. The allowable stresses shall be as set forth in Article 10.32.

3.27.3.3 Edges of open grid steel floors shall be supported by suitable means as required. These supports may be longitudinal or transverse, or both, as may be required to support all edges properly.

3.27.3.4 When investigating for fatigue, the minimum cycles of maximum stress shall be used.

3.28 DISTRIBUTION OF LOADS FOR BENDING MOMENT IN SPREAD BOX GIRDERS*

3.28.1 Interior Beams

The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel load (both front and rear) determined by the following equation:

$$D.F. = \frac{2N_L}{N_B} + k\frac{S}{L} \qquad (3\text{-}33)$$

where

N_L = number of design traffic lanes (Article 3.6);
N_B = number of beams ($4 \le N_B \le 10$);
S = beam spacing in feet ($6.57 \le S \le 11.00$);
L = span length in feet;
$k = 0.07\,W - N_L\,(0.10N_L - 0.26) - 0.20N_B - 0.12$; (3-34)
W = numeric value of the roadway width between curbs expressed in feet ($32 \le W \le 66$).

3.28.2 Exterior Beams

The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than $2N_L/N_B$.

3.29 MOMENTS, SHEARS, AND REACTIONS

Maximum moments, shears, and reactions are given in tables, Appendix A, for H 15, H 20, HS 15, and HS 20 loadings. They are calculated for the standard truck or the lane loading applied to a single lane on freely supported spans. It is indicated in the table whether the standard truck or the lane loadings produces the maximum stress.

*The provisions of Article 3.12, Reduction in Load Intensity, were not applied in the development of the provisions presented in 3.28.1 and 3.28.2.

3.30 TIRE CONTACT AREA

The tire contact area shall be assumed as a rectangle with an area in square inches of 0.01P, and a Length in Direction of Traffic/Width of Tire ratio of 1/2.5, in which P = wheel load in pounds.

FROM AASHTO 1986, USE FOR OTHER DESIGN VEHICLES.

FROM: 2002 AASHTO STD. SPECS

Section 13
WOOD STRUCTURES

13.1 GENERAL AND NOTATIONS

13.1.1 General

The following information on wood design is generally based on the National Design Specification for Wood Construction (NDS®), 1991 Edition. See the 1991 Edition of the NDS® for additional information.

13.1.2 Net Section

In determining the capacity of wood members, the net section of the member shall be used. Unless otherwise noted, the net section shall be determined by deducting from the gross section, the projected area of all material removed by boring, grooving, dapping, notching or other means.

13.1.3 Impact

In calculating live load stresses in wood, impact shall be neglected unless otherwise noted. See Article 3.8.1.

13.1.4 Notations

a = coefficient based on support conditions for tapered columns (Article 13.7.3.4.2)

b = width of bending member (Article 13.6.4.3)

c = coefficient based on sawn lumber, round timber piles, glued laminated timber or structural composite lumber (Article 13.7.3.3.5)

C_D = load duration factor (Article 13.5.5.2)

C_F = bending size factor for sawn lumber, structural composite lumber, and for glued laminated timber with loads applied parallel to the wide face of the laminations (Article 13.6.4.2)

C_F = compression size factor for sawn lumber (footnotes to Table 13.5.1A)

C_F = tension size factor for sawn lumber (footnotes to Table 13.5.1A) and structural composite lumber (footnotes to Tables 13.5.4A and 13.5.4B)

C_H = sheer stress factor (footnotes to Table 13.5.1A)

C_L = beam stability factor (Article 13.6.4.4)

C_M = wet service factor (Article 13.5.5.1)

C_P = column stability factor (Article 13.7.3.3)

C_V = volume factor for glued laminated timber with loads applied perpendicular to the wide face of the laminations (Article 13.6.4.3)

C_b = bearing area factor (Article 13.6.6.3)

C_f = form factor (Article 13.6.4.5)

C_{fu} = flat use factor for sawn lumber (footnotes to Table 13.5.1A)

C_r = repetitive member factor for sawn lumber (footnotes to Table 13.5.1A)

d = depth of member (Article 13.6.4.2.2)

d_{max} = maximum column face dimension (Article 13.7.3.4.2)

d_{min} = minimum column face dimension (Article 13.7.3.4.2)

d_{rep} = representative dimension for a tapered column face (Article 13.7.3.4.2)

E = tabulated modulus of elasticity (Article 13.6.3)

E' = allowable modulus of elasticity (Article 13.6.3)

F_b = tabulated unit stress in bending (Article 13.6.4.1)

F'_b = allowable unit stress in bending (Article 13.6.4.1)

F^*_b = adjusted tabulated bending stress for beam stability (Article 13.6.4.4.5)

F_c = tabulated unit stress in compression parallel to grain (Article 13.7.3.2)

F'_c = allowable unit stress in compression parallel to grain (Article 13.7.3.2)

F_c* = adjusted tabulated stress in compression parallel to grain for column stability (Article 13.7.3.3.5)

f_c = actual unit stress in compression parallel to grain (Article 13.7.3.1)

$F_{c\perp}$ = tabulated unit stress in compression perpendicular to grain (Article 13.6.6.2)

$F'_{c\perp}$ = allowable unit stress in compression perpendicular to grain (Article 13.6.6.2)

F_g = tabulated unit stress in bearing parallel to grain (Article 13.7.4.1)

F'_g = allowable unit stress in bearing parallel to grain (Article 13.7.4.1)

F_t = tabulated unit stress in tension parallel to grain (Article 13.8.1)
F'_t = allowable unit stress in tension parallel to grain (Article 13.8.1)
F_v = tabulated unit stress in shear parallel to grain (Article 13.6.5.3)
F'_v = allowable unit stress in shear parallel to grain (Article 13.6.5.3)
f_v = actual unit stress in shear parallel to grain (Article 13.6.5.2)
$F_{\theta'}$ = allowable unit stress for bearing on an inclined surface (Article 13.6.7)
K = column effective length factor (Article 13.7.3.3.3)
K_{bE} = material factor for beam stability (Article 13.6.4.4.5)
K_{cE} = material factor for column stability (Article 13.7.3.3.5)
L = length of bending member between points of zero moment (Article 13.6.4.3.1)
l = actual column length between points of lateral support (Article 13.7.3.3.3)
l_b = length of bearing (Article 13.6.6.3)
l_e = effective bending member length (Article 13.6.4.4.3)
l_e = effective column length (Article 13.7.3.3.3)
l_u = unsupported bending member length (Article 13.6.4.4.3)
m = parameter for the specific material determined in accordance with the requirements of ASTM D-5456 (Tables 13.5.4A and 13.5.4B)
R_B = bending member slenderness ratio (Article 13.6.4.4.4)
V = vertical shear (Article 13.6.5.2)
V_{LD} = maximum vertical shear at 3d or L/4 due to wheel loads distributed laterally as specified for moment (Article 13.6.5.2)
V_{LL} = distributed live load vertical shear (Article 13.6.5.2)
V_{LU} = maximum vertical shear at 3d or L/4 due to undistributed wheel loads (Article 13.6.5.2)
x = species variable for computing the volume factor (Article 13.6.4.3.1)
θ = angle between the direction of load and the direction of grain (Article 13.6.7)

13.2 MATERIALS

13.2.1 Sawn Lumber

13.2.1.1 General

Sawn lumber shall comply with the requirements of AASHTO M 168.

13.2.1.2 Dimensions

13.2.1.2.1 Structural calculations for sawn lumber shall be based on the net dimensions of the member for the anticipated use conditions. These net dimensions depend on the type of surfacing, whether dressed, rough-sawn or full-sawn.

13.2.1.2.2 For dressed lumber, the net dry dimensions given in Table 13.2.1A shall be used for design, regardless of the moisture content at the time of manufacture or in use.

13.2.1.2.3 Where the design is based on rough, full-sawn or special sizes, the applicable moisture content and dimensions used in design shall be noted in the plans and specifications.

TABLE 13.2.1A Net Dry Dimensions for Dressed Lumber

Nominal Thickness	Dry Thickness	Nominal Width	Dry Width
Dimension Lumber (inches):			
2	1-1/2	2	1-1/2
2-1/2	2	3	2-1/2
3	2-1/2	4	3-1/2
3-1/2	3	5	4-1/2
4	3-1/2	6	5-1/2
4-1/2	4	8	7-1/4
		10	9-1/4
		12	11-1/4
		14	13-1/4
		16	15-1/4
Beams and Stringers and Posts and Timbers (inches):			
5 and greater	1/2 less than nominal	5 and greater	1/2 less than nominal

13.2.2 Glued Laminated Timber

13.2.2.1 General

Glued laminated timber shall comply with the requirements of AASHTO M 168 and shall be manufactured using wet-use adhesives.

13.2.2.2 Dimensions

13.2.2.2.1 Structural calculations for glued laminated timber shall be based on the net finished dimensions.

13.4.3 Members having simple or continuous spans preferably should be designed so that the deflection due to service live load does not exceed 1/500 of the span.

13.4.4 For timber deck structures with timber girders or stringers of equal stiffness, and cross-bracing or diaphragms sufficient in depth and strength to ensure lateral distribution of loads, the deflection may be computed by considering all girders or stringers as acting together and having equal deflection. When the cross-bracing or diaphragms are not sufficient to laterally distribute loads, deflection shall be distributed as specified for moment.

13.4.5 For concrete decks on wood girders or stringers, the deflection shall be assumed to be resisted by all beams or stringers equally.

13.5 DESIGN VALUES

13.5.1 General

Stress and modulus of elasticity values used for design, referred to as allowable design values, shall be the tabulated values modified by all applicable adjustments required by this Section. The actual stress due to loading shall not exceed the allowable stress.

13.5.2 Tabulated Values for Sawn Lumber

13.5.2.1 Tabulated values for sawn lumber are given in Table 13.5.1A for visually graded lumber and Table 13.5.1B for mechanically graded lumber. Values for bearing parallel to grain are given in Table 13.5.2A. These values are taken from the 1991 Edition of the NDS® and represent a partial listing of available species and grades. Refer to the 1991 Edition of the NDS® for a more complete listing.

13.5.2.2 Stress Grades in Flexure

13.5.2.2.1 The tabulated unit bending stress for Dimension (2 to 4 inches thick) and Post and Timber grades applies to material with the load applied either to the narrow or wide face.

13.5.2.2.2 The tabulated unit bending stress for Decking grades applies only when the load is applied to the wide face.

13.5.2.2.3 The tabulated unit bending stress for Beam and Stringer grades applies only when the load is applied to the narrow face. When Post and Timber sizes are graded to Beam and Stringer grade requirements, the tabulated unit bending stress for the applicable Beam and Stringer grades may be used.

13.5.2.2.4 Beam and Stringer grades are normally graded for use as a single, simple span. When used as a continuous beam, the grading provisions customarily applied to the middle third of the simple span length shall be applied to the middle two-thirds of the length for two-span beams, and to the entire length for beams continuous over three or more spans.

13.5.3 Tabulated Values for Glued Laminated Timber

13.5.3.1 Tabulated values for glued laminated timber of softwood species are given in Tables 13.5.3A and 13.5.3B. Values for bearing parallel to grain are given in Table 13.5.2A. These values are taken from the 1993 Edition of the American Institute of Timber Construction, AITC 117-93 Design, "Standard Specifications for Structural Glued Laminated Timber of Softwood Species." Refer to AITC 117-93 Design for a more complete listing.

13.5.3.2 Tabulated values for hardwood species shall be as given in the 1985 Edition of American Institute of Timber Construction, AITC 119, "Standard Specifications for Hardwood Glued Laminated Timber."

13.5.3.3 Species other than those specifically included or referenced in this Section may be used, provided that tabulated values are established for each species in accordance with AASHTO M 168.

13.5.4 Tabulated Values for Structural Composite Lumber

13.5.4.1 Representative tabulated design values for structural composite lumber are given in Table 13.5.4A for laminated veneer lumber and Table 13.5.4B for parallel strand lumber.

13.5.5 Adjustments to Tabulated Design Values

13.5.5.1 Wet Service Factor, C_M

13.5.5.1.1 Tabulated values for sawn lumber assume that the material is installed and used under continuously dry conditions where the moisture content of the wood does not exceed 19 percent. When the moisture content at installation or in service is expected to exceed 19 percent, tabulated values shall be reduced by the wet service fac-

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers

Species and Commercial Grade	Size Classification	Design Values in Pounds per Square Inch (psi)						Grading Rules Agency
		Bending F_b	Tension Parallel to Grain F_t	Shear Parallel to Grain F_v	Compression Perpendicular to Grain $F_{c\perp}$	Compression Parallel to Grain F_c	Modulus of Elasticity E	
DOUGLAS FIR-LARCH								
Select Structural		1450	1000	95	625	1700	1,900,000	
No. 1 & Btr	2″–4″ thick	1150	775	95	625	1500	1,800,000	WWPA
No. 1		1000	675	95	625	1450	1,700,000	
No. 2	2″ & wider	875	575	95	625	1300	1,600,000	WCLIB
Dense Select Structural		1900	1100	85	730	1300	1,700,000	
Select Structural	Beams and	1600	950	85	625	1100	1,600,000	
Dense No. 1	Stringers	1550	775	85	730	1100	1,700,000	
No. 1		1350	675	85	625	925	1,600,000	
No. 2		875	425	85	625	600	1,300,000	WCLIB
Dense Select Structural		1750	1150	85	730	1350	1,700,000	
Select Structural	Posts and	1500	1000	85	625	1150	1,600,000	
Dense No. 1	Timbers	1400	950	85	730	1200	1,700,000	
No. 1		1200	825	85	625	1000	1,600,000	
No. 2		750	475	85	625	700	1,300,000	
Dense Select Structural		1850	1100	85	730	1300	1,700,000	
Select Structural		1600	950	85	625	1100	1,600,000	
Dense No. 1	Beams and	1550	775	85	730	1100	1,700,000	
No. 1	Stringers	1350	675	85	625	925	1,600,000	
Dense No. 2		1000	500	85	730	700	1,400,000	
No. 2		875	425	85	625	600	1,300,000	WWPA
Dense Select Structural		1750	1150	85	730	1350	1,700,000	
Select Structural		1500	1000	85	625	1150	1,600,000	
Dense No. 1	Posts and	1400	950	85	730	1200	1,700,000	
No. 1	Timbers	1200	825	85	625	1000	1,600,000	
Dense No. 2		800	550	85	730	550	1,400,000	
No. 2		700	475	85	625	475	1,300,000	
EASTERN SOFTWOODS								
Select Structural		1250	575	70	335	1200	1,200,000	NELMA
No. 1	2″–4″ thick	775	350	70	335	1000	1,100,000	
No. 2	2″ & wider	575	275	70	335	825	1,100,000	NSLB

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued)

Species and Commercial Grade	Size Classification	Design Values in Pounds per Square Inch (psi)						Grading Rules Agency
		Bending F_b	Tension Parallel to Grain F_t	Shear Parallel to Grain F_v	Compression Perpendicular to Grain $F_{c\perp}$	Compression Parallel to Grain F_c	Modulus of Elasticity E	
RED OAK								
Select Structural		1150	675	85	820	1000	1,400,000	
No. 1	2″–4″ thick	825	500	85	820	825	1,300,000	
No. 2	2″ & wider	800	475	85	820	625	1,200,000	
Select Structural	Beams and	1350	800	80	820	825	1,200,000	
No. 1	Stringers	1150	550	80	820	700	1,200,000	
No. 2		725	375	80	820	450	1,000,000	NELMA
Select Structural	Posts and	1250	850	80	820	875	1,200,000	
No. 1	Timbers	1000	675	80	820	775	1,200,000	
No. 2		575	400	80	820	350	1,000,000	
SOUTHERN PINE								
Select Structural		2850	1600	100	565	2100	1,800,000	SPIB
No. 1	2″–4″ thick	1850	1050	100	565	1850	1,700,000	
No. 2	2″–4″ wide	1500	825	90	565	1650	1,600,000	
Select Structural		2550	1400	90	565	2000	1,800,000	
No. 1	2″–4″ thick	1650	900	90	565	1750	1,700,000	
No. 2	5″–6″ wide	1250	725	90	565	1600	1,600,000	
Select Structural		2300	1300	90	565	1900	1,800,000	
No. 1	2″–4″ thick	1500	825	90	565	1650	1,700,000	
No. 2	8″ wide	1200	650	90	565	1550	1,600,000	
Select Structural	2″–4″ thick	2050	1100	90	565	1850	1,800,000	
No. 1	10″ wide	1300	725	90	565	1600	1,700,000	
No. 2		1050	575	90	565	1500	1,600,000	
Select Structural		1900	1050	90	565	1800	1,800,000	
No. 1	2″–4″ thick	1250	675	90	565	1600	1,700,000	
No. 2	12″ wide	975	550	90	565	1450	1,600,000	

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued)

1. Design values are taken from the 1991 Edition of the NDS® and are for a 10-year load duration and dry service conditions. Refer to the 1991 NDS® for additional species and grades and for a summary of grading rules agencies and commercial species classifications.

2. Wet Service Factor, C_M. When dimension lumber, 2" to 4" thick is used where moisture content will exceed 19%, design values shall be multiplied by the following wet service factors:

WET SERVICE FACTORS, C_M

F_b	F_t	F_v	$F_{c\perp}$	F_c	E
0.85*	1.0	0.97	0.67	0.8**	0.9

* when $(F_b)(C_F) \leq 1{,}150$ psi, $C_M = 1.0$
** when $F_c \leq 750$ psi, $C_M = 1.0$

When timbers 5" by 5" and larger are used where moisture content will exceed 19%, design values shall be multiplied by the following wet service factors (for Southern Pine and Mixed Southern Pine, use tabulated values without further adjustment):

WET SERVICE FACTORS, C_M

F_b	F_t	F_v	$F_{c\perp}$	F_c	E
1.00	1.00	1.00	0.67	0.91	1.00

3. Size Factor, C_F. For all species other than Southern Pine and Mixed Southern Pine, tabulated bending, tension, and compression parallel to grain design values for dimension lumber 2" to 4" thick shall be multiplied by the following size factors:

SIZE FACTORS, C_F

		F_b		F_t	F_c
		Thickness			
Grades	Width	2" & 3"	4"		
Select Structural, No. 1 & Btr. No. 1, No. 2, No. 3	2", 3" & 4"	1.5	1.5	1.5	1.15
	5"	1.4	1.4	1.4	1.1
	6"	1.3	1.3	1.3	1.1
	8"	1.2	1.3	1.2	1.05
	10"	1.1	1.2	1.1	1.0
	12"	1.0	1.1	1.0	1.0
	14" & wider	0.9	1.0	0.9	0.9

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued)

For Southern Pine and Mixed Southern Pine dimension lumber, 2″ to 4″ thick, appropriate size adjustment factors have been incorporated in tabulated values, with the following exceptions:

For dimension lumber 4″ thick, 8″ and wider, tabulated bending design values shall be multiplied by the size factor, $C_F = 1.1$.

For dimension lumber wider than 12″, tabulated bending, tension, and compression parallel to grain design values for 12″ wide lumber shall be multiplied by the size factor, $C_F = 0.9$.

4. Flat Use Factor, C_{fu}. Bending design values are based on edgewise use (load applied to narrow face). When dimension lumber 2″ to 4″ thick is used flatwise (load applied to wide face), the bending design value shall be multiplied by the following flat use factors:

FLAT USE FACTORS, C_{fu}

	Thickness	
Width	2″ & 3″	4″
2″ & 3″	1.0	...
4″	1.1	1.0
5″	1.1	1.05
6″	1.15	1.05
8″	1.15	1.05
10″ & wider	1.2	1.1

5. Repetitive Member Factor, C_r. Bending design values for dimension lumber 2″ to 4″ thick shall be multiplied by the repetitive member factor $C_r = 1.15$, when such members are used as stringers, decking or similar members which are in contact or are spaced not more than 24″ on centers, are not less than 3 in number and are joined by load distributing elements adequate to support the design load.

6. Shear Stress Factor, C_H. Tabulated shear design values parallel to grain, F_v, have been reduced to allow for the occurrence of splits, checks, and shakes and may be multiplied by the shear stress factors given below when the length of split, or size of check or shake is known and no increase in them is anticipated. When the shear stress factor is applied to Southern Pine or Mixed Southern Pine, a tabulated design value of $F_v = 90$ lb/in.^{2} shall be used for all grades. Shear stress factors shall be linearly interpolated.

SHEAR STRESS FACTORS, C_H

Length of split on wide face of 2″ (nominal) lumber	C_H	Length of split on wide face of 3″ (nominal) and thicker lumber	C_H	Size of shake* in 2″ (nominal) and thicker lumber	C_H
no split	2.00	no split	2.00	no shake	2.00
1/2 × wide face	1.67	1/2 × narrow face	1.67	1/6 × narrow face	1.67
3/4 × wide face	1.50	3/4 × narrow face	1.50	1/4 × narrow face	1.50
1 × wide face	1.33	1 × narrow face	1.33	1/3 × narrow face	1.33
1-1/2 × wide face or more	1.00	1-1/2 × narrow face or more	1.00	1/2 × narrow face or more	1.00

* Shake is measured at the end between lines enclosing the shake and perpendicular to the loaded face.

