

LITERATURE REVIEW SUMMARY

Ministry of Natural
Resource Operations

*Development of Standard
Curb Design Parameters*



JANUARY 2011

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Executive Summary

Although the CAN/CSA-S6-06, the Canadian Highway Bridge Design Code (CHBDC) is a primary reference for forest road bridge design, the bridge barrier design and selection requirements are not considered relevant to industrial forest road bridge curb design. Historically, forest road bridge curbs have been considered visual guide rails, providing delineation of the deck edge only. Although there is a long history of successful utilization of timber curbs/guide rails and the recently adopted alternative W-beam and HSS rails, the Ministry of Natural Resource Operations (MNRO) have not established specific guidelines on which to base the design of these elements.

As a result, the **MNRO** retained **Associated Engineering** to assist in the development of reasonable bridge barrier design guidelines, including specified design parameters, for Forest Service Road Bridge Guide Rails.

The first phase of this assignment comprises a literature review of current practices for the design and installation of bridge barriers on low volume roads and forestry road bridges in North America.

We focussed the review on three areas:

- (1) Current regulatory requirements and guidelines.
- (2) Research related to barrier design and selection.
- (3) Standard bridge barriers currently used by various regulatory authorities.

The documents reviewed included:

- Canadian Highway Bridge Design Code including Provincial modifications.
- AASHTO “Load and Resistance Factor Design (LRFD) Bridge Design Specifications” including State modifications.
- AASHTO “Standard Specifications for Highway Bridges”.
- AASHTO documentation related to the design of low volume roads.
- US Forest Service “Transportation Structures Handbook” which governs the design of bridges operated by the US Forest Service.
- Occupational Health and Safety Regulations in several Canadian and American jurisdictions.
- Ontario Ministry of Natural Resources Crown Land Bridge Management Guidelines.
- Research pertaining to bridge barriers on low volume roads including a report from the University of British Columbia (UBC), which summarizes the static testing of bridge barriers currently utilized by the MNRO and an Iowa State University survey on US bridge barrier practices on low volume roads.
- Standard bridge barriers adopted in other jurisdictions that may be applicable on MNRO roads.

Upon review of the documents, we are able to draw the following conclusions:

- .1 All jurisdictions, with the exception of the MNRO and Ministry of Natural Resources in Ontario, typically require the use of crash-tested bridge barriers.
- .2 Occupational Health and Safety Regulatory requirements in British Columbia and the Western United States mandate, as a minimum, the installation of a timber curb on single lane forestry bridges.
- .3 No jurisdictions specifically address the containment of heavy industrial traffic.
- .4 The US Forest Service, BC Ministry of Transportation and Infrastructure and Ontario Ministry of Transportation provide warrants for the use of crash tested TL-1 bridge barriers. These warrants vary in requirements but typically include a maximum allowable AADT and a maximum height above water.
- .5 Bridges with higher traffic volumes or where the bridge deck is higher than 5 m above water typically require the installation of TL-2 (CHBDC PL-1) crash-tested barriers.
- .6 Most jurisdictions require the installation of pre-approved standard barriers.

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1 Introduction

Although the CAN/CSA-S6-06, Canadian Highway Bridge Design Code (CHBDC) is a primary reference for forest road bridge design, the bridge barrier design requirements are not considered relevant to industrial forest road bridge curb design. Historically, forest road bridge curbs have been considered visual guide rails, providing delineation of the deck edge only. Although there is a long history of successful utilization of timber curbs/guide rails and the recently adopted alternative W-beam and HSS rails (**Figure 1(A) and (B)**), the Ministry of Natural Resource Operations (MNRO) have not established specific guidelines on which to base the design of these elements.

As a result, the **MNRO** retained **Associated Engineering** to assist in the development of reasonable bridge barrier design guidelines, including specified design parameters, for Forest Service Road bridge guide rails.



(A) Timber Curb



(B) W-Beam Rail

Figure 1
Typical Forestry Bridge Barriers in British Columbia

This project aims to address this question by providing the MNRO with assistance in developing standard bridge barrier design parameters for forest road applications. To facilitate the completion of the project, we have divided the project into the following three phases:

- Phase 1:** Literature Review.
- Phase 2:** Develop a conceptual approach to designing bridge barriers and evaluating bridge barrier designs.
- Phase 3:** Develop design parameters that the MNRO can adopt to design and evaluate bridge barrier systems.