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TABLE 13.5.1B Tabulated Design Values for Mechanically Graded Dimension Lumber

Species and Commercial Grade	Size Classification	Design Values in Pounds per Square Inch (psi) Bending F_b	Tension Parallel to Grain F_t	Compression Parallel to Grain F_c	Modulus of Elasticity E	Grading Rules Agency
MACHINE STRESS RATED (MSR) LUMBER						
900f-1.0E		900	350	1050	1,000,000	WCLIB, WWPA
1200f-1.2E		1200	600	1400	1,200,000	NLGA, SPIB, WCLIB, WWPA
1350f-1.3E		1350	750	1600	1,300,000	SPIB, WCLIB, WWPA
1450f-1.3E		1450	800	1625	1,300,000	NLGA, WCLIB, WWPA
1500f-1.3E		1500	900	1650	1,300,000	SPIB
1500f-1.4E		1500	900	1650	1,400,000	NLGA, SPIB, WCLIB, WWPA
1650f-1.4E		1650	1020	1700	1,400,000	SPIB
1650f-1.5E		1650	1020	1700	1,500,000	NLGA, SPIB, WCLIB, WWPA
1800f-1.6E		1800	1175	1750	1,600,000	NLGA, SPIB, WCLIB, WWPA
1950f-1.5E	2" & less in thickness	1950	1375	1800	1,500,000	SPIB
1950f-1.7E		1950	1375	1800	1,700,000	NLGA, SPIB, WWPA
2100f-1.8E	2" & wider	2100	1575	1875	1,800,000	NLGA, SPIB, WCLIB, WWPA
2250f-1.6E		2250	1750	1925	1,600,000	SPIB
2250f-1.9E		2250	1750	1925	1,900,000	NLGA, SPIB, WWPA
2400f-1.7E		2400	1925	1975	1,700,000	SPIB
2400f-2.0E		2400	1925	1975	2,000,000	NLGA, SPIB, WCLIB, WWPA
2550f-2.1E		2550	2060	2025	2,100,000	NLGA, SPIB, WWPA
2700f-2.2E		2700	2150	2100	2,200,000	NLGA, SPIB, WCLIB, WWPA
2850f-2.3E		2850	2300	2150	2,300,000	SPIB, WWPA
3000f-2.4E		3000	2400	2200	2,400,000	NLGA, SPIB
3150f-2.5E		3150	2500	2250	2,500,000	SPIB
3300f-2.6E		3300	2650	2325	2,600,000	SPIB
900f-1.2E		900	350	1050	1,200,000	NLGA, WCLIB
1200f-1.5E	2" & less in thickness	1200	600	1400	1,500,000	NLGA, WCLIB
1350f-1.8E		1350	750	1600	1,800,000	NLGA
1500f-1.8E	6" & wider	1500	900	1650	1,800,000	WCLIB
1800f-2.1E		1800	1175	1750	2,100,000	NLGA, WCLIB

1. Design values are taken from the 1991 Edition of the NDS® and are for a 10-year load duration and dry service conditions. Refer to the 1991 NDS® for additional grades and for a summary of grading rules agencies.
2. Design values for shear parallel to grain and compression perpendicular to grain shall be as specified in Table 13.5.1A for No. 2 visually graded dimension lumber of the appropriate species.
3. Use of the wet service factor, shear stress factor, repetitive member factor, and flat use factor shall be as specified in Table 13.5.1A for visually graded dimension lumber.

tors, C_M, given in footnotes to Tables 13.5.1A and 13.5.1B.

13.5.5.1.2 Tabulated values for glued laminated timber and structural composite lumber assume that the material is used under continuously dry conditions where the moisture content in service does not exceed 16 percent. When the moisture content in service is expected to exceed 16 percent, tabulated values shall be reduced by the wet service factors, C_M, given in the footnotes to Tables 13.5.3A and 13.5.3B for glued laminated timber and Tables 13.5.4A and 13.5.4B for structural composite lumber.

13.5.5.1.3 The moisture content of wood used in exposed bridge applications will normally exceed 19 percent and tabulated values shall be reduced by the wet service factor unless an analysis of regional, geographic, and climatological conditions that affect moisture content indicate that the in-service moisture content will not exceed 19 percent for sawn lumber and 16 percent for glued laminated timber and structural composite lumber over the life of the structure.

TABLE 13.5.2A Tabulated Design Values for Bearing Parallel to Grain

Species Combination	Wet Service Conditions	Dry Service Conditions		
		Sawn Lumber		
		5" × 5" & Larger	2" to 4" Thick	Glued Laminated Timber
Douglas Fir-Larch (Dense)	1570	1730	2360	2750
Douglas Fir-Larch	1350	1480	2020	2360
Eastern Softwoods	880	—	1340	—
Hem-Fir	1110	1220	1670	1940
Mixed Southern Pine	1270	1390	1900	—
Northern Red Oak	1150	1270	1730	2010
Red Maple	1100	1210	1650	1930
Red Oak	1010	1110	1520	1770
Southern Pine	1320	1450	1970	2300
Southern Pine (Dense)	1540	1690	2310	2690
Spruce-Pine-Fir	940	1040	1410	1650
Spruce-Pine-Fir (South)	810	900	1220	1430
Yellow Poplar	890	—	1340	1560

1. Design values are taken from the 1991 Edition of the NDS®. Refer to the 1991 NDS® for additional species.
2. Wet and dry service conditions are as defined in Article 13.5.5.1. The wet service factor has been applied to values tabulated for wet service conditions and further adjustment by this factor is not required.

13.5.5.2 Load Duration Factor, C_D

13.5.5.2.1 Wood can sustain substantially greater maximum loads for short load durations than for long load durations. Tabulated stresses for sawn lumber, glued laminated timber, and structural composite lumber are based on a normal load duration which contemplates that the member is stressed to the maximum stress level, either continuously or cumulatively, for a period of approximately 10 years, and/or stressed to 90 percent of the maximum design level continuously for the remainder of the member life.

13.5.5.2.2 When the full maximum load is applied either cumulatively or continuously for periods other than 10 years, tabulated stresses shall be multiplied by the load duration factor, C_D, given in Table 13.5.5A.

13.5.5.2.3 The provisions of this article do not apply to modulus of elasticity or to compression perpendicular to grain, but do apply to mechanical fastenings, except as otherwise noted. The load duration factor for impact does not apply to members pressure-impregnated with preservative salts to the heavy retentions required for marine exposure.

13.5.5.2.4 Increases in tabulated stresses resulting from various load duration factors are not cumulative and the load duration factor for the shortest duration load in a combination of loads shall apply for that load combination. The resulting structural members shall not be smaller than required for a longer duration of loading (refer to the 1991 Edition of the NDS® for additional commentary).

13.5.5.2.5 Modification of design stresses for load combinations, as specified in Section 3, are cumulative with load duration adjustments.

13.5.5.3 Adjustment for Preservative Treatment

Tabulated values apply to untreated wood and to wood that is preservatively treated in accordance with the requirements of AASHTO M 133. Unless otherwise noted, no adjustment of tabulated values is required for preservative treatment.

13.6 BENDING MEMBERS

13.6.1 General

13.6.1.1 The provisions of this article are applicable to straight members and to slightly curved bending members where the radius of curvature exceeds the span in inches divided by 800. Additional design requirements for

TABLE 13.5.5A Load Duration Factor, C_D

Load Duration	C_D
Permanent	0.90
2 months (vehicle live load)	1.15
7 days	1.25
1 day	1.33
5 minutes (railing only)	1.65

curved glued laminated timber members shall be as specified in the 1991 Edition of the NDS®.

13.6.1.2 For simple, continuous, and cantilevered bending members, the span shall be taken as the clear distance between supports plus one-half the required bearing length at each support.

13.6.1.3 Bending members shall be transversely braced to prevent lateral displacement and rotation and transmit lateral forces to the bearings. Transverse bracing shall be provided at the supports for all span lengths and at intermediate locations as required for lateral stability and load transfer (Article 13.6.4.4). The depth of transverse bracing shall not be less than ¾ the depth of the bending member.

13.6.1.4 Support attachments for bending members shall be of sufficient size and strength to transmit vertical, longitudinal and transverse loads from the superstructure to the substructure in accordance with the requirements of Section 3.

13.6.1.5 Glued laminated timber and structural composite lumber girders shall preferably be cambered a minimum 3 times the computed dead load deflection, but not less than ½ inch.

13.6.2 Notching

Notching of bending members can severely reduce member capacity and is not recommended. When notching is required for sawn lumber members, design limitations and requirements shall be in accordance with the NDS®, 1991 Edition. Design requirements and limitations for notching glued laminated timber members shall be as given in the "Timber Construction Manual," 1985 Edition by the American Institute of Timber Construction, published by John Wiley & Sons, New York, New York. Design requirements and limitations for notching structural composite lumber shall be as specified for glued laminated timber.

13.6.3 Modulus of Elasticity

The modulus of elasticity used for stiffness and stability computations shall be the tabulated modulus of elasticity adjusted by the applicable adjustment factor given in the following equation:

$$E' = EC_M \qquad (13\text{-}1)$$

where:

E' = allowable modulus of elasticity in psi;
E = tabulated modulus of elasticity in psi;
C_M = wet service factor from Article 13.5.5.1.

13.6.4 Bending

13.6.4.1 Allowable Stress

The allowable unit stress in bending shall be the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F_b' = F_bC_MC_DC_FC_VC_LC_fC_{fu}C_r \qquad (13\text{-}2)$$

where:

F_b' = allowable unit stress in bending in psi
F_b = tabulated unit stress in bending in psi
C_M = wet service factor from Article 13.5.5.1
C_D = load duration factor from Article 13.5.5.2
C_F = bending size factor for sawn lumber and structural composite lumber, and for glued laminated timber with loads applied parallel to the wide face of the laminations, from Article 13.6.4.2
C_v = volume factor for glued laminated timber with loads applied perpendicular to the wide face of the laminations, from Article 13.6.4.3
C_L = beam stability factor from Article 13.6.4.4.
C_f = form factor from Article 13.6.4.5
C_{fu} = flat use factor for sawn lumber from footnotes to Tables 13.5.1A and 13.5.1B
C_r = repetitive member factor for sawn lumber from footnotes to Table 13.5.1A.

The volume factor, C_v, shall not be applied simultaneously with the beam stability factor, C_L, and the lesser of the two factors shall apply in Equation 13-2.

13.6.4.2 Size Factor, C_F

13.6.4.2.1 The tabulated bending stress, for dimension lumber 2 inches to 4 inches thick shall be multiplied by the bending size factor, C_F, given in the footnotes to Table 13.5.1A.

13.6.4.2.2 For rectangular sawn lumber bending members 5 inches or thicker and greater than 12 inches in depth, and for glued laminated timber with loads applied parallel to the wide face of the laminations and greater than 12 inches in depth, the tabulated bending stress shall be multiplied by the size factor, C_F, determined from the following relationship:

$$C_F = \left(\frac{12}{d}\right)^{1/9} \qquad (13\text{-}3)$$

where d is the member depth in inches.

13.6.4.2.3 For structural composite lumber bending members of any width, the tabulated bending stress shall be reduced by the size factor, C_F, given by the following equation:

$$C_F = (21/L)^{1/m}(12/d)^{1/m} \qquad (13\text{-}4)$$

where:

L = length of bending member between points of zero moment in feet;
d = depth of bending member in inches;
m = parameter for the specific material determined in accordance with the requirements of ASTM D 5456.

13.6.4.3 Volume Factor, C_v

13.6.4.3.1 The tabulated bending stress for glued laminated timber bending members with loads applied perpendicular to the wide face of the laminations shall be adjusted by the volume factor, C_v, as determined by the following relationship:

$$C_V = (21/L)^{1/x}\,(12/d)^{1/x}\,(5.125/b)^{1/x} \le 1.0 \qquad (13\text{-}5)$$

where:

L = length of bending member between points of zero moment in feet;
d = depth of bending member in inches;
b = width of bending member in inches;
x = 20 for Southern Pine;
x = 10 for all other species.

13.6.4.3.2 When multiple piece width layups are used, the width of the bending member used in Equation 13-4 shall be the width of the widest piece used in the layup.

13.6.4.4 Beam Stability Factor, C_L

13.6.4.4.1 Tabulated bending values are applicable to members which are adequately braced. When members are not adequately braced, the tabulated bending stress shall be modified by the beam stability factor, C_L.

13.6.4.4.2 When the depth of a bending member does not exceed its width, or when lateral movement of the compression zone is prevented by continuous support and points of bearing have lateral support to prevent rotation, there is no danger of lateral buckling and $C_L = 1.0$. For other conditions, the beam stability factor shall be determined in accordance with the following provisions.

13.6.4.4.3 The bending member effective length, l_e, shall be determined from the following relationships for any loading condition:

$l_e = 2.06l_u$ when $l_u/d < 7$
$l_e = 1.63l_u + 3d$ when $7 \le l_u/d \le 14.3$
$l_e = 1.84l_u$ when $l_u/d > 14.3$

where:

l_e = effective length in inches;
l_u = unsupported length in inches;
d = depth of bending member in inches.

If lateral support is provided to prevent rotation at the points of bearing, but no other lateral support is provided throughout the bending member length, the unsupported length, l_u, is the distance between points of bearing, or the length of a cantilever.

If lateral support is provided to prevent rotation and lateral displacement at intermediate points as well as at the bearings, the unsupported length, l_u, is the distance between such points of intermediate lateral support.

13.6.4.4.4 The slenderness ratio for bending members, R_B, is determined from the following equation:

$$R_B = \sqrt{\frac{l_e d}{b^2}} \le 50 \qquad (13\text{-}6)$$

where:

R_B = bending member slenderness ratio;
d = depth of bending member in inches;
b = width of bending member in inches.

13.6.4.4.5 The beam stability factor, C_L, shall be computed as follows on the next page.

$$C_L = \frac{1+(F_{bE}/F_b^*)}{1.90} - \sqrt{\frac{(1+F_{bE}/F_b^*)^2}{3.61} - \frac{F_{bE}/F_b^*}{0.95}} \quad (13\text{-}7)$$

$$F_{bE} = \frac{K_{bE}E'}{R_B^2} \quad (13\text{-}8)$$

where:

F_b^* = tabulated bending stress adjusted by all applicable adjustment factors given in Equation 13-2 except the volume factor, C_v, the beam stability factor, C_L, and the flat-use factor, C_{fu};

K_{bE} = 0.438 for visually graded sawn lumber 0.609 for glued laminated timber, structural composite lumber, and machine stress rated lumber;

E' = allowable modulus of elasticity in psi as determined by Article 13.6.3.

13.6.4.5 Form Factor, C_f

For bending members with circular cross sections the tabulated bending stress shall be adjusted by the form factor, C_f = 1.18. A tapered circular section shall be considered as a bending member of variable cross section.

13.6.5 Shear Parallel to Grain

13.6.5.1 General

13.6.5.1.1 The provisions of this article apply to shear parallel to grain (horizontal shear) at or near the points of vertical support of solid bending members. Refer to the 1991 edition of the NDS® for additional design requirements for other member types.

13.6.5.1.2 The critical shear in wood bending members is shear parallel to grain. It is unnecessary to verify the strength of bending members in shear perpendicular to grain.

13.6.5.2 Actual Stress

The actual unit stress in shear parallel to grain due to applied loading on rectangular members shall be determined by the following equation:

$$f_v = \frac{3V}{2bd} \quad (13\text{-}9)$$

where:

f_v = actual unit stress in shear parallel to grain in psi;
b = width of bending member in inches;
d = depth of bending member in inches;
V = vertical shear in pounds, as determined in accordance with the following provisions.

For uniformly distributed loads, such as dead load, the magnitude of vertical shear used in Equation 13-9 shall be the maximum shear occurring at a distance from the support equal to the bending member depth, d. When members are supported by full bearing on one surface, with loads applied to the opposite surface, all loads within a distance from the supports equal to the bending member depth shall be neglected.

For vehicle live loads, the loads shall be placed to produce the maximum vertical shear at a distance from the support equal to three times the bending member depth, 3d, or at the span quarter point, L/4, whichever is the lesser distance from the support. The distributed live load shear used in Equation 13-9 shall be determined by the following expression:

$$V_{LL} = 0.50\,[(0.60\,V_{LU}) + V_{LD}] \quad (13\text{-}10)$$

where:

V_{LL} = distributed live load vertical shear in pounds;
V_{LU} = maximum vertical shear, in pounds, at 3d or L/4 due to undistributed wheel loads;
V_{LD} = maximum vertical shear, in pounds, at 3d or L/4 due to wheel loads distributed laterally as specified for moment in Article 3.23.

For undistributed wheel loads, one line of wheels is assumed to be carried by one bending member.

13.6.5.3 Allowable Stress

The allowable unit stress in shear parallel to grain shall be the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F_v' = F_v C_M C_D \quad (13\text{-}11)$$

where:

F_v' = allowable unit stress in shear parallel to grain in psi;
F_v = tabulated unit stress in shear parallel to grain in psi;
C_M = wet service factor from Article 13.5.5.1;
C_D = load duration factor from Article 13.5.5.2.

For sawn lumber beams, further adjustment by the shear stress factor may be applicable as described in the footnotes to Table 13.5.1A.

For structural composite lumber, more restrictive adjustments to the tabulated shear stress parallel to grain shall be as recommended by the material manufacturer.

13.6.6 Compression Perpendicular to Grain

13.6.6.1 General

When calculating the bearing stress in compression perpendicular to grain at beam ends, a uniform stress distribution shall be assumed.

13.6.6.2 Allowable Stress

The allowable unit stress in compression perpen-337dicular to grain shall be the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F'_{c\perp} = F_{c\perp} C_M C_b \qquad (13\text{-}12)$$

where:

$F'_{c\perp}$ = allowable unit stress in compression perpendicular to grain, in psi;
$F_{c\perp}$ = tabulated unit stress in compression perpendicular to grain, in psi;
C_M = wet service factor from Article 13.5.5.1;
C_b = bearing area factor from Article 13.6.6.3.

13.6.6.3 Bearing Area Factor, C_b

Tabulated values in compression perpendicular to grain apply to bearings of any length at beam ends, and to all bearings 6 inches or more in length at any other location. For bearings less than 6 inches in length and not nearer than 3 inches to the end of a member, the tabulated value shall be adjusted by the bearing area factor, C_b, given by the following equation:

$$C_b = \frac{l_b + 0.375}{l_b} \qquad (13\text{-}13)$$

where l_b is the length of bearing in inches, measured parallel to the wood grain. For round washers, or other round bearing areas, the length of bearing shall be the diameter of the bearing area.

The multiplying factors for bearing lengths on small areas such as plates and washers are given in Table 13.6.1A.

TABLE 13.6.1A Values of the Bearing Area Factor, C_b, for Small Bearing Areas

Length of Bearing, l_b (in.)	1/2	1	1-1/2	2	3	4	6 or more
Bearing Area Factor, C_b	1.75	1.38	1.25	1.19	1.13	1.10	1.00

13.6.7 Bearing on Inclined Surfaces

For bearing on an inclined surface, the allowable unit stress in bearing shall be as given by the following equation:

$$F'_\theta = \frac{F'_g F'_{c\perp}}{F'_g \sin^2\theta + F'_{c\perp} \cos^2\theta} \qquad (13\text{-}14)$$

where:

F'_θ = allowable unit stress for bearing on an inclined surface, in psi;
F'_g = allowable unit stress in bearing parallel to grain from Article 13.7.4;
$F_{c\perp}$ = allowable unit stress in compression perpendicular to the grain from Article 13.6.6;
θ = angle in degrees between the direction of load and the direction of grain.

13.7 COMPRESSION MEMBERS

13.7.1 General

13.7.1.1 The provisions of this article apply to simple solid columns consisting of a single piece of sawn lumber, piling, structural composite lumber, or glued laminated timber. Refer to the 1991 Edition of the NDS® for design requirements for built-up columns, consisting of a number of solid members joined together with mechanical fasteners, and for spaced columns consisting of two or more individual members with their longitudinal axes parallel, separated and fastened at the ends and at one or more interior points by blocking.

13.7.1.2 The term "column" refers to all types of compression members, including members forming part of a truss or other structural components.

13.7.1.3 Column bracing shall be provided where necessary to provide lateral stability and resist wind or other lateral forces.

13.7.2 Eccentric Loading or Combined Stresses

Members with eccentric loading or combined stresses shall be designed in accordance with the provisions of the NDS®, 1991 Edition.

13.7.3 Compression

13.7.3.1 Net Section

The actual unit stress in compression parallel to grain, f_c, shall be based on the net section as described in Article 13.1, except that it may be based on the gross section when the reduced section does not occur in the critical part of the column length that is most subject to potential buckling.

13.7.3.2 Allowable Stress

The allowable unit stress in compression parallel to grain shall not exceed the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F_c' = F_c C_M C_D C_F C_P \qquad (13\text{-}15)$$

where:

F_c' = allowable unit stress in compression parallel to grain in psi;
F_c = tabulated unit stress in compression parallel to grain in psi;
C_M = wet service factor from Article 13.5.5.1;
C_D = load duration factor from Article 13.5.5.2;
C_F = compression size factor for sawn lumber from footnotes to Table 13.5.1A;
C_P = column stability factor from Article 13.7.3.3.

13.7.3.3 Column Stability Factor, C_P

13.7.3.3.1 Tabulated values in compression parallel to grain are applicable to members which are adequately braced. When members are not adequately braced, the tabulated stress shall be modified by the column stability factor, C_P.

13.7.3.3.2 When a compression member is supported throughout its length to prevent lateral displacement in all directions, $C_P = 1.0$. For other conditions, the column stability factor shall be determined in accordance with the following provisions.

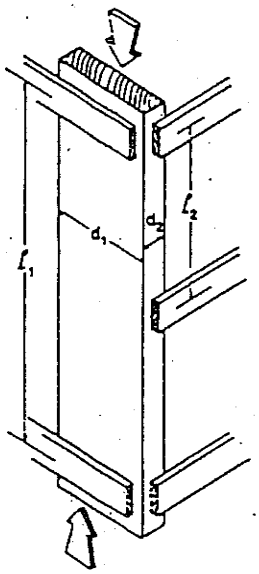
13.7.3.3.3 The effective column length, l_e, shall be determined in accordance with good engineering practice. Actual column length, l, may be multiplied by an effective length factor to determine the effective column length:

$$l_e = Kl \qquad (13\text{-}16)$$

where:

l_e = effective column length in inches
K = effective length factor from Table C-1 of Appendix C
l = actual column length between points of lateral support in inches.

13.7.3.3.4 For columns of rectangular cross section, the column slenderness ratio, l_e/d, shall be taken as the larger of the ratios, l_{e1}/d_1 or l_{e2}/d_2. (See Figure 13.7.1A.) The slenderness ratio shall not exceed 50.



l_1 and l_2 = distances between points of lateral support in planes 1 and 2, inches.
d_1 and d_2 = cross-sectional dimensions of rectangular compression member in planes of lateral support, inches.

FIGURE 13.7.1A

13.7.3.3.5 The column stability factor, C_P, shall be as given by the following expressions:

$$C_p = \frac{1+F_{cE}/F_c^*}{2c} - \sqrt{\frac{\left(1+F_{cE}/F_c^*\right)^2}{(2c)^2} - \frac{F_{cE}/F_c^*}{c}} \qquad (13\text{-}17)$$

$$F_{cE} = \frac{K_{cE}E'}{(l_e/d)^2} \qquad (13\text{-}18)$$

TABLE 13.7.1A Support Condition Coefficients for Tapered Columns

Support Condition	Support Condition Coefficient, a
Large end fixed, small end unsupported	0.70
Small end fixed, large end unsupported	0.30
Both ends simply supported	
Tapered toward one end	0.50
Tapered towards both ends	0.70

where:

F_c^* = tabulated stress in compression parallel to grain adjusted by all applicable modification factors given in Equation 13-14 except C_P;
K_{cE} = 0.300 for visually graded sawn lumber; 0.418 for glued laminated timber, structural composite lumber, and machine stress-rated lumber;
c = 0.80 for sawn lumber;
0.85 for round piles;
0.90 for glued laminated timber and structural composite lumber.

For especially severe service conditions or extraordinary hazardous conditions, the use of lower design values than those obtained above may be necessary. Refer to the 1991 Edition of the NDS®.

13.7.3.4 Tapered Columns

13.7.3.4.1 For rectangular columns tapered at one or both ends, the cross-sectional area shall be based on the representative dimension of each tapered face. The representative dimension, d_{rep}, of each tapered face shall be based on the support condition coefficient given in Table 13.7.1A.

13.7.3.4.2 For support conditions given in Table 13.7.1A, the representative dimension, d_{rep}, of each tapered face shall be as given by the following equation:

$$d_{rep} = d_{min} + (d_{max} - d_{min})\left[a - 0.15\left(1 - \frac{d_{min}}{d_{max}}\right)\right] \qquad (13\text{-}19)$$

where:

d_{rep} = representative dimension for a tapered column face, in inches;
d_{min} = minimum column face dimension, in inches;
d_{max} = maximum column face dimension, in inches;
a = coefficient based on support conditions.

13.7.3.4.3 For support conditions other than those in Table 13.7.1A, the representative dimension of each tapered face shall be as given by the following equation:

$$d_{rep} = d_{min} + 0.33(d_{max} - d_{min}) \qquad (13\text{-}20)$$

13.7.3.4.4 For any tapered column, the actual stress in compression parallel to grain, f_c, shall not exceed the allowable stress determined by Equation 13-14, assuming the column stability factor $C_P = 1.0$.

13.7.3.5 Round Columns

The design of a round column shall be based on the design of a square column of the same cross-sectional area with the same degree of taper.

13.7.4 Bearing Parallel to Grain

13.7.4.1 The actual stress in bearing parallel to grain shall be based on the net area and shall not exceed the tabulated stress for bearing parallel to grain adjusted by the applicable adjustment factor given in the following equation:

$$F_g' = F_g C_D \qquad (13\text{-}21)$$

where:

F_g' = allowable unit stress in bearing parallel to grain in psi;
F_g = tabulated unit stress in bearing parallel to grain from Table 13.5.2A, in psi;
C_D = load duration factor from Article 13.5.5.2.

13.7.4.2 When the bearing load is at an angle to the grain, the allowable bearing stress shall be determined by Equation 13-14, using the design values for end-grain bearing parallel to grain and design values in compression perpendicular to grain.

13.7.4.3 When bearing parallel to grain exceeds 75% of the allowable value determined by Equation 13-21, bearing shall be on a metal plate or on other durable, rigid, homogeneous material of adequate strength and stiffness to distribute applied loads over the entire bearing area.

13.8 TENSION MEMBERS

13.8.1 Tension Parallel to Grain

The allowable unit stress in tension parallel to grain hall be the tabulated value adjusted by the applicable adjustment factors given in the following equation:

$$F_t' = F_t C_M C_D C_F \qquad (13\text{-}22)$$

where:

F'_t = allowable unit stress in tension parallel to grain in psi;
F_t = tabulated unit stress in tension parallel to grain in psi;
C_M = wet service factor from Article 13.5.5.1;
C_D = load duration factor from Article 13.5.5.2;
C_F = tension size factor for sawn lumber from footnotes to Table 13.5.1A and for structural composite lumber from footnotes to Tables 13.5.4A and 13.5.4B.

13.8.2 Tension Perpendicular to Grain

Designs which induce tension perpendicular to the grain of wood members should not be used. When tension perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist all such forces should be used. Refer to the 1991 Edition of the NDS® for additional information.

13.9 MECHANICAL CONNECTIONS

13.9.1 General

13.9.1.1 Except as otherwise required by this specification, mechanical connections and their installation shall conform to the requirements of the NDS®, 1991 Edition.

13.9.1.2 Components at mechanical connections, including the wood members, connecting elements, and fasteners, shall be proportioned so that the design strength equals or exceeds the required strength for the loads acting on the structure. The strength of the connected wood components shall be evaluated considering the net section, eccentricity, shear, tension perpendicular to grain and other factors that may reduce component strength.

13.9.2 Corrosion Protection

13.9.2.1 Except as permitted by this section, all steel hardware for wood structures shall be galvanized in accordance with AASHTO M 232 or cadmium plated in accordance with AASHTO M 299.

13.9.2.2 All steel components, timber connectors, and castings, other than malleable iron, shall be galvanized in accordance with AASHTO M 111.

13.9.2.3 Alternative corrosion protection coatings, such as epoxies, may be used when the demonstrated performance of the coating is sufficient to provide adequate protection for the intended exposure conduction.

13.9.2.4 Heat-treated alloy components and fastenings shall be protected by an approved alternative protective treatment that does not adversely affect the mechanical properties of the material.

13.9.3 Fasteners

13.9.3.1 Fastener design values shall be adjusted by the applicable adjustment factors for the intended use condition.

13.9.3.2 When determining fastener design values, wood shall be assumed to be used under wet-use or exposed to weather conditions.

13.9.3.3 Glulam rivets shall not be used in permanent structures.

13.9.4 Washers

13.9.4.1 Washers shall be provided under bolt and lag screw heads and under nuts that are in contact with wood. Washers may be omitted under heads of special timber bolts or dome-head bolts when the size and strength of the head is sufficient to develop connection strength without excessive wood crushing.

13.9.4.2 Washers shall be of sufficient size and strength to prevent excessive wood crushing when the fastener is tightened. For bolts or rods loaded in tension, washers shall be of sufficient size and strength to develop the tensile strength of the connection without excessive bending or exceeding wood strength in compression perpendicular to grain.

from: 2002 AASHTO STD SPECS

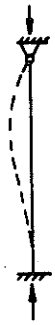
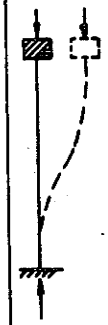
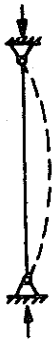
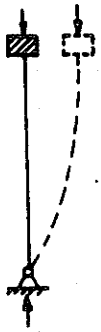
APPENDIX C *(Continued)*

EFFECTIVE LENGTH FACTOR, K

The Effective Length of a column, KL, has been used in the equations for allowable compression stress in the column. K is the ratio of the effective length of an idealized pin-end column to the actual length of a column with various other end conditions. KL represents the length between inflection points of a buckled column. Restraint against rotation and translation of column ends influences the position of the inflection points in a column. Theoretical values of K for some idealized column end conditions are given in Table C-1. Since column end conditions seldom comply fully with idealized restraint against rotation and translation, the recommended values suggested by the Column Research Council are higher than the idealized values.

Columns in continuous frames unbraced by adequate attachment to shear walls, diagonal bracing, or adjacent structures depend on the bending stiffness of the rigidly connected beams for lateral stability. The effective length factor, K, is dependent on the amount of bending stiffness supplied by the beams at the column ends. If the amount of stiffness supplied by the beams is small, the value of K could exceed 2.0.

TABLE C-1

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

[1]For riveted and bolted truss members (partially restrained), use K = 0.75. For pinned connections in truss members, use K = 0.875 (pin friction).

If it is assumed that elastic action occurs and that all columns buckle simultaneously in a frame, it can be rationally shown that*

$$\frac{G_a G_b(\pi/K)^2 - 36}{6(G_a + G_b)} = \frac{\pi/K}{\tan(\pi/K)} \qquad \text{(C-1)}$$

where subscripts a and b refer to the two ends of the column.

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} \qquad \text{(C-2)}$$

Σ = summation of all members rigidly connected to an end of the column in the plane of bending;
I_c = moment of inertia of column;
L_c = unbraced length of column;
I_g = moment of inertia of beam or other restraining member;
L_g = unsupported length of beam or other restraining member;
K = effective length factor.

*See "Steel Structures Design and Behavior" by Charles G. Salmon and John E. Johnson, published by International Text Book Company, 1971.

Table C-2 is a graphical representation of the relationship between K, G_a, and G_b, and can be used to obtain the value of K easily. In frames which have columns that fall in the inelastic buckling range, (i.e., $KL/r < C_c = (2\pi^2 E/F_y$ 1/2), K may often be reduced. The procedure for reducing K can be found in "Effective Length of Columns in Unbraced Frames" by Joseph A. Yura, *AISC Engineering Journal*, published by the American Institute of Steel Construction, 101 Park Avenue, New York, New York 10017.

From: Forest Service, Timber Bridge, 1992

vehicle. It is generally most convenient to start with a selected species and grade of lumber, size the deck thickness based on deflection, and then check bending and shear. Because of the susceptibility of the deck to loosening or delamination, a maximum live load deflection of $L/500$ is recommended. Effective deck section properties and typical dead loads are given in Tables 8-3 and 8-4, respectively.

Table 8-3.—Effective deck section properties for continuous longitudinal nail-laminated decks and longitudinal nail-laminated deck panels with adequate shear transfer between panels.

	H 15-44 and HS 15-44 12,000-pound wheel load				H 20-44 and HS 20-44 16,000-pound wheel load			
t (in.)	D_w (in.)	A (in²)	S (in³)	I (in⁴)	D_w (in.)	A (in²)	S (in³)	I (in⁴)
5-1/2	28.32	155.76	142.78	392.65	31.00	170.50	156.29	429.80
6	29.32	175.92	175.92	527.76	32.00	192.00	192.00	576.00
7-1/4	31.82	230.70	278.76	1,010.49	34.50	250.13	302.23	1,095.60
8	33.32	266.56	355.41	1,421.65	36.00	288.00	384.00	1,536.00
9-1/4	35.82	331.34	510.81	2,362.49	38.50	356.13	549.03	2,539.25
10	37.32	373.20	622.00	3,110.00	40.00	400.00	666.67	3,333.33
11-1/4	39.82	447.98	839.95	4,724.74	42.50	478.13	896.48	5,042.72
12	41.32	495.84	991.68	5,950.08	44.00	528.00	1,056.00	6,336.00
13-1/4	43.82	580.62	1,282.19	8,494.52	46.50	616.13	1,360.61	9,014.04
14	45.32	634.48	1,480.45	10,363.17	48.00	672.00	1,568.00	10,976.00
15-1/4	47.82	729.26	1,853.52	14,133.11	50.50	770.13	1,957.40	14,925.
16	49.32	789.12	2,104.32	16,834.56	52.00	832.00	2,218.67	17,749.33

Table 8-4.—Deck dead load for the wheel distribution width (D_w) in lb/ft of deck span for longitudinal continuous nail-laminated decks and longitudinal nail-laminated deck panels with adequate shear transfer between panels.

	H 15-44 and HS 15-44 12,000-pound wheel load			H 20-44 and HS 20-44 16,000-pound wheel load		
t (in.)	D_w (in.)	Deck only (lb/ft)	Deck plus 3 in. of asphalt (lb/ft)	D_w (in.)	Deck only (lb/ft)	Deck plus 3 in. of asphalt (lb/ft)
5-1/2	28.32	54.08	142.58	31.00	59.20	156.08
6	29.32	61.08	152.71	32.00	66.66	166.67
7-1/4	31.82	80.10	179.54	34.50	86.84	194.65
8	33.32	92.55	196.67	36.00	99.99	212.50
9-1/4	35.82	115.04	226.97	38.50	123.65	243.97
10	37.32	129.58	246.21	40.00	138.88	263.88
11-1/4	39.82	155.54	279.97	42.50	166.01	298.83
12	41.32	172.16	301.28	44.00	183.32	320.82
13-1/4	43.82	201.59	338.53	46.50	213.92	359.23
14	45.32	220.29	361.95	48.00	233.32	383.32
15-1/4	47.82	253.20	402.66	50.50	267.39	425.20
16	49.32	273.98	428.11	52.00	288.87	451.40

From: Duluth Tech College & MnDOT 1994

HS 15 Loading:

Use 3/4 of the values obtained from the formulas for HS 20 loading

Moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading.

3.25.2 E. Plank and Nail Laminated Longitudinal Flooring

3.25.2.1 In the direction of the span, the wheel load shall be considered a point loading.

3.25.2.2 Normal to the direction of the span the wheel load shall be distributed as follows:

Plank floor: width of plank
Non-interconnected nail laminated floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected laminated floor, with adequate shear transfer between panels, not less than 6 inches thick: width of tire plus twice thickness of floor.

3.25.2.3 For longitudinal flooring the span shall be taken as the clear distance between floor beams plus one-half the width of one beam but shall not exceed the clear span plus the floor thickness.

3.25.3 F. Longitudinal Glued Laminated Timber Decks

3.25.3.1 **Bending Moment**

In calculating bending moments in glued laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

Longitudinal nail-laminated decks are constructed from lumber laminations that are 2 to 4 inches thick, and 5 inches or wider, in the Joist and Plank size classification.[11, 12] Both continuous and panelized configurations can be constructed from any lumber size provided it is a minimum of 6 inches in nominal depth (AASHTO 3.25.2.2). From a practical standpoint, however, continuous decks are normally constructed of 2-inch nominal material that is 6 to 12 inches wide to facilitate field nailing and handling. Panelized systems commonly use 4-inch nominal material that is 10 to 16 inches wide, which is more economical and practical for shop fabrication.

CONTINUOUS NAIL-LAMINATED LUMBER BRIDGES

Continuous nail-laminated lumber bridges are practical for simple spans up to approximately 19 feet for HS 20-44 and H 20-44 loads and 21 feet for HS 15-44 and H 15-44 loads. Load distribution and continuity across the bridge are provided by the nails that are placed through two and one-half laminations, in the same pattern used for transverse nail-laminated decks (Figure 8-10). Transverse stiffener beams are not required. The performance of longitudinal nail-laminated bridges is similar in many respects to transverse nail-laminated decks and depends primarily on the effectiveness of the nails in transferring loads between adjacent laminations. Field experience has shown that many nail-laminated decks demonstrate a tendency to loosen or delaminate from cyclic loading and moisture content changes in the laminations. This subsequently leads to reduced load distribution and deterioration of asphalt wearing surfaces. In longitudinal deck bridges, the potential for delamination is normally higher than for transverse configurations because the deck spans and associated deflections are generally larger. Performance can be improved by limiting live load deflections and using edge-grain lumber for laminations, but these measures may not be totally effective in eliminating deck loosening.

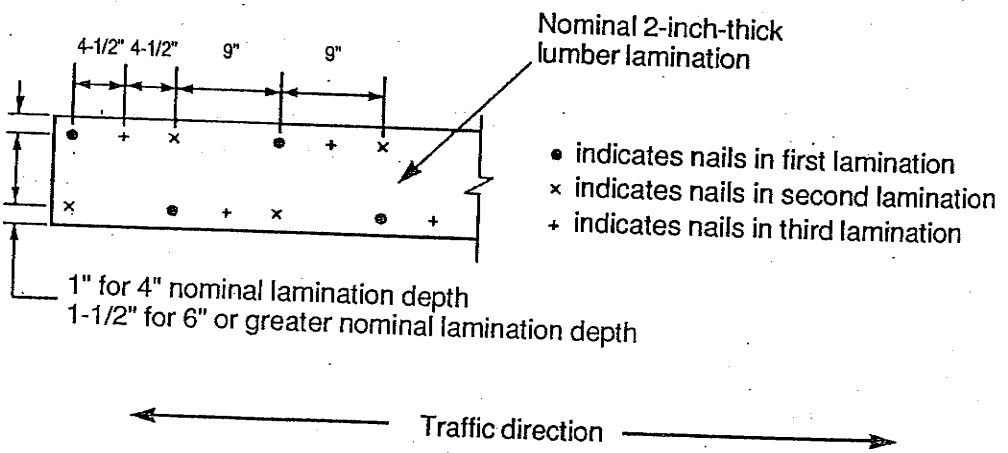


Figure 8-10.—Nailing pattern for continuous longitudinal nail-laminated lumber decks constructed of nominal 2-inch-thick sawn lumber.

From: US Forest Service, Timber Bridges Design, Construction, Inspection & Maintenance, 1992

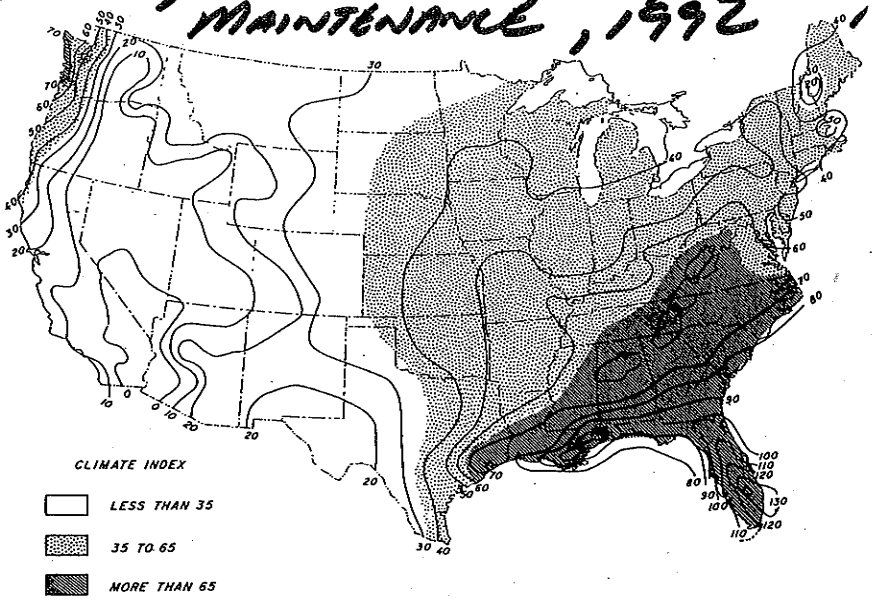


Figure 13-35.—Climate index map for decay hazard. The higher numbers indicate a higher decay hazard.

Areas Susceptible to Decay

Wood decay can occur only when proper conditions prevail for fungal growth. Although timber bridges differ in many respects, there are several common areas where decay is most likely to occur. These areas involve situations where the wood moisture is high and where breaks in the preservative envelope (or insufficient preservative penetration) provide an entry point for decay organisms. Signs of high moisture content and sites around fasteners, checks, or mechanical damage should be considered areas of high decay potential (Figure 13-36).

The moisture content of bridge components is not uniform, and substantial variations occur within and between members. End-grain surfaces absorb water much quicker than do side-grain surfaces (Figure 13-37). With other conditions equal, permeability in the longitudinal direction (parallel to grain) is 50 to 100 times greater than in the transverse direction (perpendicular to grain). Decay development is most affected by the moisture content of the wood in the immediate vicinity of the infection. Therefore, a member may remain generally dry and uninfected along most of its length but be severely decayed in localized areas where untreated wood is exposed and water is continuously or intermittently trapped. Bridge moisture conditions are also subject to seasonal variations and may be altered by maintenance operations or changes in drainage patterns. Wood that appears thoroughly dry may have been exposed to high moisture contents in the past and could be seriously decayed. The inspector must be alert for any visual or intuitive indications of wetting. Visual signs may appear as

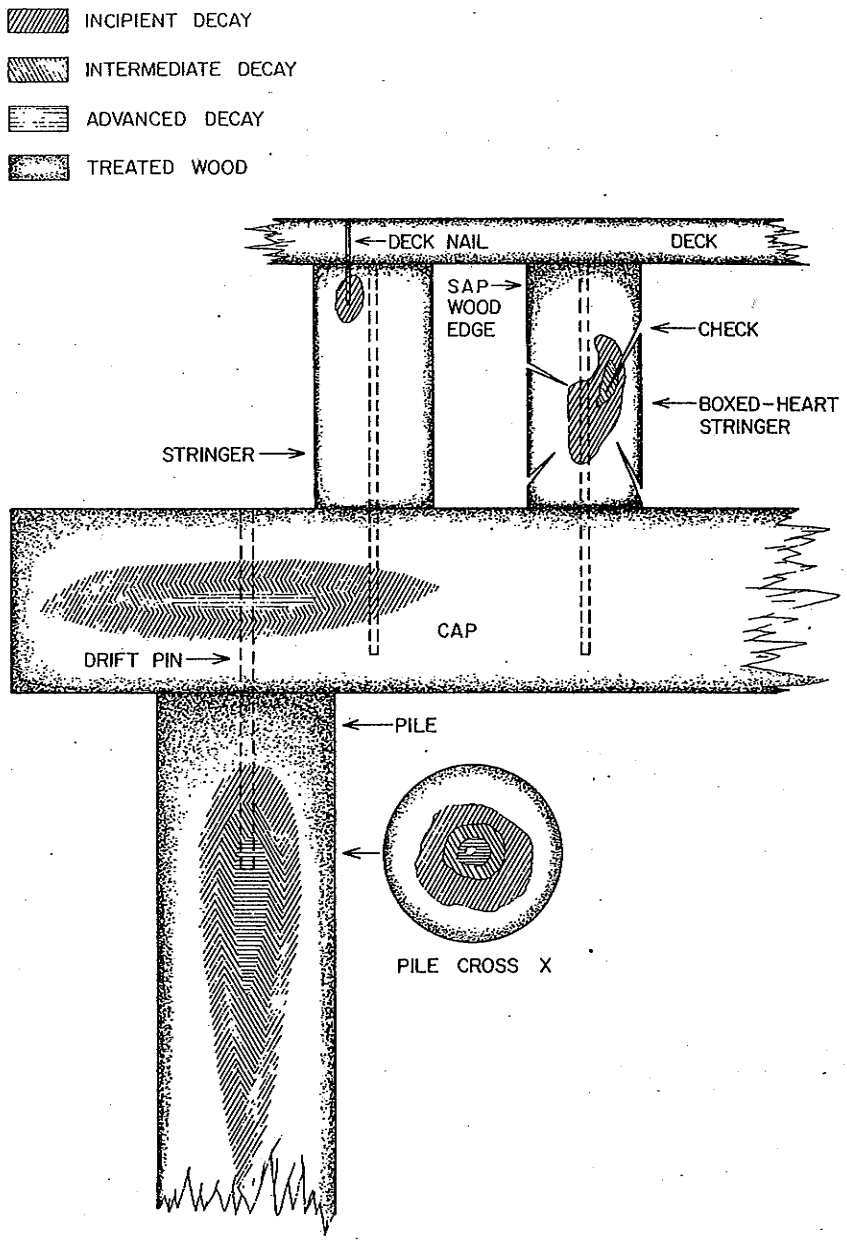


Figure 13-36.—Diagram depicting potential decay locations in a timber bridge.

watermarks, staining, or light mud stains. Intuitive signs include any horizontal surfaces, contact areas, depressions, or other features that may trap water and therefore indicate potentially high moisture exposures.