Upon completion of this assignment, we anticipate that the MNRO will have guidelines and design parameters to facilitate the evaluation of existing bridge barriers and the design of new barriers on forest road bridges.

This report summarizes the first phase of the assignment; a literature review of current practices for the design and installation of bridge barriers on low volume roads and forestry road bridges in North America. To facilitate the literature review, we focused on three categories of documents:

- .1 **Current Regulatory Requirements:** This includes the regulatory framework developed by owners to guide the design and installation of bridge barriers.
- .2 **Research:** This includes investigative work covering the design of bridge barriers on low volume road or forest road bridges. We also reviewed a number of “bridge barrier guidelines” and previously completed literature reviews focused on bridge barrier design.
- .3 **Standard Bridge Barriers:** This includes typical bridge barriers recommended by various regulatory bodies for installation on low-volume road or forest road bridges.

As this project focuses on the design of bridge barriers for bridges located on low volume industrial roads, we have focused our efforts on documents governing or related to this specific situation. Since the majority of regulatory requirements refer owners and designers to the governing bridge design codes, we have also reviewed the requirements of the Canadian Highway Bridge Design Code, AASHTO LRFD and AASHTO 17th Edition. In summary, we reviewed the following regulatory documents:

- 2006 Canadian Highway Bridge Design Code.
- Alberta, British Columbia and Ontario supplements/modifications to the Canadian Highway Bridge Design Code.
- Ontario Ministry of Natural Resources Crown Land Bridge Management Guidelines.
- National Cooperative Highway Research Program (NCHRP) Report 230 and Report 350.
- 1994 – 2007 AASHTO LRFD Bridge Design Specifications.
- 1992 – 2002 AASHTO Specifications for Highway Bridges.
- 1989 AASHTO Guide Specifications for Bridge Railings.
- 2001 AASHTO Guidelines for Geometric Design of Very Low-Volume Roads (ADT < 400).
- 2006 AASHTO Roadside Design Guide.
- 2009 AASHTO Manual for Assessing Safety Hardware.
- 2005 USDA – US Forest Service Transportation Structures Handbook.
- Various US State bridge design guides and supplements to national standards.

Prior to presenting a review of the current regulatory practices, we have presented a brief summary of the historical development of bridge barrier design to provide a framework for the interpretation of the regulations.

We were unable to source a significant body of research related to the design and/or selection of bridge barriers on low volume roads. The documents we did source included:

- Recent research and testing completed at the University of British Columbia for the MNRO.
- Iowa State University Institute for Transportation survey on bridge barrier practices on low volume roads.
- Research related to weak post bridge barriers.
- Unpublished research paper by the Ontario Ministry of Transportation on bridge barriers.

In addition, we also reviewed a number of standard bridge barriers that may be applicable on MNRO roads.

To help understand the evolution of bridge barrier design and the development of the associated regulatory regime and accompanying literature, the following presents a very brief historical overview of the development of bridge barrier design in Canada and the United States. We address individual items in more detail in the subsequent sections of the report.

- **Pre-1980:** Extensive full-scale crash testing of in-service bridge barriers that had been designed using traditional static load methods of the day reveals poor performance relating to strength and geometry.
- **Pre-1981:** Bridge barriers designed using static design forces specified in AASHTO Standard Specifications for Highway Bridges or the Canadian Highway Bridge Design Code.
- **1981 – NHCRP Report 230:** "Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances". This document incorporated new procedures, updated the evaluation criteria, and brought the procedures up to date with available technology and practices. This document served as the primary reference for full-scale crash testing of highway safety appurtenances in the United States.
- **1986:** Federal Highway Administration mandates that all barriers incorporated into Federal-aid projects be crash-tested in accordance with NHCRP Report 230.
- **1989:** AASHTO adopts requirements for full-scale crash testing in the 1989 AASHTO Guide Specification for Bridge Railings. This specification mandated three barrier performance levels (PL-1, PL-2 and PL-3) that were consistent with NCHRP 230.
- **1991:** Ministry of Transport in Ontario adopts full-scale crash testing of bridge barriers in accordance with AASHTO and NCHRP 230.
- **1993:** NHCRP 350 "Recommended Procedures for the Safety Performance Evaluation of Highway Features". This document is an update to the previously released NCHRP 230 report and provides for a wider range of test procedures and vehicles, including increasing the number of performance levels from three to six.
- **1994:** AASHTO LRFD Bridge Design Specifications adopts a modified version of the 1989 AASHTO Guide Specification for Bridge Railings.