As discussed, the potential for bridge decay is highest where untreated wood is exposed. This condition occurs most often in the vicinity of seasoning checks, fasteners, and areas of mechanical damage. Conditions for deterioration are enhanced at these locations because moisture enters cracks or other crevices where air circulation and drying are inhibited. Seasoning checks commonly develop in large lumber members, and, t

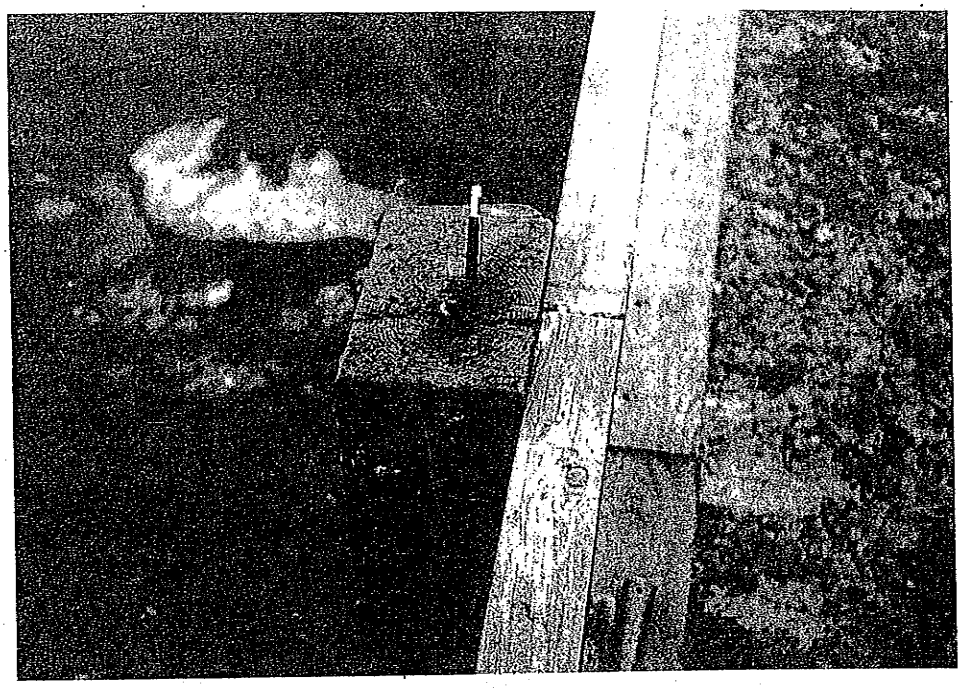


Figure 13-37.—Decay in the end grain of a timber rail post (photo courtesy of Duane Yager, USDA Forest Service).

lesser extent, in glulam. Although the size of the check influences the area of exposed untreated material, very small openings are still sufficient to allow entry of decay organisms (Figure 13-38). Holes for bolts, nails, or other hardware can trap water, which will be absorbed deep into the wood end grain by capillary action. Decay susceptibility at connections is higher because fasteners may be placed in field-bored holes that are not adequately treated with preservatives (Figure 13-39). Mechanical damage from improper handling, overloads, vehicle abrasion, and support settlements also breaks the preservative barrier and provides an entry point for decay organisms. In addition to increasing the decay hazard, mechanical damage may also affect structural capacity, depending on the decay's nature, location, and extent (Figure 13-40).

Component Inspection

Component inspection involves the systematic examination of individual bridge members. When deterioration is found, its location and extent must be defined and noted so that the load-carrying capacity of the structure can be determined by engineering analysis. At some locations, deterioration may have no significant effect on member strength. In other locations, any deterioration will reduce capacity. In both cases, the inspector must accurately locate, define, and record all deterioration, notwithstanding its perceived effects on structural capacity.

Because of the large number of structural components and the variety of locations where conditions for decay development exist in a bridge, the degree of accuracy for assessing the extent of deterioration depends on the judgment of the inspector. Regardless of bridge size, no inspection can

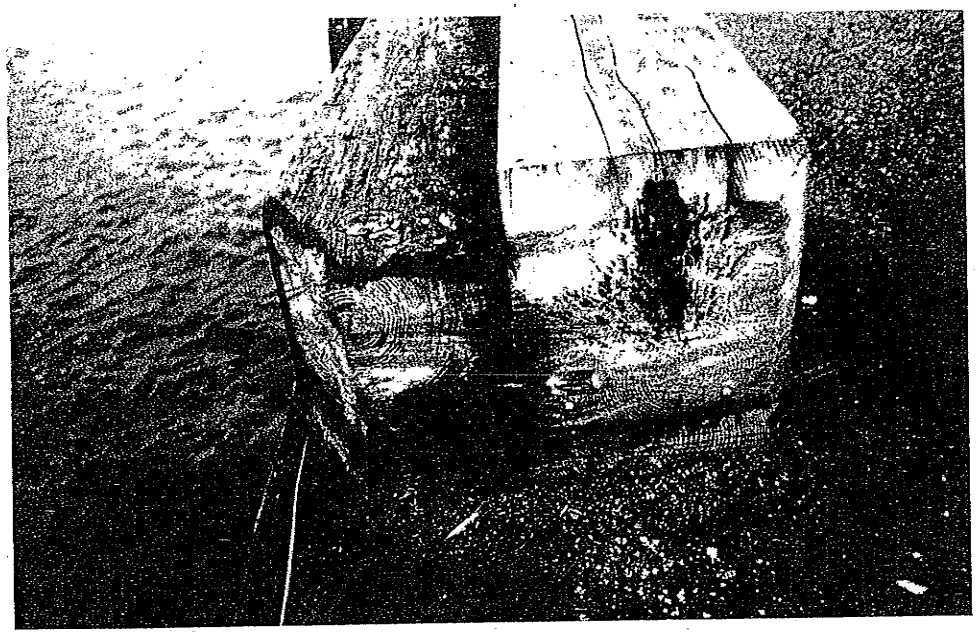


Figure 13-38.—Cross section of a timber curb, exposed by sawing, reveals interior decay resulting from seasoning checks in the upper surface.

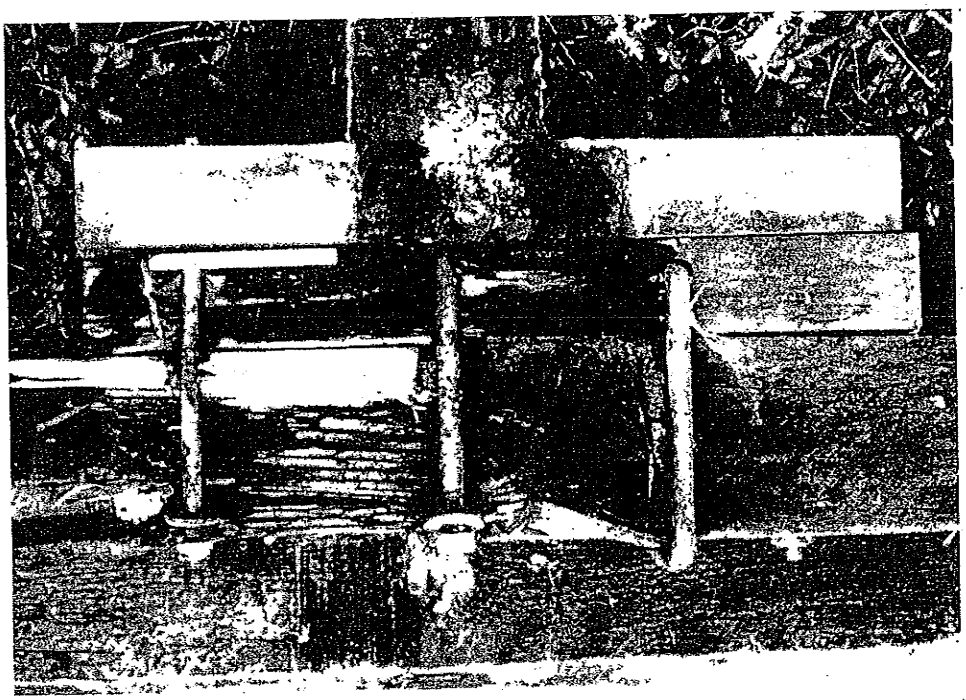


Figure 13-39.—Decay in timber members around field-bored fastener holes.

Figure 13-40.—Large crack in a sawn lumber bridge beam caused by vehicle overloads (photo courtesy of Duane Yager, USDA Forest Service).

reasonably or economically examine every bridge component. Rather, the inspector must base the degree of inspection on information from the preinspection evaluation and knowledge of bridge deterioration and its causes, signs, and probable locations. For example, it may not be practical to examine the area around each fastener when deck members are attached with penetrating fasteners in each beam. Instead, the inspector should select the most probable areas of deterioration for evaluation. If deterioration is found, its extent is determined and additional inspections are made at other locations. If no deterioration is found in high-hazard zones, it is unlikely that other areas are affected.

One of the most important aspects of component inspection is the sequence and coordination of inspection efforts. To ensure that all critical areas are covered, a systematic, well-defined plan must be developed. When more than one inspector is involved, the responsibilities of each must be clearly defined to avoid either missing areas or excessive duplication. The preferred inspection sequence generally follows the sequence of construction. After initially surveying the structure, the inspector begins with the lower substructure members and progresses upward to the top of the superstructure. Following this sequence, the inspector can observe the behavior of members under load before their actual inspection.

Initial Survey

The best way to begin a bridge inspection is to take a brief walk across and around the structure, observing general features and looking for

obvious signs of deterioration or distress. Particular attention should be given to changes in the longitudinal or transverse deck elevation that may indicate foundation movement, deck swelling, or other adverse conditions. The rail and curb elements should also be checked for position and alignment. Slanted posts or separated rails may indicate deck swelling or superstructure movement. This is also a good time to observe drainage patterns on approach roadways and obstructions to deck drains, as well as the effectiveness of the deck and wearing surface in protecting underlying components. General observations of this type can alert the inspector to potentially adverse situations requiring more detailed examination later in the inspection. This inspection also can provide an opportunity to prepare initial sketches of the structure and to define the directions and other features used in recording inspection findings.

Substructure Inspection

The substructure is the portion of a bridge that is probably most susceptible to deterioration. Soil-contacting members such as posts, piling, abutments, and wing walls are exposed in varying degrees to nearly constant wetting, resulting in wood moisture contents suitable for decay. Surrounding soil frequently contains large numbers of fungal spores and woody plant material in which decay fungi can live and spread to infect bridge members. Substructure decay potential is also greater because of the high incidence of field fabrication (cutting and drilling) and the large number of penetrating fasteners.

Initial inspection of the substructure should begin with a visual examination of abutments for signs of deterioration, mechanical damage, and settlement. The most probable locations for decay are in the vicinity of the ground line, at connections between the cap and column, and at framing connections for bracing, tie rods, and backwall or wingwall planks. Starting at the base of the abutment, soil should be removed around a representative number of members in order to inspect for indications of decay or insect attack. When soil is very wet or covered by water, decay is generally limited to areas close to ground level because the lack of oxygen below the surface limits the growth of most fungi. As soil moisture content decreases, conditions below ground become more favorable, and decay may occur at depths of 2 feet or more in moderately dry soils. Surface decay and insect damage can be revealed by visual observation and probing. When evidence of decay is found, its extent is further defined by drilling or coring (Figure 13-41). Detecting internal decay is generally accomplished by using a combination of sounding and drilling or coring. Because sounding will reveal only serious internal defects, it should never be the only method used.

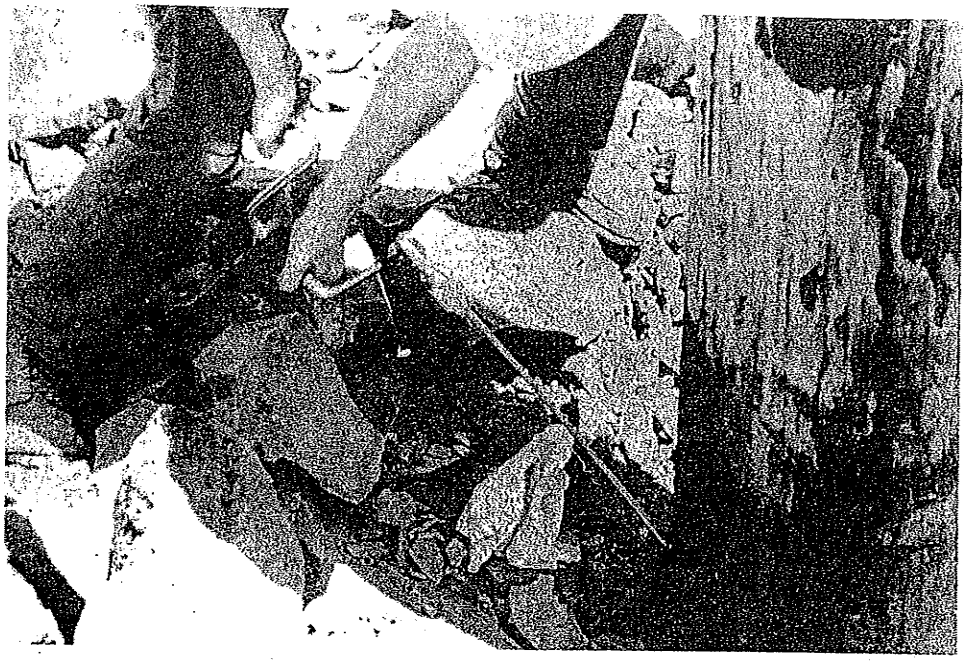


Figure 13-41.—Hand drilling at the base of a timber pile.

From below the ground line, inspection should proceed upward, with particular attention given to connections, seasoning checks, and mechanical damage. Timber backwalls, wingwalls, and incidental bracing should also be examined for breakage or bulging from earth pressure. Exposed end grain on pile or post tops should also be inspected for decay. Many tops are intentionally cut at an angle in the belief that water will run off. Instead, angled cuts expose more untreated end grain, increasing the decay potential. When tops are provided with protective sheet-metal caps, the condition of the cap should be checked for holes or tears in the surface. Damaged caps allow water to enter through the break and penetrate end grain, creating ideal conditions for internal decay (Figure 13-42).

Above the supporting piles or posts, the cap supporting the superstructure provides a horizontal surface that traps debris and water runoff from the deck. Connections into the cap and horizontal checks that trap water and debris are critical zones. The connection between the cap and column is especially important because many connections are made with drift pins or bolts that extend deep into the column end grain. Water from the cap flows into these connections and can result in substantial internal decay with little evidence of exterior damage (Figure 13-43). The inspector should also check for crushed zones at bearing points along the cap that trap water and damage the treated wood shell. Crushing can also indicate overloads or load redistribution from settlement and should be further investigated in other components of the structure.

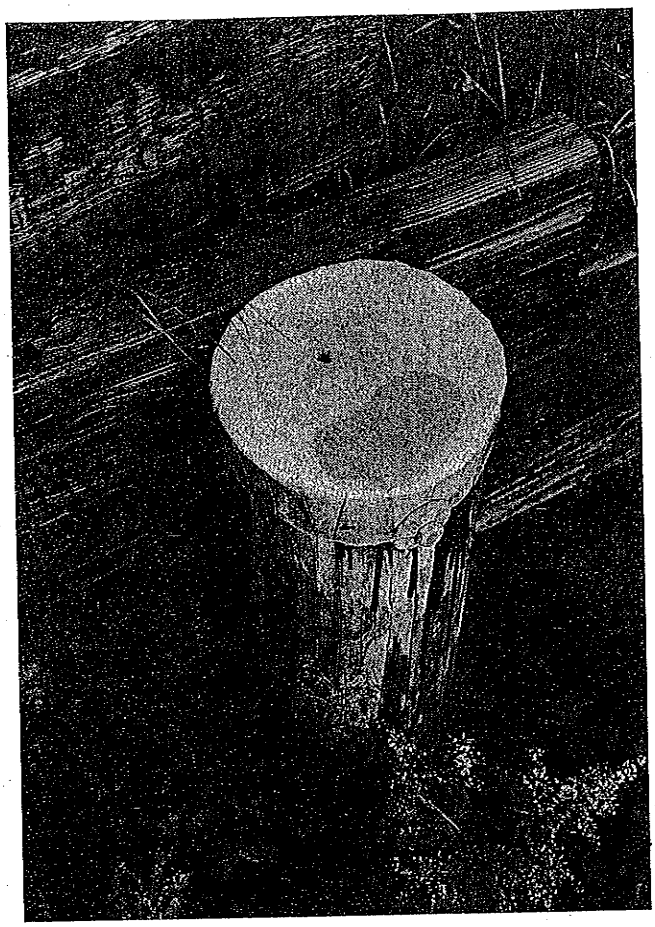


Figure 13-42.—(Top) Damaged metal pile caps allow water to enter, but restrict air circulation and drying. (Bottom) Pile decay is exposed when the damaged cap is removed.

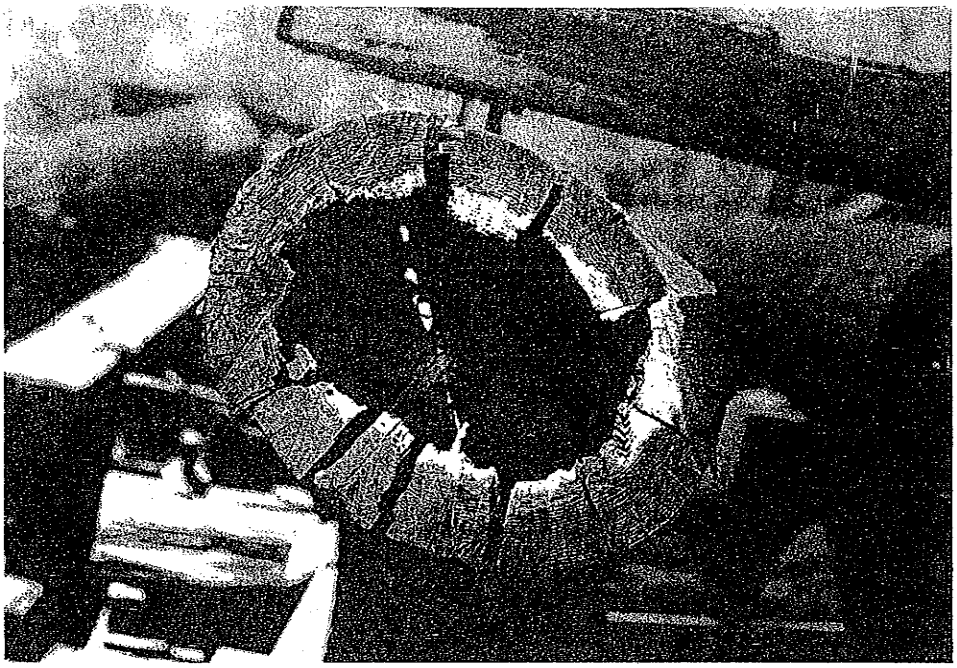


Figure 13-43.—Internal decay in a timber pile where it was drift-pinned to the cap. Before the breakage of the outside shell, caused by cap removal, the pile showed little exterior sign of the interior decay.

Portions of the substructure containing piers or bents use the same basic inspection criteria for the same potential problem areas as abutments. If these structures are in water, however, inspection is much more difficult because access is limited. In water locations, members are also more susceptible to mechanical damage from floating debris and ice. In shallow water, inspectors can wear hip-waders to examine exposed members, whereas in deeper water a small boat or float is required. When inspection below the water level is necessary, the service of a diver is required. Underwater inspections require a high degree of skill and must be well coordinated to accurately identify and record deficiencies.[7,8]

For substructures located in seawater, low tides present the best opportunity to inspect for marine borer damage. Low-tide inspection is best suited for detecting *Limnoria,* which attack the external faces of members. A scraper and probe can be used to remove fouling organisms from the pile surface and thus permit better examination around bolt holes and adjoining wood members. Damage signs include an hourglass shape of piles in the tidal zone, bore holes; a general softening of wood in the attack areas; and loose bolts and bracing. Intertidal inspection is less effective for detecting damage by shipworms because they leave only a very small entrance hole on the wood surface, making visual detection difficult. Inspection methods using sonic instruments represent the best method for evaluating shipworm damage.

In areas where marine borer attack is suspected, an assessment of the hazard potential can be made by immersing sacrificial blocks of untreated wood at various depths around the substructure. These blocks are then removed periodically and examined for evidence of borer attack. Do not depend on the collection of driftwood to evaluate marine borer hazard because there is no way of knowing whether the wood came from sites outside the immediate area. Exposing wood samples can accurately assess marine borer hazard while providing a means for continually monitoring the long-term hazard.

Superstructure Inspection

After completing the substructure inspection, the inspector moves to the underside of the superstructure. It is best to thoroughly inspect all components from the bridge underside before moving to the roadway, since critical components are obscured by the wearing surface and deck. Superstructure inspection is generally hindered because access to the center portions of the underside is difficult or impossible without specialized equipment. When areas cannot be reached with ladders, a vehicle equipped with a mechanical arm or snooper may be required in order to adequately inspect the structure. Because ladders and other inspection equipment must be moved frequently to provide access to elevated areas, it is advisable that the inspection be performed by zones rather than by components. For the purposes of clarity, the following discussions are ordered by component.

Although most elements of the superstructure are out of ground contact, decay potential can be high in areas where water passes from the deck and collects at member interfaces, connections, checks, and crevices where air circulation and drying are inhibited. In many cases, this decay occurs with little or no surface evidence, although the member may be severely decayed inside. As a result, the inspector must be alert for conditions conducive to decay and must investigate areas where these conditions are likely to occur. As previously discussed, a moisture meter is a good tool for locating moisture conditions favorable to decay development (Figure 13-44). At least one boring should be made in areas of high moisture content where decay potential is considered highest. If decay is detected, additional borings should be taken to define its area, degree, and extent. If no decay is detected, but preservative penetration is shallow or moisture content is above 30 percent, it is desirable to remove a core for culturing to determine whether decay fungi are present.

The highest potential for decay in beams occurs at the deck-beam interface and attachment points, framing connections to other members, bearings, and seasoning checks. The deck-beam interface is one of the most frequent decay areas because water passing through the deck is trapped and enters fastener holes at the beam top. The hazard is highest when decks are attached with nails or lag screws that penetrate the top surface of the beam

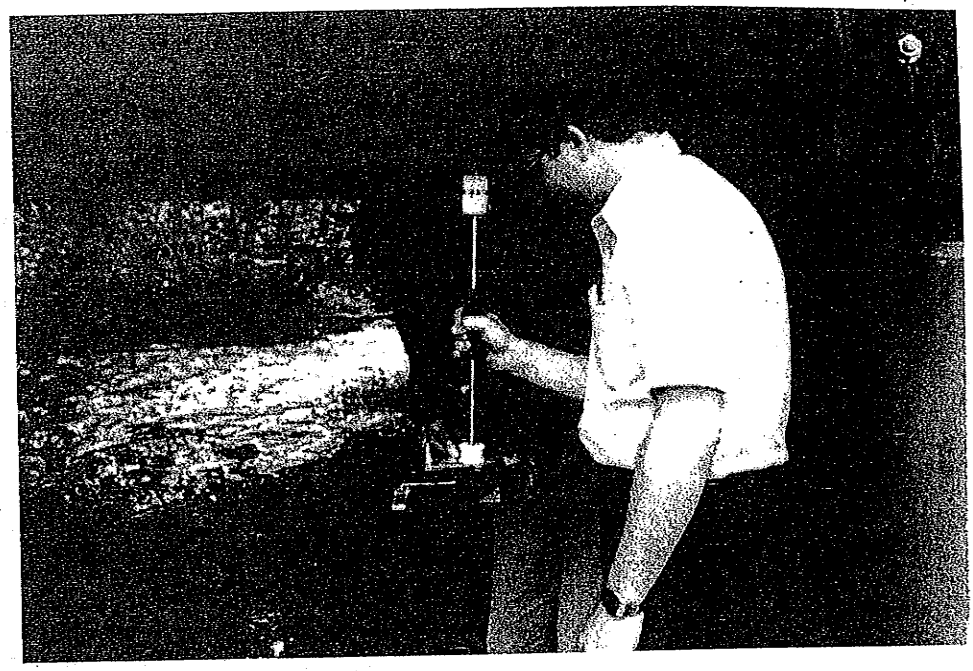


Figure 13-44.—The moisture content of a timber beam is measured with a resistance-type moisture meter.

(Figure 13-45). Glulam deck panels with bolted brackets do not involve attachments that penetrate the beam; thus, there is no significant increase in decay potential. On the deck underside, the inspector should be alert for signs of water movement and the presence of moisture at joint interfaces. Although stains are generally visible when water has passed through the deck, asphalt wearing surfaces tend to filter runoff, and visible signs are more difficult to detect. If significant decay is found along beam tops, it is advisable to remove deck sections to further examine beam condition.

In addition to the deck-beam interface and attachments, beam decay may develop in checks or delaminations, especially in the areas where end grain is exposed. Large checks or delaminations are not common in glulam and may be an indication of more severe structural problems. Bearings that trap water or show signs of beam crushing, and fasteners for transverse bracing or diaphragms are other potential decay locations. Sagging, splintering, or excessive deflections under load may also indicate mechanical damage or possible advanced decay. In some situations, surface decay may be present on a beam side or bottom that does not appear to be in an environment conducive to decay (Figure 13-46). Decay in such locations can occur in sawn lumber beams because of incomplete preservative penetration of heartwood.

Concurrent with beam inspection, the deck underside should be examined for signs of deterioration and conditions conducive to decay. Signs to observe include abnormal deflections and loose joints or fasteners, both of which may result from decay. Nail-laminated decks are frequently

Figure 13-45.—Severe decay in the tops of sawn lumber beams where the deck was attached to the beams with spikes.

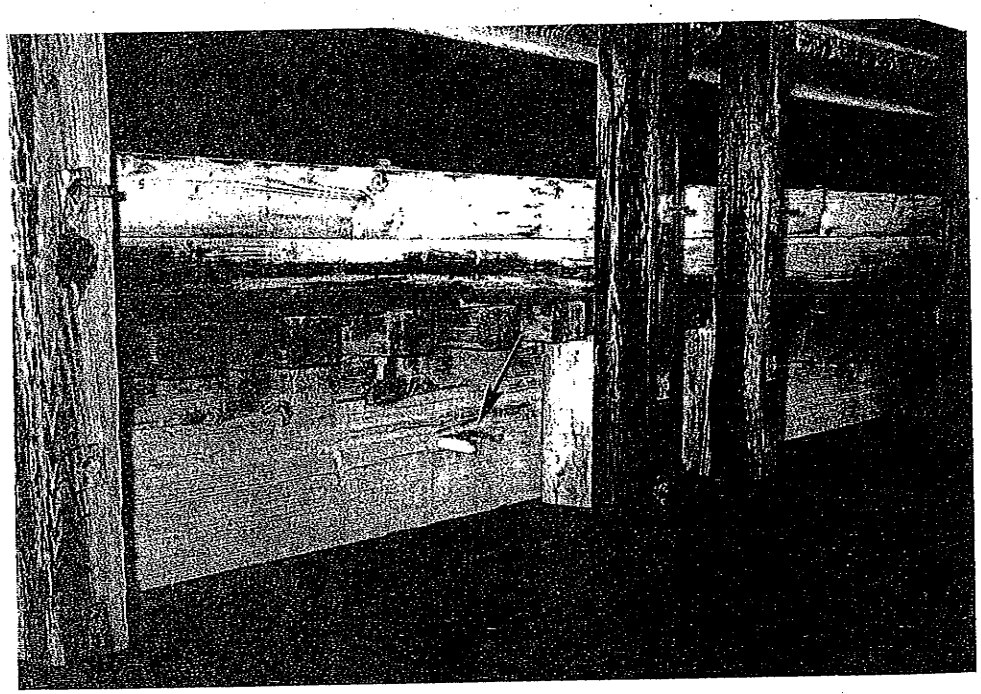


Figure 13-46.—Surface decay on the side of a sawn lumber beam (arrow). Decay in s locations is usually the result of poor preservative penetration of the heartwood.

delaminated by dynamic loading. Although delamination may not adversely affect strength, it does create voids between laminations, allowing water to flow on supporting beams and other components. Susceptibility to internal deck decay is highest with nail-laminated lumber or plank decks because they are interconnected and/or attached with nails or spikes (Figure 13-47). All fabrication for glulam panels is generally done before preservative treatment and the decay potential is lower unless panels are attached with spikes, lag screws, or other fasteners placed after the deck is treated.

Figure 13-47.—Decay on the underside of a spike-attached lumber plank deck at the deck-beam interface (arrow).

When inspection of the bridge underside is complete, efforts are next directed to the roadway portion of the deck. The upper deck is subject to wear and abrasion from traffic, and the horizontal surface facilitates water and debris accumulation. The highest decay potential occurs at fasteners or zones of mechanical damage and is influenced by the degree of protection provided by the wearing surface. A partial wearing surface affords the least deck protection because the gap between the running surfaces traps debris and moisture. On watertight glulam or stress-laminated timber decks, standing water may accumulate between running planks and remain for long periods. Moisture is also trapped under steel plate or full-plank surfaces where penetrating fasteners are normally placed after deck treatment. Asphalt wearing surfaces do not use mechanical fasteners, but moisture can accumulate at the deck interface when the surface is cracked or otherwise broken from excessive deflection.

The moisture content of timber decks generally averages 20 percent, but may frequently be much higher.[35] The inspector should carefully check exposed deck surfaces for moisture content and other conditions conducive to decay. When deck moisture contents are high, it is advisable to remove a number of cores from sites near the fasteners and other high-hazard locations. If necessary, portions of the wearing surface should be removed to assess deck condition. If evidence of substantial deterioration is found, the entire wearing surface should be removed to thoroughly inspect the deck.

Timber rails and curbs (wheel guards) are some of the most exposed elements of the bridge superstructure, yet are often ignored in bridge inspection. Although they are not critical for support of the structure, they are important for user safety and should be thoroughly inspected. Rails and curbs are susceptible to weathering, seasoning checks, and vehicle impact or abrasion. Rails and curbs are commonly the last components installed during the construction process and their installation presents an increased potential for field cutting and boring to meet alignment requirements. The inspector should pay particular attention to fasteners and areas that trap water and debris. One very probable decay situation occurs when approach railposts are embedded in concrete (Figure 13-48).

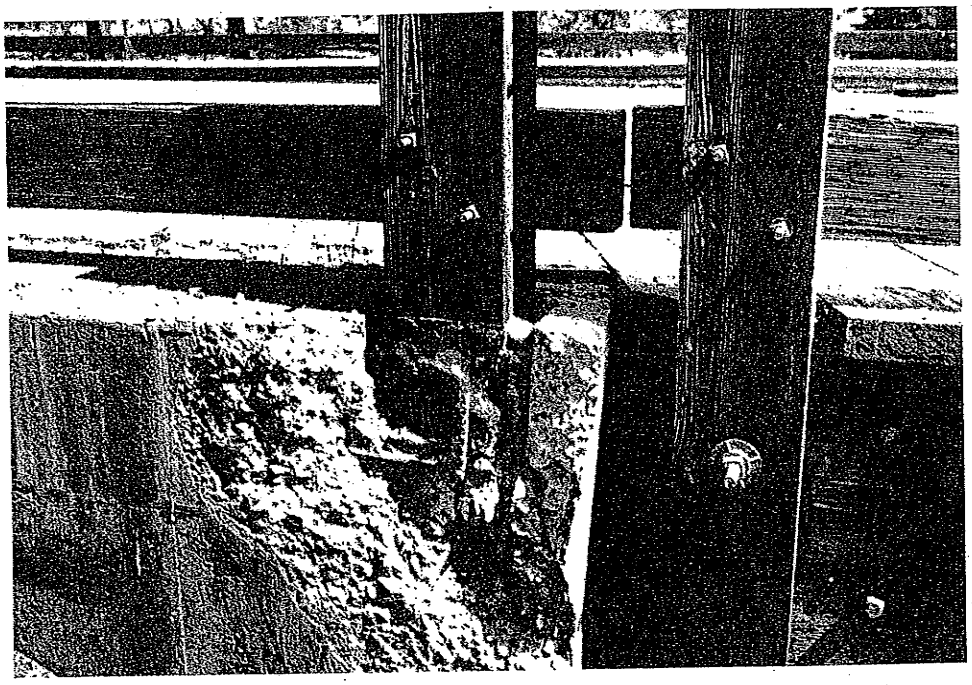


Figure 13-48.—Decay in a timber railpost embedded in concrete at the abutment. Concrete spalling was caused when water trapped in the post cavity was subjected to freeze-thaw cycles.

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SECTION 6—LOAD AND RESISTANCE FACTOR RATING

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SECTION 6

LOAD AND RESISTANCE FACTOR RATING

6.1 INTRODUCTION

6.1.1 General

The load and resistance factor rating procedures of this section provide a methodology for load rating a bridge consistent with the load and resistance factor design philosophy of the *AASHTO LRFD Bridge Design Specifications*. The specific load ratings are used in identifying the need for load posting or bridge strengthening and in making overweight-vehicle permit decisions. Load ratings are routinely reported to the NBI for national bridge administration and are also used in local bridge management systems.

Bridge ratings are based on information in the bridge file, including the results of a recent field inspection. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or loading noted during the inspection.

C6.1.1

There is a large inventory of existing bridges that have been rated using Allowable Stress or Load Factor methods. These ratings may be used for reporting existing bridges to the NBI while new bridges designed by LRFD should be reported using LRFR. Appendix D6.1 contains the other rating specifications.

6.1.2 Scope

This section provides procedures for the rating of bridges using the load and resistance factor philosophy. Procedures are presented for load rating bridges for the LRFD design loading, AASHTO and State legal loads, and overweight permit loads. These procedures are consistent in philosophy and approach of the *AASHTO LRFD Bridge Design Specifications*. The methodology is presented in a format using load and resistance factors that have been calibrated based upon structural reliability theory to achieve a minimum target reliability for the strength limit state. Guidance is provided on service limit states that are applicable to bridge load rating.

This Manual is intended for use in evaluating the types of highway bridges commonly in use in the United States that are subjected primarily to permanent loads and vehicular loads. Methods for the evaluation of existing bridges for extreme events such as earthquake, vessel collision, wind, flood, ice, or fire are not included herein. Rating of long-span bridges, movable bridges, and other complex bridges may involve additional considerations and loadings not specifically addressed in this Manual and the rating procedures should be augmented with additional evaluation criteria where required.

Specific provisions for the evaluation of curved girder steel bridges are not included in this Manual as criteria are still being developed for LRFD curved-girder design.

C6.1.2

The service limit states are not calibrated based upon reliability theory to achieve a target reliability, but are based on past practice. This Manual provides guidance to incorporate these traditional service limit states into the evaluation.

This Manual's primary focus is the assessment of the safety of bridges for live loads (including overloads) and fatigue. Extreme events have a very low probability of occurrence, but impart very high-magnitude forces on a structure. Study of past bridge failures indicates that failure due to hydraulics (scour/ice/debris) is the most common failure mode across the United States. Earthquake can also be a significant failure mode for bridges in regions considered to be seismically active. Bridges over navigable waterways with inadequate pier protection may be highly vulnerable to failure by vessel collision. The vulnerability to extreme events is an important bridge design consideration but it holds even greater significance in the overall safety assessment of existing bridges. It is important that Bridge Owners and evaluators recognize the vulnerabilities to these other failure modes so that a comprehensive safety assurance program may be developed for in-service bridges on a consistent and rational basis.

though the specification does not predict such distress, deviation from the governing specifications based upon the known behavior of the member under traffic may be used and shall be fully documented. Material sampling, instrumentation, and load tests may be helpful in establishing the load capacity for such members.

bridge types that use materials and details no longer in common use. However, the *AASHTO LRFD Bridge Design Specifications* incorporate the state-of-the-art in design and analysis methods, loadings, and strength of materials.

Specifications are calibrated documents in which the loads, load factors, and design methods are part of the whole and should not be separated. Combining factors contained in the original design specifications with those in the current LRFD design specifications should be avoided.

One of the purposes of this Manual is to provide guidance and data on older bridge types and materials that are not covered by the *AASHTO LRFD Bridge Design Specifications*, thereby allowing its application to a large inventory of existing bridges without having to resort to their original design specifications. The Manual seeks to extend the LRFD design philosophy for new bridges to the inventory of existing bridges in a consistent manner.

Evaluators are encouraged to research older materials and design methods as they provide valuable insight into the behavior of the country's older bridges.

6.1.6 Evaluation Methods

This Manual provides three methods for evaluating the safe maximum live-load capacity of bridges or for assessing their safety under a particular loading condition:

- Load and resistance factor rating of bridges,
- Load rating by load testing, and
- Safety evaluation using structural reliability methods for special cases.

Only the first method, load and resistance factor rating of bridges, is discussed in this section. Load testing and safety evaluation for special cases are discussed in Sections 8 and 9, respectively.

C6.1.6

Load testing may be used as an alternative method to directly assess the load capacity of a bridge when analytical methods of evaluation are not applicable or need verification.

Safety assessment of a bridge using structural reliability methods is intended for use in special cases where the uncertainty in load or resistance is significantly different from that assumed in this Manual.

6.1.7 Load and Resistance Factor Rating

Bridge evaluations are performed for varied purposes using different live-load models and evaluation criteria. Evaluation live-load models are comprised of the design live load, legal loads, and permit loads. This section specifies a systematic approach to bridge load rating for these load models, using the load and resistance factor philosophy, aimed at addressing the different uses of load rating results.

C6.1.7

Bridge load ratings are performed for specific purposes, such as: NBI and BMS reporting, local planning and programming, determining load posting or bridge strengthening needs, and overload permit review. Live load models, evaluation criteria, and evaluation procedures are selected based upon the intended use of the load rating results.

6.1.7.2 Legal Load Rating

This second level rating provides a single safe load capacity (for a given truck configuration) applicable to AASHTO and State legal loads. Live-load factors are selected based upon the truck traffic conditions at the site. Strength is the primary limit state for load rating; service limit states are selectively applied. The results of the load rating for legal loads could be used as a basis for decision making related to load posting or bridge strengthening.

6.1.7.3 Permit Load Rating

Permit load rating checks the safety and serviceability of bridges in the review of permit applications for the passage of vehicles above the legally established weight limitations. This is a third level rating that should be applied only to bridges having sufficient capacity for AASHTO legal loads. Calibrated load factors by permit type and traffic conditions at the site are specified for checking the load effects induced by the passage of the overweight truck. Guidance is also provided on the serviceability criteria that should be checked when reviewing permit applications.

6.1.8 Component-Specific Evaluation

6.1.8.1 Decks

Stringer-supported concrete deck slabs and metal decks that are carrying normal traffic satisfactorily need not be routinely evaluated for load capacity. The bridge decks should be inspected regularly to verify satisfactory performance. The inspection of metal decks should emphasize identifying the onset of fatigue cracks.

Timber decks that exhibit excessive deformations or deflections under normal traffic loads are considered suitable candidates for further evaluation and often control the rating. Capacity of timber plank decks is often controlled by horizontal shear.

C6.1.8.1

Test data indicates that the primary structural action of concrete decks is not flexure, but internal arching or membrane action. There is significant reserve strength in concrete decks designed by the AASHTO *Standard Specifications*. Heavily spalled and deteriorated concrete decks may be checked for punching shear under wheel loads.

6.1.8.2 Substructures

Members of substructures need not be routinely checked for load capacity. Substructure elements such as pier caps and columns should be checked in situations where the Engineer has reason to believe that their capacity may govern the load capacity of the entire bridge.

Where deemed necessary by the Engineer, load rating of substructure elements and checking of

C6.1.8.2

Examples of distress that could trigger a load rating of substructure components include: a high degree of corrosion and section loss, changes in column end conditions due to deterioration, changes in column unbraced length due to scour, or columns with impact damage.

Special-emphasis inspection would entail a 100 percent hands-on visual inspection. Fracture-critical

permanent loads and vehicular loads are considered to be of consequence in load rating. Environmental loads such as wind, ice, temperature, stream flow, and earthquake are usually not considered in rating except when unusual conditions warrant their inclusion.

6.2.2 Permanent Loads and Load Factors

The load rating of bridges shall consider all permanent loads. Permanent loads include dead loads and locked-in force effects from the construction process.

6.2.2.1 Dead Loads: *DC* and *DW*

The dead-load effects on the structure shall be computed in accordance with the conditions existing at the time of analysis. Dead loads should be based on dimensions shown on the plans and verified with field measurements. Where present, utilities, attachments, and thickness of wearing surface should be field verified at the time of inspection. Minimum unit weights of materials used in computing dead loads should be in accordance with LRFD Table 3-3, in the absence of more precise information.

6.2.2.2 Permanent Loads Other Than Dead Loads: *P*

Secondary effects from prestressing shall be considered as permanent loads.

6.2.2.3 Load Factors

Load factors for permanent loads are as given in Table 6-1.

Where the permanent load increases the live load effects, the maximum load factor should be used. If the wearing surface thickness is field measured, γ_{DW} may be taken as 1.25.

A load factor of 1.0 shall be applied to the secondary effects from post-tensioning, cited in A6.2.2.2 ($\gamma_p = 1.0$).

6.2.3 Transient Loads

6.2.3.1 Vehicular Live Loads (Gravity Loads): *LL*

The nominal live loads to be used in the evaluation of bridges are selected based upon the purpose and intended use of the evaluation results. Live-load models for load rating include:

C6.2.2

Allowance for future wearing surface need not be provided in evaluation.

C6.2.2.1

Care should be exercised in estimating the weight of concrete decks because significant variations of deck thickness have been found. Wearing surface thicknesses are also highly variable. Multiple measurements at curbs and roadway centerline should be used to determine an average wearing surface thickness.

C6.2.2.2

In continuous post-tensioned bridges, secondary moments are introduced as the member is stressed.