- **1998:** AASHTO LRFD Bridge Design Specifications (2nd Edition) eliminates the barrier selection tables referenced in the 1989 AASHTO Guide Specification for Bridge Railings, but maintains the concept of multiple performance levels and recommends that owners develop warrants for each bridge site and select a railing that best satisfies the concerns of the warrant. In addition, the 1998 LRFD replaced the three performance levels with six performance levels. These six levels are consistent with the new testing procedures and criteria specified in NCHRP 350.
- **2000:** The Canadian Highway Bridge Design Code adopts the three barriers performance levels and crash-testing requirements included in the 1989 AASHTO Guide Specification for Bridge Railings. The most recent updates of the CHBDC (2006 and 2010) have not revised these criteria although we anticipate that the 2012 CHBDC will incorporate six performance levels used in NCHRP 350.
- **2009:** AASHTO publishes the “Manual for Analysis of Safety Hardware (MASH)” to replace NCHRP 350. This document further refines the crash-testing criteria and evaluation.

2 Review of Canadian Regulatory Requirements

The following provides a MNRO and summary of the bridge barrier requirements mandated by the Canadian Highway Bridge Design Code, MNRO, Ontario Ministry of Transportation, Ontario Ministry of Natural Resources, Alberta Transportation, WorkSafeBC, and the Ontario Ministry of Labour. With the exceptions of these authorities, we found no literature from Canadian Provinces and Territories regarding bridge barriers on low-volume and forestry/industrial roads.

2.1 2006 CANADIAN HIGHWAY BRIDGE DESIGN CODE

The Canadian Highway Bridge Design Code (CHBDC) CAN/CSA-S6-06 offers a prescriptive approach to railing design. It recognizes three barrier ‘performance levels’, with PL-1 being the lowest performance level, and PL-3 being the highest. The code also includes a rationale for alternative performance levels when approved by the regulatory authority. CHBDC bases the selection of a performance level on a “Barrier Exposure Index” that accounts for Annual Average Daily Traffic (AADT) and factors related to bridge width, curvature, grade, and height. Based on the Barrier Exposure Index, design speed, percentage of truck traffic and barrier clearance, CHBDC provides guidance on the selection of the most appropriate performance level. Once the designer adopts a performance level, CHBDC requires that the preferred barrier be crash-tested to the 1981 NCHRP Report 230 (see [Section 3.1](#)). CHBDC, however, provides an exception for determining the adequacy of a barrier by evaluating the performance of a similar barrier when struck by vehicles.

The CHBDC Commentary offers several crash-tested barrier details for each performance level. Additionally, several Provincial Ministries of Transportation publish crash-tested barrier details on their respective websites and typically require that barriers conform to these standards. Though crash testing is specified for all barriers, the CHBDC presents design forces to facilitate the design of barrier-to-deck connections. [Table 2-1](#) summarizes the CHBDC design forces.

**Table 2-1
Summary of Barrier Design forces in the 2006 CHBDC
(factored loads in parentheses)**

Design Forces	Barrier Performance Level		
	PL-1	PL-2	PL-3
Transverse Load, kN	50 (85)	100 (170)	210 (357)
Longitudinal Load, kN	20 (34)	30 (51)	70 (119)
Vertical Load (kN)	10 (17)	30 (51)	90 (153)

Table 2-2 summarizes the Barrier Exposure Index calculations for the following three typical scenarios that the MNRO may encounter:

- **Scenario 1:** Typical single lane bridge on horizontal tangent with vertical grade less than 2% and overall height above water/ground less than 8.0 m.
- **Scenario 2:** Single lane bridge on a horizontal curve (radius less than 300 m) with a vertical grade greater than 6% and height above water/ground greater than 5.0 m, i.e., short bridge on a curve.
- **Scenario 3:** High-level bridge on horizontal tangent with vertical grade less than 4% and overall height above water/ground greater less than 24.0 m.