C6.2.3.1

The evaluation of bridge components to include the effects of longitudinal braking forces, specified in LRFD Article 3.6.4 in combination with dead- and live-load effects, should be done only where the

6.2.3.4 Pedestrian Live Loads: *PL*

Pedestrian loads on sidewalks need not be considered simultaneously with vehicular loads when load rating a bridge unless the Engineer has reason to expect that significant pedestrian loading will coincide with maximum vehicular loading. Pedestrian loads considered simultaneously with vehicular loads in calculations for load ratings shall be the probable maximum loads anticipated, but in no case should the loading exceed the value specified in LRFD Article 3.6.1.6.

6.2.3.5 Wind Loads: *WL* and *WS*

Wind loads need not be considered unless special circumstances justify otherwise.

C6.2.3.5

Wind loads are not normally considered in load rating. However, the effects of wind on special structures such as movable bridges, long-span bridges, and other high-level bridges should be considered in accordance with applicable standards.

6.2.3.6 Temperature Effects: *TG* and *TU*

Temperature effects need not be considered in calculating load ratings for non-segmental bridge components that have been provided with well-distributed steel reinforcement to control thermal cracking.

C6.2.3.6

Where temperature effects are considered, a reduced long-term modulus of elasticity for concrete may be used in the analysis.

Temperature gradient (*TG*) may be considered when evaluating segmental bridges.

6.2.3.7 Earthquake Effects: *EQ*

Earthquake effects need not be considered in calculating load ratings.

C6.2.3.7

In regions prone to seismic activity, the safety of bridges under earthquake loads may be evaluated in accordance with the provisions of *Seismic Retrofitting Manual for Highway Bridges;* FHWA-RD-94-052, May 1995.

6.2.3.8 Creep and Shrinkage: *CR* and *SH*

Creep and shrinkage effects do not need to be considered in calculating load ratings where there is well-distributed reinforcement to control cracking in non-segmental, non-prestressed components.

6.3 STRUCTURAL ANALYSIS

6.3.1 General

Methods of structural analysis suitable for the evaluation of bridges shall be as described in Section 4 of the *AASHTO LRFD Bridge Design Specifications* and in this section.

C6.3.1

Evaluation seeks to verify adequate performance of existing bridges with an appropriate level of effort. Within a given evaluation procedure, the evaluator has the option of using simplified methods that tend to be somewhat conservative or pursue a more refined approach for improved accuracy. It is recommended that wherever feasible,

The distribution factor is generally lower for increased gage widths.

Exterior girders of existing bridges may have been designed for less capacity than the interior girders. Additionally, they may also be subjected to increased deterioration due to their increased environmental exposure. Approximate methods of analysis for exterior girders are often less reliable than interior girders due to the structural participation of curbs and parapets. The level of structural participation could vary from bridge to bridge. Field testing (load testing) procedures described in Section 8 may be employed to verify the behavior of exterior girders.

Prestressed concrete adjacent box-beam and slab bridges built prior to 1970 may not have sufficient transverse post-tensioning (LRFD C4.6.2.2.1 requires a minimum prestress of 0.25 ksi) to act as a unit. These bridges should be analyzed using the S/D method of live-load distribution provided in the *AASHTO LRFD Bridge Design Specifications*.

6.3.3 Refined Methods of Analysis

Bridges that exhibit insufficient load capacity when analyzed by approximate methods, and bridges or loading conditions for which accurate live-load distribution formulas are not readily available may be analyzed by refined methods of analysis as described in LRFD Article 4.6.3.

When a refined method of analysis is used, a table of distribution factors for extreme force effects in each span should be provided in the load rating report to aid in future load ratings.

C6.3.3

Some cases where refined analysis methods would be considered appropriate include:

- Girder spacings and span lengths outside the range of LRFD-distribution formulas.
- Varying skews at supports.
- Curved bridges.
- Low-rated bridges.
- Permit loads with nonstandard gage widths and large variations in axle configurations.

Many older bridges have parapets, railings, and curbs that are interrupted by open joints. The stiffness contribution of these elements to bridge response should be verified by load testing, if they are to be included in a refined analysis.

Most analytical models are based on linear response, where load effect is proportional to the load applied. Conversely, the resistance models used for design and evaluation assume nonlinear response at the strength limit state. The rationale for this inconsistency is found in the "lower bound theorem" which states that for a structure that behaves in a ductile manner the collapse load computed on the basis of an assumed equilibrium moment diagram is less than or equal to the true ultimate collapse load. Restated in simpler terms, the theorem implies that as long as the requirements of ductility and equilibrium are satisfied, the exact

6.4.2 General Load-Rating Equation

6.4.2.1 General

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_p)(P)}{(\gamma_L)(LL + IM)} \qquad \text{Eq. (6-1)}$$

For the Strength Limit States:

$$C = \varphi_c \varphi_s \varphi R_n$$

Where the following lower limit shall apply:

$$\varphi_c \varphi_s \geq 0.85$$

For the Service Limit States:

$$C = f_R$$

Where:

RF = Rating factor
C = Capacity
f_R = Allowable stress specified in the LRFD code
R_n = Nominal member resistance (as-inspected)
DC = Dead-load effect due to structural components and attachments
DW = Dead-load effect due to wearing surface and utilities
P = Permanent loads other than dead loads
LL = Live-load effect
IM = Dynamic load allowance
γ_{DC} = LRFD load factor for structural components and attachments
γ_{DW} = LRFD load factor for wearing surfaces and utilities
γ_p = LRFD load factor for permanent loads other than dead loads = 1.0
γ_L = Evaluation live-load factor
φ_c = Condition factor
φ_s = System factor
φ = LRFD resistance factor

The load rating shall be carried out at each applicable limit state and load effect with the lowest value determining the controlling rating factor. Limit states and load factors for load rating shall be selected from Table 6-1.

C6.4.2.1

It should be noted that load modifiers (η) relating to ductility, redundancy, and operational importance contained in the *AASHTO LRFD Bridge Design Specifications* (Article 1.3.2.1) are not included in the general load-rating equation. In load rating, ductility is considered in conjunction with redundancy and incorporated in the system factor φ_s. Operational importance is not included as a factor in the load rating provisions of this Manual.

The load rating of a deteriorated bridge should be based on a recent thorough field inspection. Only sound material should be considered in determining the nominal resistance of the deteriorated section. Load ratings may also be calculated using as-built member properties to serve as a baseline for comparative purposes.

Resistance factor φ has the same value for new design and for load rating. Also, $\varphi = 1.0$ for all nonstrength limit states. For condition factors, see Article 6.4.2.3. For system factors, see Article 6.4.2.4.

6.4.2.3 Condition Factor: φ_c

Use of Condition Factors as presented below may be considered optional based on an agency's load-rating practice.

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

Table 6-2 Condition Factor: φ_c.

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

C6.4.2.3

The uncertainties associated with the resistance of an existing intact member are at least equal to that of a new member in the design stage. Once the member experiences deterioration and begins to degrade, the uncertainties and resistance variabilities are greatly increased (scatter is larger).

Additionally, it has been observed that deteriorated members are generally prone to an increased rate of future deterioration when compared to intact members. Part of φ_c relates to possible further section losses prior to the next inspection and evaluation.

Improved inspections will reduce, but not totally eliminate, the increased scatter or resistance variability in deteriorated members. Improved inspection and field measurements will reduce the uncertainties inherent in identifying the true extent of deterioration for use in calculating the nominal member resistance. If section properties are obtained accurately, by actual field measurement of losses rather than by an estimated percentage of losses, the values specified for φ_c in Table 6-2 may be increased by 0.05 ($\varphi_c \leq 1.0$).

The condition factor, φ_c, tied to the structural condition of the member, accounts for the member deterioration due to natural causes (i.e., atmospheric corrosion). Damage caused by accidents is specifically not considered here.

If condition information is collected and recorded in the form of NBI condition ratings only (not as element level data), then the following approximate conversion may be applied in selecting φ_c.

Table C6-1 Approximate Conversion in Selecting φ_c.

Superstructure Condition Rating (SI & A Item 59)	Equivalent Member Structural Condition
6 or higher	Good or Satisfactory
5	Fair
4 or lower	Poor

6.4.2.4 System Factor: φ_s

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings.

System factors that correspond to the load factor modifiers in the *AASHTO LRFD Bridge Design Specifications* should be used. The system factors in Table 6-3 are more conservative than the LRFD-design values and may be used at the

C6.4.2.4

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Bridge redundancy is the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members. Internal redundancy and structural redundancy that exists as a result of continuity are neglected when classifying a member as non-redundant.

If Table 6-3 is used, the system factors are used to maintain an adequate level of system safety. Non-

6.4.3 Design-Load Rating

6.4.3.1 Purpose

The design-load rating assesses the performance of existing bridges utilizing the LRFD-design loading (HL-93) and design standards. The design-load rating of bridges may be performed at the same design level (Inventory level) reliability adopted for new bridges by the *AASHTO LRFD Bridge Design Specifications* or at a second lower-level reliability comparable to the Operating level reliability inherent in past load-rating practice. The design-load rating produces Inventory and Operating level rating factors for the HL-93 loading.

The design-load rating serves as a screening process to identify bridges that should be load rated for legal loads, per the following criteria:

- Bridges that pass HL-93 screening at the Inventory level will have adequate capacity for all AASHTO legal loads and State legal loads that fall within the exclusion limits described in the *AASHTO LRFD Bridge Design Specifications*.

- Bridges that pass HL-93 screening only at the Operating level will have adequate capacity for AASHTO legal loads, but may not rate (RF > 1) for all State legal loads, specifically those vehicles significantly heavier than the AASHTO trucks.

The results are also suitable for use in NBI reporting, and bridge management and bridge administration, at a local or national level. The rating results for service and fatigue limit states could also guide future inspections by identifying vulnerable limit states for each bridge.

6.4.3.2 Live Loads and Load Factors

6.4.3.2.1 Live Load

The LRFD-design, live-load HL-93 (see Appendix B.6.1) shall be used.

6.4.3.2.2 Live-Load Factors

The evaluation live-load factors for the Strength I limit state shall be taken as shown in Table 6-4.

Table 6-4 Load Factors for Design Load: γ_L.

Evaluation Level	Load Factor
Inventory	1.75
Operating	1.35

C6.4.3.1

The design-load rating is performed using dimensions and properties for the bridge in its present condition, obtained from a recent field inspection.

No further evaluation is necessary for bridges that have adequate capacity ($RF > 1$) at the Inventory level reliability for HL-93. Bridges that pass HL-93 screening only at the Operating level reliability will not have adequate capacity for State legal loads significantly heavier than the AASHTO legal loads. Existing bridges that do not pass a design-load rating at the Operating level reliability should be evaluated by load rating for AASHTO legal loads using procedures provided in this section.

C6.4.3.2.2

Service limit states that are relevant to design-load rating are discussed under the articles on resistance of structures (see Sections 6.5, 6.6, and 6.7).

- For negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 multiplied by 0.75 heading in the same direction separated by 30 ft.

In addition, for span lengths greater than 200 ft., critical load effects shall be created by:

- AASHTO Type 3-3 multiplied by 0.75 and combined with a lane load of 0.2 klf.

Dynamic load allowance shall be applied to the AASHTO legal vehicles and not the lane loads. If the ADTT is less than 500, the lane load may be excluded and the 0.75 factor changed to 1.0 if, in the Engineer's judgment, it is warranted.

AASHTO legal vehicles, designated as Type 3, Type 3S2, and Type 3-3 are sufficiently representative of average truck configurations in use today, and are used as vehicle models for load rating. These vehicles are also suitable for bridge posting purposes. Load ratings may also be performed for State legal loads that have only minor variations from the AASHTO legal loads using the live-load factors provided in Table 6-5 for the AASHTO vehicles. It is unnecessary to place more than one vehicle in a lane for spans up to 200 ft. because the load factors provided have been modeled for this possibility.

The federal bridge formula (Reference: TRB Special Report 225, *Truck Weight Limits Issues and Options*, 1990) restricts truck weights on interstate highways through (a) a total, or gross, vehicle weight limit of 80 kips; (b) limits on axle loads (20 kips for single axles, 34 kips for tandem axles); and (c) a bridge formula that specifies the maximum allowable weight on any group of consecutive axles. based on the number of axles in the group and the distance from first to the last axles. Grandfather provisions in the federal statutes allow states to retain higher limits than these if such limits were in effect when the applicable federal statutes were first enacted.

The objective of producing new *AASHTO LRFD Bridge Design Specifications* that will yield designs having uniform reliability required as its basis a new live-load model with a consistent bias when compared with the exclusion vehicles. The model consisting of either the HS-20 truck plus the uniform lane load or the tandem plus the uniform lane load (designated as HL-93 loading) resulted in a tight clustering of data around a 1.0 bias factor for all force effects over all span lengths. This combination load was therefore, found to be an adequate basis for a notional design load in the *AASHTO LRFD Bridge Design Specifications.*

While this notional design load provides a convenient and uniform basis for design and screening of existing bridges against new bridge safety standards, it has certain limitations when applied to evaluation. The notional design load bears no resemblance or correlation to legal truck limits on the roads and poses practical difficulties when applied to load rating and load posting of existing bridges.

A characteristic of the AASHTO family of legal loads (Type 3, Type 3S2, Type 3-3) is that the group satisfies the federal bridge formula. The AASHTO legal loads model three portions of the bridge formula which control short, medium, and long spans. Therefore, the combined use of these three AASHTO legal loads results in uniform bias over all span lengths, as achieved with the HL-93 notional load model (see Appendix B6.4). These vehicles are presently widely used for load rating

The live-load factors in Table 6-5 were determined, in part, by reducing the target beta level from the design level of 3.5 to the corresponding operating level of 2.5, according to NCHRP Report 454. Several parametric analyses indicate this reduction in beta corresponds to a reduced load factor ratio of about 0.76 (i.e., 1.35/1.75). Thus, the load factors in Table 6-5 have been calibrated to represent an equivalent Operating level of loading. Therefore, it is reasonable to increase the load factor up to the design target beta level (or equivalent Inventory level of loading), if the Engineer deems appropriate, by multiplying by the reciprocal of 0.76 or 1.3.

Site-Specific, Live-Load Factors

Consideration should be given to using site-specific load factors when a bridge on a low-volume road may carry unusually heavy trucks or industrial loads due to the proximity of the bridge to an industrial site.

When both truck weight and truck traffic volume data are available for a specific bridge site, appropriate load factors can be derived from this information. Truck weights at a site should be obtained by generally accepted weigh-in-motion technology. In general, such data should be obtained by systems able to weigh all trucks without allowing heavy overweight vehicles to bypass the weighing operation.

To obtain an accurate projection of the upper tail of the weight histogram, only the largest 20 percent of all truck weights are considered in a sample for extrapolating to the largest loading event. A sufficient number of truck samples need to be taken to provide accurate parameters for the weight histogram.

For a two- or more than two-lane loading case, the live-load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{2W^* + t_{(ADTT)}\,1.41\sigma^*}{240}\right] > 1.30 \qquad \text{Eq. (C6-1)}$$

For the single-lane loading case, the live load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{W^* + t_{(ADTT)}\,\sigma^*}{120}\right] > 1.80 \qquad \text{Eq. (C6-2)}$$

where:

W^* = Mean truck weight for the top 20 percent of the weight sample of trucks (kips)

fluctuations. The data collection period should be sufficient to capture at least 400 trucks in the upper 20 percent of the weight sample for the site. Additional guidance on determining site-specific load factors can be found in the NCHRP Report 454.

6.4.4.3 Dynamic Load Allowance: *IM*

The static effects of the truck loads shall be increased by 33 percent for strength and service limit states to account for the dynamic effects due to moving vehicles. The dynamic load allowance shall be applied only to the axle loads when the lane type load given in Appendix B.6.2 is used for evaluation.

The dynamic load allowance for the evaluation of wood components shall be reduced to 50 percent of the values specified.

C6.4.4.3

The factor to be applied to the static load effects shall be taken as: (1+ I*M*/100). The factors are applicable to simple and continuous span configurations.

The dynamic response of a bridge to a crossing vehicle is a complex problem affected by the pavement surface conditions and by the dynamic characteristics of both the bridge and vehicle. In the majority of bridge load tests, roadway imperfections, and irregularities were found to be a major factor influencing bridge response to traffic loads. The 33 percent dynamic load allowance specified deliberately reflects conservative conditions that may prevail under certain distressed approach and bridge deck conditions with bumps, sags, or other major surface deviations and discontinuities. In longitudinal members having spans greater than 40 ft. with less severe approach and deck surface conditions, the dynamic load allowance (*IM*) may be decreased as given below in Table C6-3:

Table C6-3 Dynamic Load Allowance: *IM*.

Riding Surface Conditions	*IM*
Smooth riding surface at approaches, bridge deck, and expansion joints	10%
Minor surface deviations or depressions	20%

Providing a dynamic load allowance primarily as a function of pavement surface conditions is considered a preferred approach for evaluation. Pavement conditions that were not known to the designer are apparent to the inspector/evaluator. The riding surface conditions used in Table C6-3 are not tied to any measured surface profiles, but are to be selected based on field observations and judgment of the evaluator. Condition of deck joints and concrete at the edges of deck joints affect rideability and dynamic forces induced by traffic. Inspection should carefully note these and other surface discontinuities in order to benefit from a reduced dynamic load allowance.

The dynamic load allowance for components determined by field testing may be used in lieu of values specified herein. The use of full-scale dynamic testing under controlled or normal traffic conditions remains the most reliable way of

6.4.5.2 Purpose

Section 6.4.5 provides procedures for checking bridges to determine the load effects induced by the overweight permit loads and their capacity to safely carry these overloads. Permit load rating should be used only if the bridge has a rating factor greater than 1.0 when evaluated for AASHTO legal loads.

C6.4.5.2

Permit vehicles should be rated by using load-rating procedures given in Section 6.4.5, with load factors selected based upon the permit type, loading condition, and site traffic data. The live load to be used in the load-rating equation for permit decisions shall be the actual permit vehicle weight and axle configuration.

The factors recommended for evaluating permit loads are calibrated with the assumptions that the bridge, as a minimum, can safely carry AASHTO legal loads, as indicated by the evaluation procedures given in Article 6.4.4. This requirement is especially evident when using reduced live-load factors for permits based on a small likelihood that there will be multiple presence of more than one heavy vehicle on the span at one time. Such multiple presence situations are considered in the calibration of the checking equations of both the *AASHTO LRFD Bridge Design Specifications* and the evaluation procedures given in this Manual.

6.4.5.3 Permit Types

6.4.5.3.1 Routine (Annual) Permits

Routine permits are usually valid for unlimited trips over a period of time, not to exceed one year. The permit vehicles may mix in the traffic stream and move at normal speeds without any movement restrictions.

6.4.5.3.2 Special (Limited Crossing) Permits

Special permits are usually valid for a single trip only or for a limited number of trips. These permit vehicles are usually heavier than those vehicles issued routine permits.

Single-trip permits are good for only one trip during a specified period of time (typically 3–5 days). Multiple-trip permits grant permission to transport overweight shipments during a 30–90 day period.

Single-trip permits for excessively heavy loads may have certain conditions and restrictions imposed to reduce the load effect, including, but not limited to:

- Requiring the use of escorts to restrict all other traffic from the bridge being crossed.
- Requiring the permit vehicle to be in a certain position on the bridge (e.g., in the center or to one side) to reduce the loading on critical components.

C6.4.5.3.2

Upper limit of 100 special permit crossings was used for calibration purposes in this Manual. Permits operating at a higher frequency should be evaluated as routine permits.

Table 6-6 Permit load Factors: γ_L

Permit Type	Frequency	Loading Condition	*DF*[a]	ADTT (one direction)	Load Factor by Permit Weight[b]	
					Up to 100 kips	≥150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80	1.30
				=1000	1.60	1.20
				<100	1.40	1.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

Notes:

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

[b] For routine permits between 100 kips and 150 kips, interpolate the load factor by weight and ADTT value. Use only axle weights on the bridge.

The live-load distribution analysis for routine permits is done using LRFD two-lane distribution factors which assume the simultaneous side-by-side presence of two equally heavy vehicles in each lane. This condition is too conservative for permit load analysis. The live-load factors herein were derived to account for the possibility of simultaneous presence of non-permit heavy trucks on the bridge when the permit vehicle crosses the span. Thus, the load factors are higher for spans with higher ADTTs and lower for heavier permits. The live-load factors in Table 6-6 for routine permits must be applied together with the upper limit of permit weights operating under a single permit and the corresponding two-lane distribution factor.

For situations where the routine permit is below 100 kips, the live-load factors are the same as those given for evaluating legal loads. This requirement reflects the fact that in a traffic stream, the presence of random, heavy, overloaded vehicles may control the extreme loading case when compared to permit weights, which are close to the limit of 80 kips. When the routine permit weight is above 100 kips, then the live-load factors are reduced as shown in Table 6-6. This reduction reflects the lower probability of two simultaneously heavy vehicles equal to the permit weight crossing the span at the same instant (LRFD two-lane distribution factor assumes that an identical vehicle is simultaneously present in each lane). The calibration of these live-load factors for routine permits uses the same traffic statistics used in calibrating the *AASHTO LRFD Bridge Design Specifications* as well as the evaluation factors

6.4.5.5 Dynamic Load Allowance: *IM*

The dynamic load allowance to be applied for permit load rating shall be as specified in Article 6.4.4.3 for legal loads, except that for slow moving (≤10 mph) permit vehicles the dynamic load allowance may be eliminated.

6.4.5.6 Exterior Beams

Permit load factors given in Table 6-6 are applicable to both interior and exterior beam ratings. Distribution of live load to exterior beams as defined in LRFD Article 4.6.2.2.2d shall apply with the following modifications:

- For special permits, use a one-lane loaded condition only. Where a one-lane loaded condition is assumed, the LRFD multiple presence factor need not be applied (the built-in multiple presence factor in the LRFD one-lane distribution factor should be divided out).
- For routine permits, a multi-lane loaded condition shall be assumed. Permit trucks of equal weights shall be assumed to be present in each lane in determining the governing distribution factor.

C6.4.5.6

In LRFD, live-load distribution to the exterior beams for bridges with diaphragms or cross-frames must be checked by an additional investigation that assumes rigid body behavior of the section, per LRFD Article 4.6.2.2.2d.

6.4.5.7 Continuous Spans

Closely spaced heavy axles can cause uplift in end spans of continuous bridges. During permit reviews, uplift in continuous span bridges and its effect upon bearings should be considered.

C6.4.5.7

When the upward *LL* reaction reduces the total reaction to less than 10 percent of normal downward *DL* reaction, uplift may be considered to occur. Unless the uplift is counteracted (by weights or tie-downs), the vehicle should not be permitted on the bridge.

6.5 CONCRETE STRUCTURES

6.5.1 Scope

The provisions of Section 6.5 apply to the evaluation of concrete bridge components reinforced with steel bars and/or prestressing strands or bars. The provisions of this section combine and unify the requirements for reinforced and prestressed concrete.

6.5.2 Materials

6.5.2.1 Concrete

When the compressive strength of concrete, f'_c, is unknown and the concrete is in satisfactory condition, f'_c, for reinforced concrete superstructure

C6.5.2.1

Cores may also be taken where the initial load capacity based on design concrete strength is considered inadequate. Concrete strength may have

6.5.4 Limit States

The applicable limit states and their load combinations for the evaluation of concrete members are specified for the various rating procedures. The load combinations, and the load factors which comprise them, are specified in Table 6-1 and in these articles.

Table 6-9 Tensile Strength of Prestressing Strand.

Year of Construction	Tensile Strength, f_{pu} ksi
Prior to 1963	232.0
1963 and later	250.0

6.5.4.1 Design-Load Rating

The Strength I load combinations shall be checked for reinforced concrete components. The Strength I and Service III load combinations shall be checked for prestressed concrete components.

C6.5.4.1

Service III need not be checked for HL-93 at the Operating level as Service III is a Design-level check for crack control in prestressed components.

The Service I load combination of the *AASHTO LRFD Bridge Design Specifications* need not be checked for reinforced concrete bridges, as it pertains to the distribution of reinforcement to control crack widths in reinforced concrete beams. Distribution of reinforcement for crack control is a design criterion that is not relevant to evaluation. In LRFD, Service I is also used to check compression in prestressed concrete bridges. This check may govern at prestress transfer, but usually will not govern live-load capacity under service conditions.

Most prestressed designs are designed for no cracking under full-service loads. Fatigue is not a concern until cracking is initiated. Hence, prestressed components need not be routinely checked for fatigue.

Rating factors for applicable limit states computed during design-load rating will aid in identifying vulnerable limit states for further evaluation and future inspections.

6.5.4.2 Legal load Rating and Permit load Rating

Load ratings for legal loads and permit loads shall be based on satisfying the requirements for the strength- and service limit states, guided by considerations presented in these articles.

6.5.4.2.1 Strength Limit State

Concrete bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

that materials and construction are of good quality and there is no loss of material design strength, or, when warranted, the material strength has been established by testing, and any reductions in area due to deterioration have been considered.

matrix and the separation of aggregates due to chemical agents or other causes. In such cases, material sampling and testing should be considered to assess concrete strength and quality.

Deterioration of concrete components does not necessarily reduce their flexural resistance. The actual amount of capacity reduction depends on the type of deterioration and its location. Loss in reinforcing steel area at a critical location could directly affect the moment capacity as concrete beams are normally designed as under-reinforced sections. However, loss of cover due to spalling does not have a significant influence on the flexural resistance, if the ends of the reinforcing steel are properly anchored. Spalls on the compression side could change the internal moment-arm leading to a slight reduction in load capacity.

6.5.6 Maximum Reinforcement

The area of prestressed and non-prestressed reinforcement to be used in calculating flexural resistance shall not exceed the maximum amount permitted in LRFD Article 5.7.3.3.1.

C6.5.6

Load rating of over-reinforced sections should utilize only the area of reinforcement not exceeding the limiting value for maximum reinforcement.

6.5.7 Minimum Reinforcement

Concrete members that do not satisfy the minimum flexural reinforcement provisions of LRFD Article 5.7.3.3.2 shall have their flexural resistance reduced by multiplying by a reduction factor K, where:

$$K = \frac{M_r}{M_{\min}} \qquad \text{Eq. (6-3)}$$

where: $M_{\min}$ = Lesser of 1.2 M_{cr} or 1.33 M_u

M_{cr} = Cracking Moment

$$M_{cr} = (f_r + f_{pb})\,S_{bc} - M_{d,nc}\left(\frac{S_{bc}}{S_b} - 1\right) \qquad \text{Eq. (6-4)}$$

where:

f_r = Modulus of rupture (LRFD Article 5.4.2.6)

f_{pb} = Compressive stress in concrete due to effective prestress in the precompressed tensile zone. Effective prestress for standard prestressed members may be computed using approximate lump sum estimate of time-dependent losses as provided in LRFD Article 5.9.5.3

$M_{d,nc}$ = Non-composite dead-load moment

C6.5.7

The equation for M_{cr} is adapted from the AASHTO *Standard Specifications,* Article 9.18.2.1. Where beams are designed to be non-composite, substitute S_b for S_{bc}.

quite low. Typically, locations near the 0.25 point could be critical because of relatively high levels of both shear and moment. Also contributing to the need for checking multiple locations along the beam is the fact that the stirrup spacings are typically not constant, but vary.

6.5.10 Secondary Effects from Prestressing

Secondary effects from prestressing shall be considered as permanent loads with load factors as cited in Article 6.2.2.2.

C6.5.10

Reactions are produced at the supports in continuous spans under post-tensioning loads, giving rise to secondary moments in the girders. The secondary moments are combined with the primary moments to provide the total moment effect of the post-tensioning.

6.5.11 Concrete Bridges with Unknown Reinforcement

A concrete bridge for which the details of the reinforcement are not known, need not be posted for restricted loading if it has been carrying normal traffic for an appreciable period and shows no distress. The bridge shall be inspected regularly (see Appendix A.6.1.4) to verify satisfactory performance.

C6.5.11

Bridge Owners may consider nondestructive proof load tests to establish a safe load capacity for such bridges. Knowledge of the live loading used in the original design and the current condition of the structure may also provide a basis for assigning a safe load capacity.

6.5.12 Temperature, Creep, and Shrinkage Effects

Temperature, creep, and shrinkage effects need not be considered at the strength limit state for components that have been provided with well-distributed steel reinforcement to control cracking.

C6.5.12

Temperature, creep, and shrinkage are primarily strain inducing effects. As long as the section is ductile, such changes in strain are not expected to cause failure. Where temperature cracks are evident and analysis is considered warranted, temperature effects due to time-dependent fluctuations in effective bridge temperature may be treated as long-term loads, with a long-term modulus of elasticity of concrete reduced to one-third its normal value.

6.5.13 Prestressed Concrete Beams Made Continuous in the Field

The capacity of the mild steel reinforcement over the piers should be checked for the strength limit state as it could control load ratings for this type of structure.

6.6 STEEL STRUCTURES

6.6.1 Scope

The provisions of Section 6.6 shall apply to the evaluation of steel and wrought-iron components of bridges.

6.6.2.2 Pins

If the material designation for pins is unknown, the yield strength may be selected from Table 6-12, based on the year of construction.

Table 6-12 Minimum Yield Point of Pins by Year of Construction.

Year of Construction	Minimum Yield Point, F_y ksi
Prior to 1905	25.5
1905 through 1935	30
1936 through 1963	33
After 1963	36

6.6.2.3 Wrought Iron

When the material designation is unknown for wrought iron, the minimum tensile strength, F_u, should be taken as 48 ksi and the minimum yield point, F_y, should be taken as 26 ksi.

Where practical, coupon tests should be performed to confirm the minimum mechanical properties used in the evaluation.

6.6.3 Resistance Factors

Resistance factors, φ, for steel members, for the strength limit state, shall be taken as specified in LRFD Article 6.5.4.2.

C6.6.3

For service limit states, φ = 1.0.

6.6.4 Limit States

The applicable limit states and their load combinations for the evaluation of structural steel and wrought iron members are specified for the various rating procedures. The load combinations, and the load factors which comprise them, are specified in Table 6-1 and in these articles.

6.6.4.1 Design-Load Rating

Strength I and Service II load combinations shall be checked for the design loading. Live-load factors shall be taken as tabulated in Table 6-1.

In situations where fatigue-prone details are present (category C or lower) a rating factor for infinite fatigue life should be computed. Members that do not satisfy the infinite fatigue life check may be evaluated for remaining fatigue life using procedures given in Section 7 of this Manual. This is an optional requirement.

C6.6.4.1

Rating factors for applicable strength, service, and fatigue limit states computed during the design load rating will aid in identifying vulnerable limit states for further evaluation and future inspections.

fitting to load-factor operating level serviceability rating, the built-in conservatism in the distribution analysis is considered appropriate. Escorted special permits operating with no other vehicles on the bridge may be analyzed using one-lane distribution factors for Service II.

The stress limitation of $0.8F_{yf}$ for the negative moment region of composite spans with longitudinal reinforcement has been found to be conservative. The Autostress design method places no restriction on the maximum stress due to negative moment at overload. Continuous span bridges are allowed to shake down and respond to subsequent overloads in an elastic manner. This can also be applied to the rating of existing bridges. It is reasonable to assume that any bridge that has been in service for some years would have undergone the shakedown process.

6.6.5 Effects of Deterioration on Load Rating

A deteriorated structure may behave differently than the structure as originally designed and different failure modes may govern its load capacity. Corrosion is the major cause of deterioration in steel bridges. Effects of corrosion include section loss, unintended fixities, movements and pressures, and reduced fatigue resistance.

C6.6.5

Tension Members with Section Losses Due to Corrosion

Corrosion loss of metals can be uniform and evenly distributed or it can be localized. Uniform reduction in the cross-sectional area of a tension member causes a proportional reduction in the capacity of the member. Since localized corrosion results in irregular localized reductions in area, a simplified approach to evaluating the effects of localized corrosion is to consider the yielding of the reduced net area as the governing limit state. Due to their self-stabilizing nature, stress concentrations and eccentricities induced by asymmetrical deterioration may be neglected when estimating the tension strength of members with moderate deterioration.

For eyebars and pin plates, the critical section is located at the pin hole normal to the applied stress. In evaluating eyebars with significant section loss in the head, the yielding of the reduced net section in the head should be checked as it may be a governing limit state.

Deterioration of lacing bars and batten plates in built-up tension members may affect the load sharing among the main tension elements at service loads. At ultimate load yielding will result in load redistribution among the tension elements and the effect on capacity is less significant.

Compression Members with Section Losses Due to Corrosion

a) Uniform Corrosion

Local Effects—The susceptibility of members with reduced plate thickness to local buckling should be evaluated with respect to the limiting width/thickness ratios specified in LRFD

batten plates has very little effect on the overall member capacity, as long as the resistance to local failure is satisfactory.

Flexural Members with Section Losses Due to Corrosion

Uniform Corrosion

The reduction in bending resistance of laterally supported beams with stiff webs will be proportional to the reduction in section modulus of the corroded cross-section compared to the original cross-section. Either the elastic or plastic section modulus shall be used, as appropriate. Local and overall beam stability may be affected by corrosion losses in the compression flange.

The reduction in web thickness will reduce shear resistance and bearing capacity due to both section loss and web buckling. When evaluating the effects of web losses, failure modes due to buckling and out-of-plane movement that did not control their original design may govern. The loss in shear resistance and bearing capacity is linear up to the point where buckling occurs.

Localized Corrosion

Small web holes due to localized losses not near a bearing or concentrated load may be neglected. All other web holes should be analytically investigated to assess their effect.

A conservative approach to the evaluation of tension and compression flanges with highly localized losses is to assume the flange is an independent member loaded in tension or compression. When the beam is evaluated with respect to its plastic moment capacity, the plastic section modulus for the deteriorated beam may be used for both localized and uniform losses.

6.6.6 Tension Members

Members and splices subjected to axial tension shall be investigated for yielding on the gross section and fracture on the net section as specified in LRFD Article 6.8.2.

6.6.6.1 Links and Hangers

The following provisions are given for the evaluation of pin-connected tension members other than eyebars:

1) The net section through the pin hole transverse to the axis of the member shall be 40 percent greater than the net section of the main member.

C6.6.6.1

Design of pin and hanger connections assumes free rotation at the pin. Accumulation of dirt and corrosion developed between the elements of the pin and hanger assembly could result in unintended partial or complete fixity of the pin and hanger connection. Very large in-plane bending stresses in the hangers and torsional stresses in the pins could be expected from rotational fixity. The fatigue life of the hangers could also be reduced. Build-up of

sides using either stay plates in combination with single or double lacing, perforated cover plates, or batten plates. To allow for the reduced strength of batten plate compression members only, the actual length of the member shall be multiplied by the adjustment factor given in Table 6-13 to obtain the adjusted value of L/r to be used in computing the column slenderness factor λ.

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

Adjusted L/r (batten plate both sides) β

= Actual L/r × factor

Adjusted L/r (batten plate one side)

= Actual L/r × [1 + 1/2 (factor − 1)]

6.6.8 Combined Axial Compression and Flexure

The load rating of steel members subjected to axial compression and concurrent moments, such as arches and beam-columns, shall be determined using the interaction equations specified in LRFD Article 6.9.2.2.

C6.6.8

Load rating of such members should consider second-order effects, which may be approximated by the single-step moment magnification method given in LRFD Article 4.5.3.2.2b (see Appendix C.6.2).

In compression members with asymmetrical sections (such as truss chords), the gravity axis of the section may not coincide with the working lines, resulting in an eccentric connection. Compression members having equal end eccentricities are conveniently analyzed using the secant formula. The LRFD specification does not utilize the secant formula, but provides an interaction equation for the design of members with combined axial loads and concurrent moments. Rating compression members via an interaction equation can be somewhat tedious as an iterative approach may be required to establish the governing rating. A rating approach using the interaction equation is given in Appendix C.6.2. (M_r must be known to apply this method.)

As an alternative to analyzing axial compression members with eccentric connections as combined compression-flexure members, an axial load magnification factor may be applied to rate the member as a concentrically loaded member with an equivalent load. Secant formula is used to include the first and second order bending effects to produce a magnified axial load (dead and live) that would produce a constant stress over the cross section equal to the peak stress in an eccentric member. This approach is applicable to members assumed to be pinned at the ends and without lateral loads on the member. Pin connected compression chord members in truss bridges are a common example of this type. An advantage inherent in this method is that rating factors can be computed without having to first determine M_r, which can be difficult to do for nonstandard truss sections (see Appendix C.6.3).

be checked where the top flange of the girder is fully in contact with the deck.

6.6.9.4 Encased I-Sections

Encased I-sections are partially or completely encased in the concrete deck.

If no sign of cracking, rust, or separation along the steel-concrete interface is evident, the encased I-section may be assumed to act as a composite section at the service- and fatigue-limit states. The encased I-section may only be considered composite at the strength limit state if sufficient shear transfer between the steel I-section and the concrete can be verified by calculation.

verify that there is no separation, due to corrosion of the top flange or other causes.

C6.6.9.4

Encased I-sections constructed without shear connectors may act compositely with the concrete deck due to the bond and friction between the concrete and steel. The degree of composite action varies depending upon the magnitude of loading, degree of encasement of beam flanges, and physical condition of the interface.

Guidance on evaluating composite action in slab-on-girder bridges without mechanical shear connection can be found in NCHRP Research Results Digest, November 1998—Number 234, *Manual for Bridge Rating Through Load Testing.*

6.6.9.5 Moment-Shear Interaction

The shear resistance of stiffened interior web panels subject to the simultaneous action of high moment and shear, as at interior supports of continuous spans, shall be reduced by the moment-shear interaction factor R as specified in LRFD Articles 6.10.7.3.3a and 6.10.7.3.3b. This reduction due to flexure-shear interaction is applicable where $M_u > 0.5\varphi_f M_p$ for compact sections or where $f_u > 0.75\varphi_f F_y$ for non-compact sections.

6.6.9.6 Riveted Members

The moment capacity of riveted sections and sections with holes in the tension flange should be limited to M_y.

C6.6.9.6

At sections of flexural members with holes in the tension flange, it has not been fully documented that complete plastification of the cross section can be achieved prior to fracture of the net section of the flange (see LRFD C6.10.4.1.1).

LRFD criteria could be used for older riveted sections if *b/t* ratios are satisfied. The Engineer should check the *b/t* between rivet lines, from the rivet line to the plate edge, and the spacing of the rivets. Net section failure should also be checked. This is dependent upon the yield to tensile ratio of the steel. For riveted compression members, LRFD equations for compressive resistance would be conservative for riveted construction since the riveted members should have much lower residual stresses.

6.6.10 Evaluation for Shear

Shear resistance at the strength limit state is specified in the *AASHTO LRFD Bridge Design Specifications* for I-sections, box-girders, and miscellaneous composite members.

Pin analyses should be performed during the load rating analyses of pin connected bridges because the pins may not necessarily be of equal or greater capacity than the members they adjoin.

The alignment of adjoining members relative to the pin could have a significant effect on the load capacity of the pin as the movement of a member changes the point of application of the member force on the pin. This is especially important on bridges without spacer collars between individual components at a pin. The relative positions of all members that connect to a pin should be ascertained in the field.

The pin size should be measured in the field to ascertain any reduction due to corrosion and wear.

6.6.12.5 Riveted Connections

Riveted connections shall be evaluated as bearing-type connections.

C6.6.12.5

Factored resistance values for rivets are based on AASHTO *Standard Specifications* Article 10.56.1.

6.6.12.5.1 Rivets in Shear

The factored resistance of rivets in shear shall be taken as:

$$\varphi R = \varphi F m A_r \qquad \text{Eq. (6-5)}$$

where:

φF = Factored shear strength of rivet (kips)
m = The number of faying surfaces
A_r = Cross-sectional area of the rivet before driving (sq. in.)

The following values may be used for φF:

Table 6-14 Factored Shear Strength of Rivets φF.

Rivet Type or Year of Construction	φF ksi
Constructed prior to 1936 or of unknown origin	18
Constructed after 1936 but of unknown origin	21
ASTM A 502 Grade I	25
ASTM A 502 Grade II	30

6.6.12.5.2 Rivets in Shear and Tension

Rivets that are required to develop resistance simultaneously to tensile and shear forces resulting from factored loads shall satisfy the following relationship:

$$V_u^2 + 0.56\, T_u^2 \le 0.56\, (\varphi A_r F_u)^2 \qquad \text{Eq. (6-6)}$$

6.7.4.1 Design-Load Rating

Rating factors for the design-load rating shall be based upon the Strength I load combination.

6.7.4.2 Legal load Rating and Permit load Rating

Wood bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

6.7.5 Dynamic Load Allowance

The Dynamic Load Allowance for the evaluation of wood components shall be reduced to 50 percent of the values specified in Articles 6.4.4.3 and 6.4.5.5 for steel and concrete components.

6.7.6 Evaluation of Critical Connections

Critical connections of timber bridges shall be evaluated for shear at the strength limit state.

C6.7.6

External connections of non-redundant members are considered critical connections. Split rings and shear plates may be concealed between wood members. These significantly increase the shear strength of bolted connections. Available records should be consulted to verify their presence. Sometimes a probe may be used to locate them.

6.8 POSTING OF BRIDGES

6.8.1 General

Weight limitations for the posted structure should conform to local regulations or policy, using the guidelines in this Manual. Bridge posting should not be confused with bridge-load rating. Bridge inspection and rating are engineering-related activities, whereas bridge posting is a policy decision made by the Bridge Owner.

Bridges not capable of carrying a minimum gross live-load weight of three tons must be closed. A Bridge Owner may close a structure at any higher posting threshold. When deciding whether to close or post a bridge, the Owner should consider the character of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting.

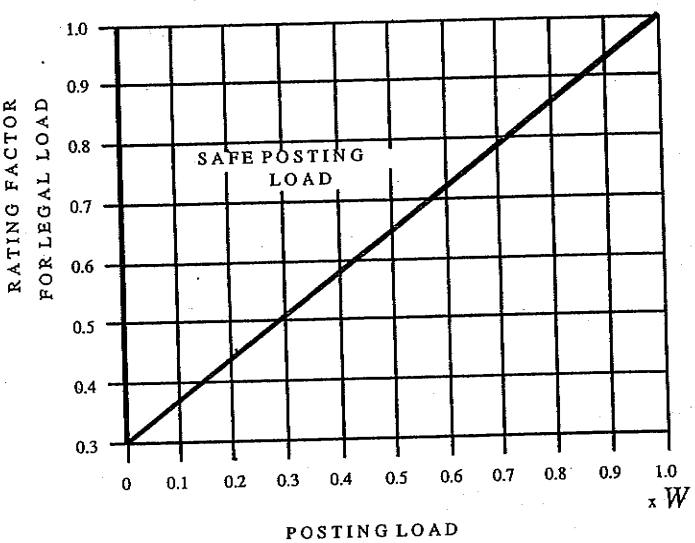
A concrete bridge need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. This general rule also applies to bridges for which details of the reinforcement are not known. The bridge should be inspected at regular intervals to verify satisfactory performance.

C6.8.1

Field experience and tests on reinforced concrete bridges (T-beam and slab bridges) have shown that there is considerable reserve capacity beyond the computed value, and that such spans show considerable distress (e.g., cracking, spalling, deflections, etc.) before severe damage and collapse actually occurs.

combination limits and still cause a load effect in excess of that assumed in the rating factor calculation which uses a standard axle distribution. This acute load distribution on the axles has been incorporated in the posting curve.

The reliability level inherent in the posting curve is raised at the lower posting loads to achieve reliability targets closer to design Inventory levels rather than the evaluation or operating reliability characteristic of other practices in this Manual.



Where: W = Weight Of Vehicle (AASHTO Legal Load)

Fig. 6-2 Calculation of Posting Load.