Table 2-2
Sample Calculations of Barrier Exposure Index for
Single Lane Bridge on Forest Road

	Typical ¹	Steep Vertical Grade and Horizontal Curve ²	High-level Crossing ³
AADT ⁵	400	400	400
K _h	2.00	2.00	2.00
K _c	1.00	4.00	1.00
K _g	1.00	2.00	1.50
K _s	1.00	0.70	2.85
Be⁴	0.8	4.48	3.42
Barrier Clearance	<2.5 m		
Design Speed	50 km/hr		
Percentage of Trucks	40%		
Performance Level	PL-1	PL-1	PL-1

Note:

1. Typical bridge on horizontal tangent with vertical grade less than 2% and overall height above water/ground less than 8.0 m.
2. Bridge on a horizontal curve (radius less than 300 m) with a vertical grade greater than 6% and height above water/ground greater than 5.0 m, i.e., short bridge on a curve.
3. High-level bridge on horizontal tangent with vertical grade less than 4% and overall height above water/ground greater less than 24.0 m.
4. If $7.4 < Be < 40.0$, PL-2 level barrier required.
5. Typical limit for low volume roads.

A review of the Barrier Exposure Index for a typical single lane bridge located on a forestry road where the road alignment is on a tangent with a limited vertical grade suggests that the PL-1 performance level is appropriate. However, the CHBDC methodology might result in a PL-2 performance level barrier for bridges located on both horizontal and vertical curves with significant clearance above ground/water.

BC Ministry of Natural Resource Operations

The BC MNRO “Forest Service Bridge Design and Construction Manual” and “Interim MFR Bridge Design Guidelines” provide guidance on barrier design for forestry roads. The interim guidelines state that bridge design must conform to the 2006 CHBDC, modified to suit forestry bridges. The manual states that bridge rails must conform to one of three standard design options: Timber Curb, W-Beam, or HSS Beam systems, all of which are shown in **Figure 2** below.

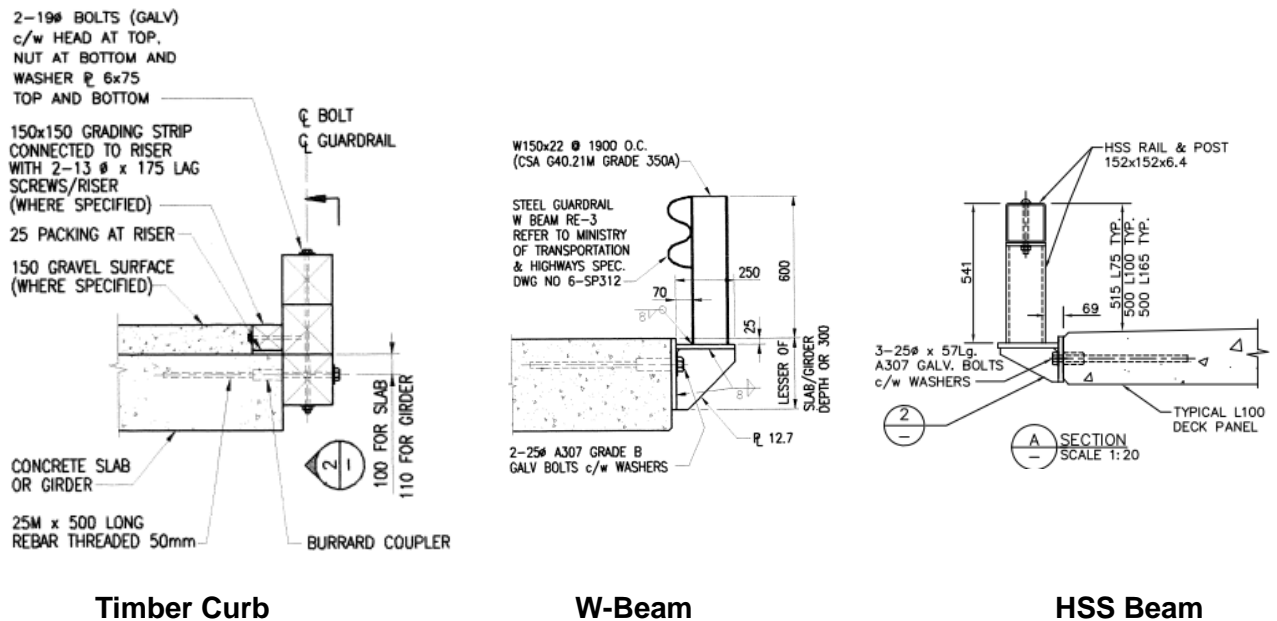


Figure 2
Standard Bridge Rails Prescribed by the MNRO

Although each of these systems appear to be performing successfully, they have not been crash-tested to NCHRP or MASH standards, nor do they appear to meet the design and selection requirements of either the CHBDC or AASHTO LRFD.

2.2 ONTARIO MINISTRY OF TRANSPORTATION

The Ontario Ministry of Transport (MTO) offers some guidance on the use of barriers with performance levels less than that mandated by the CHBDC for low-volume, low-speed, and low-hazard bridges. In “Exceptions to the Canadian Highway Bridge Design Code CAN/CSA-S6-00” The MTO defines the following two levels of barriers, LVPL1 and LVPL2 for consideration on low volume roads:

- **LVPL1:** Railing must be able to successfully redirect a pick-up truck with a speed of 25 km/hr and an impact angle of 15°. The level of performance is less than that for LVPL2 and is intended for very low volume and low speed traffic.
- **LVPL2:** Railing must satisfy Test Level 1 of NCHRP Report 350 (successfully redirect a pick-up truck with a speed of 50 km/hr and an impact angle of 25°). The level of performance is better than that for LVPL1 but less than that for PL-1 from CHBDC and is intended for low volume traffic at moderate speeds.

Table 2-3 summarizes the MTO Barrier Selection criteria for LVPL1 and LVPL2 barriers. Sample barrier details are included in **Appendix B**.

Table 2-3
MTO Barrier Selection Criteria

Barrier	ADT	Deck Height ¹	Operating Speed	Bridge Width
LVPL2 (TL-1)	≤400	≤5.0m	≤50 km/hr	No limit
			≤80 km/hr	≤5.0m
LVPL1	≤100	≤2.5m	≤25 km/hr	No limit
			≤40 km/hr	≤5.0m

Note:

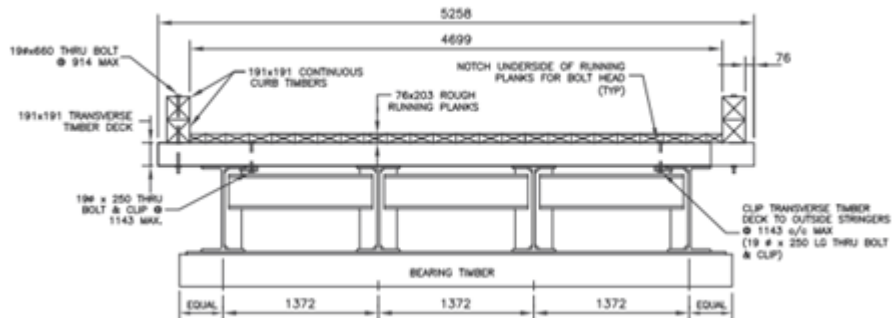
¹ Measured to top of water

2.3 ONTARIO MINISTRY OF NATURAL RESOURCES

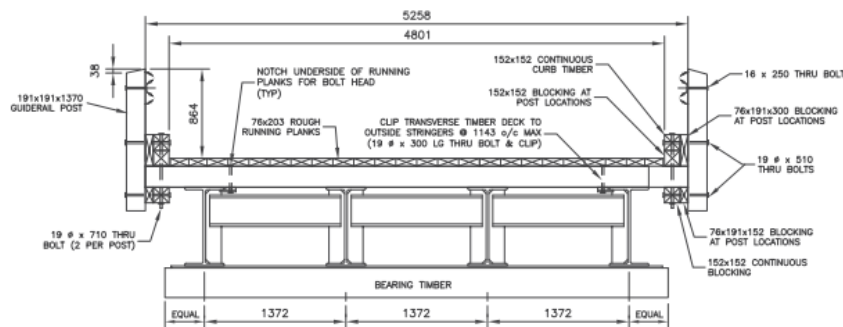
The Ontario Ministry of Natural Resources (MNR) provides guidance on the use of bridge barriers for forestry and resource roads in “Crown Land Bridge Management Guidelines”. In this document, the MNR recognizes that while the CHDBC is an up-to-date technical reference, it is not intended to apply to bridges on low volume roads. As such, the document states that MTO’s “Guidelines for the Design of Bridges on Low Volume Roads” provides exceptions and modified criteria that may be more applicable to resource road bridges.