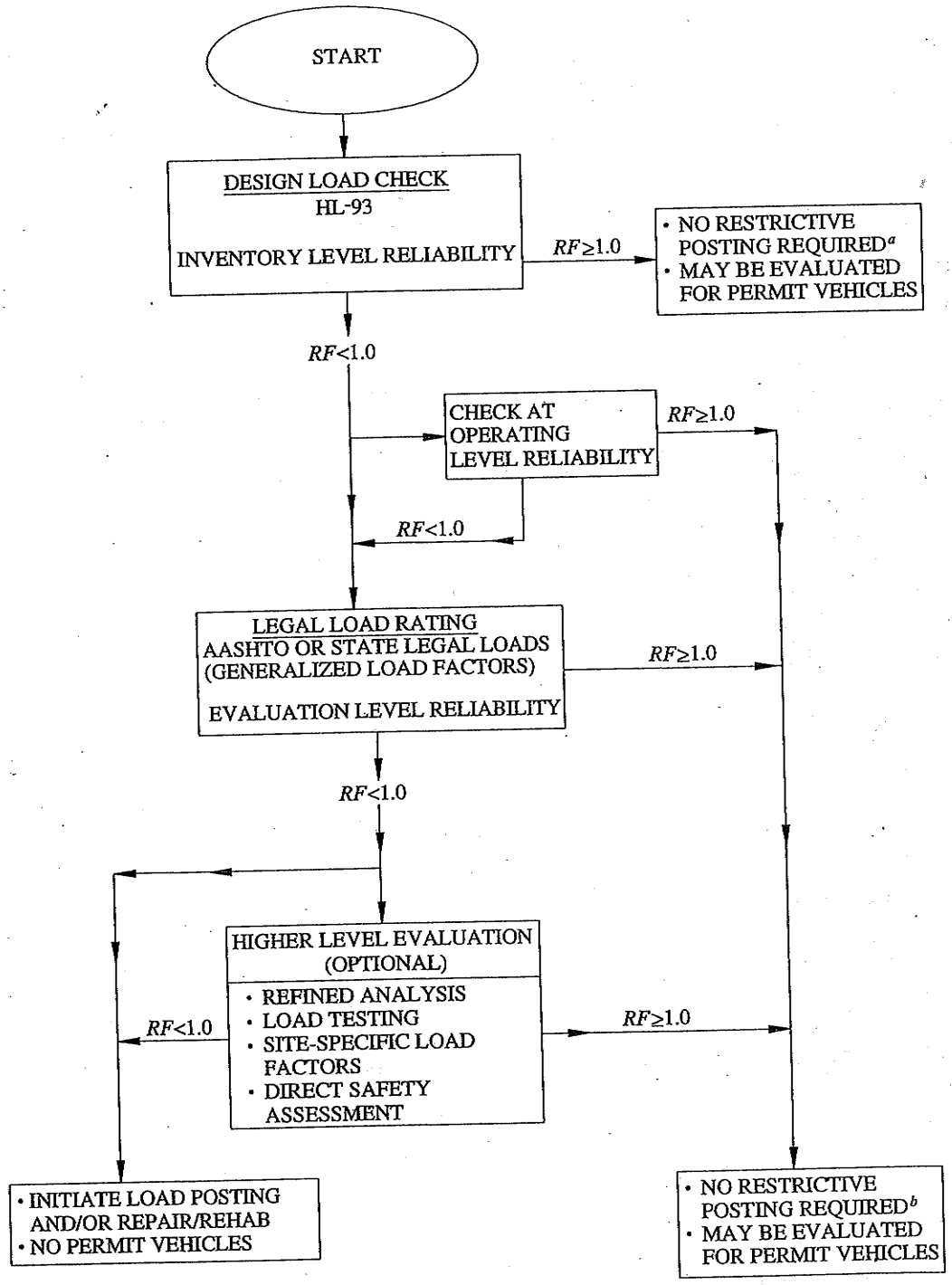
6.8.4 Regulatory Signs

Regulatory signing should conform to the requirements of the *Manual on Uniform Traffic Control Devices* (MUTCD), and should be established in accordance with the requirements of the agency having authority over the highway.

When a decision is made to close a bridge, signs and properly designed, structurally sound traffic barriers should be erected to provide adequate warning and protection to the traveling public. If pedestrian travel across the bridge is also restricted, adequate measures to prevent pedestrian use of the bridge should be installed. Signs and barriers should meet or exceed the requirements of local laws and the applicable sections of the MUTCD. Bridge closure signs and barriers should be inspected periodically to ensure their continued effectiveness.

APPENDIX A.6.1

LOAD AND RESISTANCE FACTOR RATING FLOW CHART



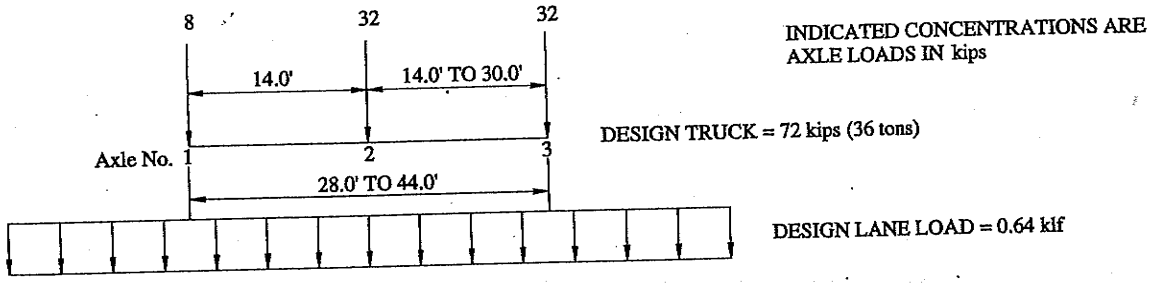
[a]– For AASHTO legal loads and state legal loads within the LRFD exclusion limits.

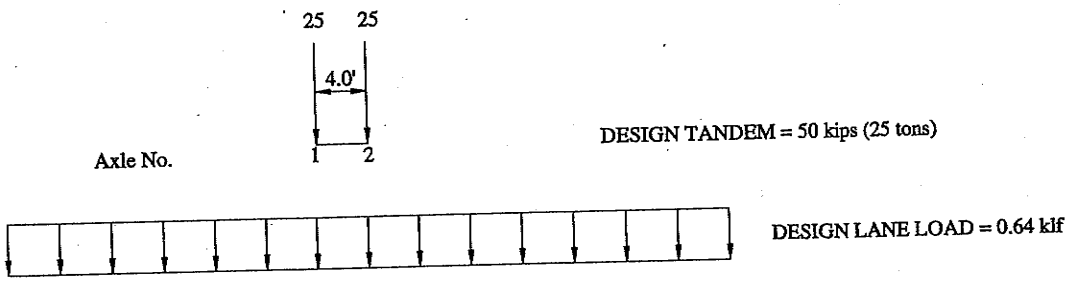
[b]– For AASHTO legal loads and state legal loads having only minor variations from the AASHTO legal loads.

APPENDIX B.6.1

LRFD DESIGN LIVE LOAD (HL-93)

(LRFD 3.6.1)





ADDITIONAL LOAD MODEL FOR NEGATIVE MOMENT AND INTERIOR REACTION
(REDUCE ALL LOADS TO 90%)

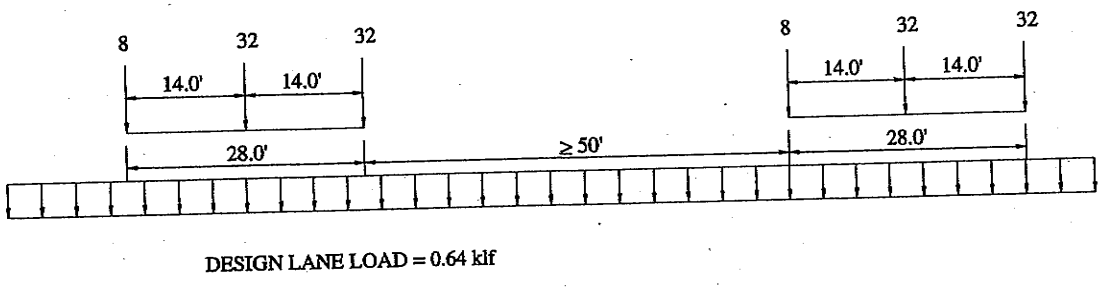
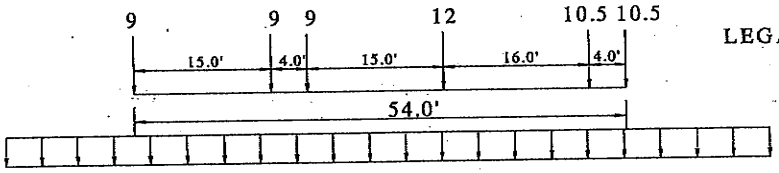


Figure B.6-1 LRFD Design Live Load (HL-93).

b) Lane-Type Legal Load Model—Apply for spans greater than 200 ft. and all load effects.



INDICATED CONCENTRATIONS ARE AXLE LOADS IN kips (75% OF TYPE 3-3)

LEGAL LANE WEIGHT/ft. = 0.2 klf

Figure B.6-5 Lane-Type Loading for Spans GreaterThan 200 ft.

c) Lane-Type Legal Load Model—Apply for negative moment and interior reaction for all span lengths.

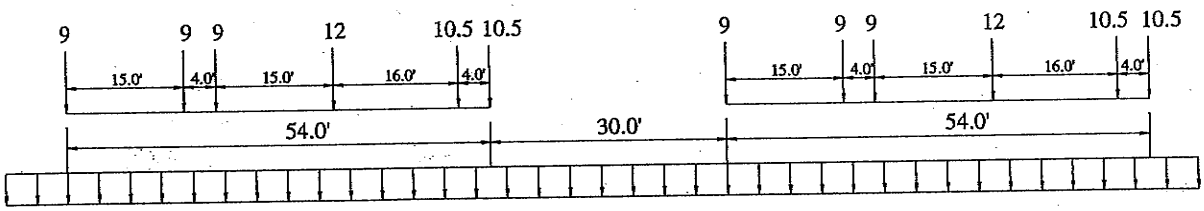
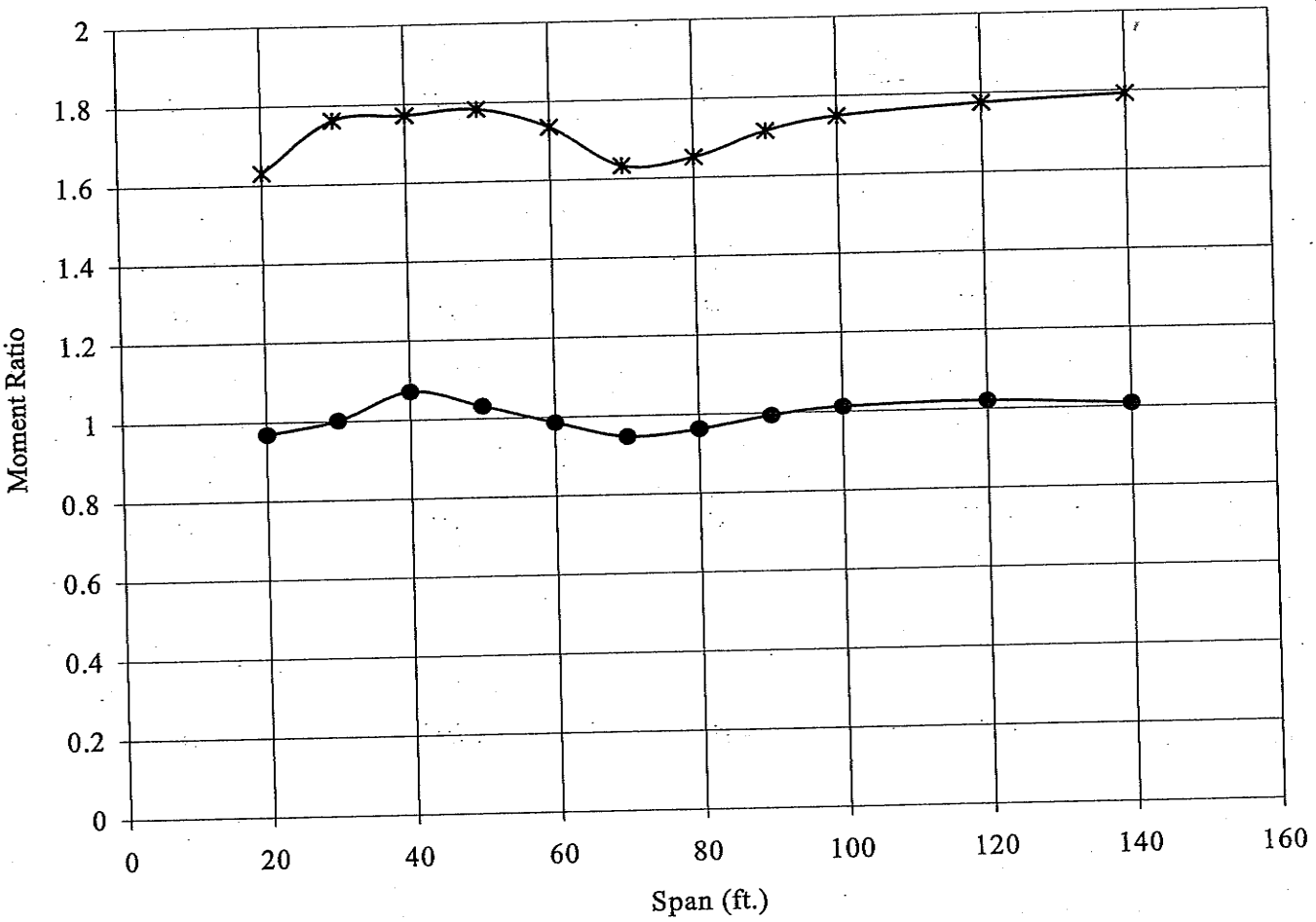


Figure B.6-6 Lane-Type Loading for Negative Moment and Interior Reaction.

APPENDIX B.6.4

VARIATION IN MOMENT RATIO WITH SPAN LENGTH

$$\text{Moment Ratio} = \frac{\text{Simple Span Maximum Moment Caused by the Exclusion Vehicle Population}}{\text{Simple Span Maximum Moment Caused by Each Load Model}}$$



$$\text{—}*\text{—} = \frac{\text{Exclusion Vehicle Population}}{\text{AASHTO Legal Loads}}$$

$$\text{—}\bullet\text{—} = \frac{\text{Exclusion Vehicle Population}}{\text{HL}-93}$$

Figure B.6-7 Variation in Moment Ratio with Span Length.

APPENDIX C.6.1

RATING OF CONCRETE COMPONENTS FOR COMPRESSION PLUS BENDING

STEPS FOR OBTAINING RATING FACTORS (see Fig.C.6-1)

1) Develop the interaction diagram, by computer or manual methods, using as-inspected section properties.

2) Point **A** represents the factored dead load moment and thrust.

3) Using the factored live load moment and thrust for the rating live load, compute the live load eccentricity ($e_1 = M_{LL}/P_{LL}$).

4) Continue from Point **A** with the live load eccentricity to the intersection with the interaction diagram.

5) Read the ultimate moment and axial capacities from the diagram.

6) Moment RF $= \dfrac{\text{Moment Capacity} - \text{Factored } M_{DL}}{\text{Factored } M_{LL+IM}}$

Axial RF $= \dfrac{\text{Axial Capacity} - \text{Factored } P_{DL}}{\text{Factored } P_{LL+IM}}$

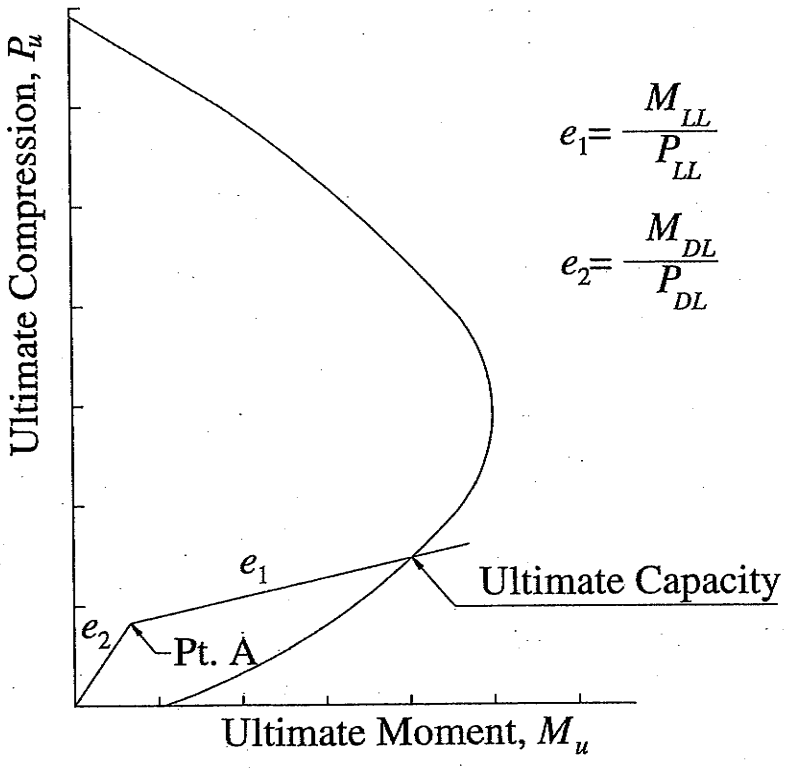


Figure C.6-1 Axial Plus Bending Interaction Diagram for Concrete Structures.

An iterative analysis could be used for improved accuracy.

If, $\frac{P_u}{P_r} < 0.2$

$\frac{P_u}{2P_r} + \frac{M_u}{M_r} \le 1.0$ for rating the correct *RF* will make this an equality

LRFD Eq. (6-15)

$$P_u = \gamma_D P_{DL} + (RF)\gamma_L P_{LL+IM}$$

$$M_u = \delta_b \left[\gamma_D M_{DL} + (RF)\gamma_L M_{LL+IM}\right]$$

$$\frac{\gamma_D P_{DL} + RF\gamma_L P_{LL+IM}}{2P_r} + \frac{\delta_b \left[\gamma_D M_{DL} + RF \times \gamma_L \times M_{LL+IM}\right]}{M_r} = 1.0$$

$$\gamma_D \left[\frac{1}{2}\frac{P_{DL}}{P_r} + \delta_b \frac{M_{DL}}{M_r}\right] + RF \times \gamma_L \left[\frac{P_{LL+IM}}{2P_r} + \delta_b \frac{M_{LL+IM}}{M_r}\right] = 1.0$$

$$RF = \frac{1 - \gamma_D \left[\frac{P_{DL}}{2P_r} + \delta_b \frac{M_{DL}}{M_r}\right]}{\gamma_L \left[\frac{P_{LL+IM}}{2P_r} + \delta_b \frac{M_{LL+IM}}{M_r}\right]}$$

(for $RF = 1.0$)

Where:

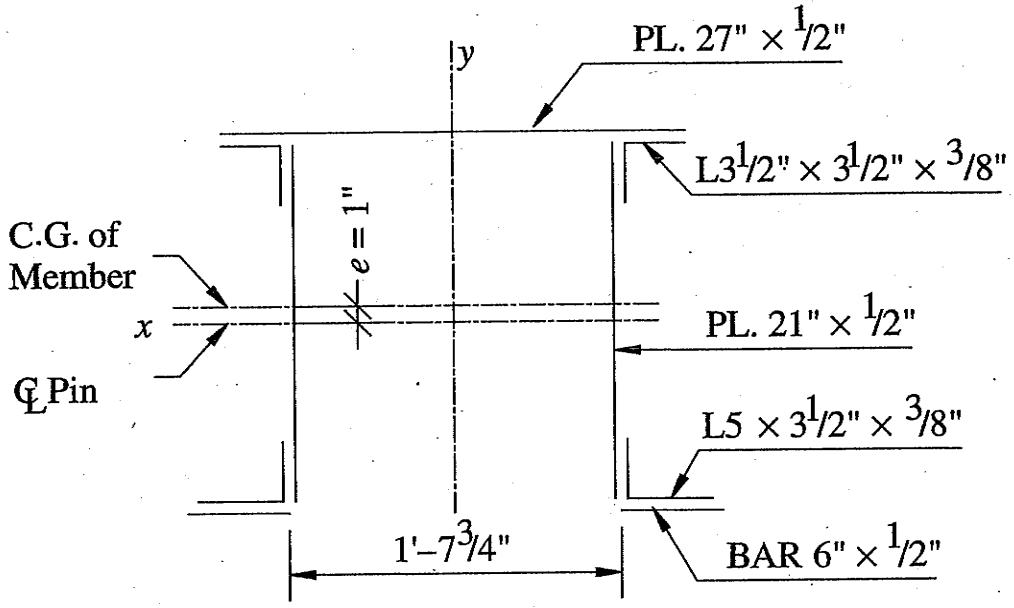
$$\delta_b = \frac{C_m}{1 - \frac{\gamma_D P_{DL} + \gamma_L P_{LL+IM}}{\varphi P_e}}$$

An iterative analysis could be used for improved accuracy.

Example rating using axial load magnification:

Section based upon member in Appendix A Example 6 but with the pins assumed to be 1 in eccentric in the negative y coordinate. Member forces calculated assuming centerline of pin to be concentric with center of gravity of top chord.

e	= 1 in.	A	= 55.3 in.^{2}	S_x *bottom*	= 376.0 in.^{3}
L	= 300 in.	E	= 29000 ksi	I_x	= 5716.8 in.^{4}
P_{DC}	= 558.1 kip	P_{DW}	= 39.4 kip	P_{LL+IM}	= 231.1 kip



$$P_u = 1.75\times231.1 + 1.25\times558.1 + 1.25\times39.4 = 1151.3 \text{ kip}$$

$$\delta_A = \left(1+\frac{1\times55.3}{376.0}\sec\left(\frac{300}{2}\sqrt{\frac{1151.3}{29000\times5716.8}}\right)\right)$$

$$\delta_A = 1.159$$

$$RF = \frac{0.85\times1.0\times0.9\times1906.6 - 1.25\times1.159\times558.1 - 1.25\times1.159\times39.4}{1.75\times1.159\times231.1}$$

$$RF = 1.26$$

(RF = 1.76 for e = 0 in Example 6.)