These exceptions include allowing a narrower deck width, and mandating that “curb and railing systems are designed to provide deck width delineation only”. The document also states that “Curbs and railings need not be designed to withstand live loads specified in the Bridge Code. They are intended to mark the edge of the bridge deck and need not be designed to deflect an impacting vehicle”.

The MNR requires that if timber curbs are used, they should be at least 28 cm (11”) above the travel surface. It recommends that for bridges with significant hazards, such as height above water or poor alignment/visibility, guardrails should be used, in accordance with MNR standard drawings. Standard MNR timber curb and metal rails are shown in **Figure 3** below, and the Full drawings can be seen in **Appendix F**.



Timber Curb



Metal Rail

Figure 3
Standard Bridge Rails Prescribed by the Ontario MNR

Upon discussion with an MNR Engineer, they clarified that timber curbs are used for the majority of bridges, though railings are used in cases of bridges with high pedestrian traffic or appreciable height above water. The MNR also stated that regardless of what type of barrier system is used, it does not specify a Performance or Test Level, as it does not want to guarantee a particular level of vehicle containment.

2.4 BRITISH COLUMBIA MINISTRY OF TRANSPORT AND INFRASTRUCTURE

The British Columbia Ministry of Transport and Infrastructure (MoTI) provides some guidance on the use of bridge barriers for low volume roads in the “Low Volume Road Bridge Design Guidelines”. In this document, the MoTI accepts the use of a lower performance barrier than that mandated by the CHBDC. Specifically, the MoTI accepts barriers crash-tested to NCHRP 350 Test Level 1 when:

- ADT \leq 50.
- Deck height above the channel bottom \leq 4.0 m.
- Operating speed \leq 50 km/hr.
- Bridge width $<$ 8.5 m.

2.5 ALBERTA TRANSPORTATION

The Alberta Ministry of Transportation “Bridge Structures Design Criteria” has an extensive list of standard barriers, the most applicable of which is a PL-1 ‘Low Height Thrie Beam Bridge Rail’ for low volume bridges with a width less than nine metres.

2.6 WORKSAFE BC

Although WorkSafeBC does not provide guidance on bridge barrier design, Part 26 of the Occupational Health and Safety (OHS) Guidelines, ‘Forestry Operations and Similar Activities’, requires that open sides of bridges used by logging trucks be equipped with timber or log curbs or bull rails of sufficient height to prevent vehicles from running off the structure, but not less than 10 inches (250 mm). The definition of ‘vehicles’ is not explicitly stated. The full passage from the OHS guidelines is included in [Appendix C](#).

2.7 ONTARIO MINISTRY OF LABOUR

The Ontario Ministry of Labour’s Occupational Health and Safety Act requires that bridges on haul roads “have curbs of a height of not less than 15 cm (6”) on each side of the travelled portion of the bridge”. Unlike WorkSafe BC’s guidelines, the Ontario act makes no mention of vehicle containment.

3

Review of United States Regulatory Requirements

3.1 NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM – REPORTS

The National Cooperative Highway Research Program (NCHRP) conducts research in problem areas that affect highway planning, design, construction, operation, and maintenance nationwide. This Federally funded program, administered by the Transportation Research Board, has produced a number of reports over the last forty years that have guided the design and testing of bridge barriers. The two most commonly referenced documents are:

NCHRP 230 Report - Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances: This Report presents procedures for conducting crash tests and in-service evaluation of roadside appurtenances including longitudinal barriers such as bridge barriers, guardrails, median barriers, transitions and terminals and other features that may occur within or alongside a roadway. The Report does not refer to Performance Levels or provide guidance on performance criteria to facilitate barrier selection; rather it ascribes this task to policy makers.

NCHRP 350 Report - Recommended Procedures for the Safety Performance Evaluation of Highway Features: This Report supersedes the NCHRP 230 Report and incorporated numerous major revisions including:

- Changes to the test vehicle.
- Changes to the number of impact conditions of the test matrices.
- Adoption of the concept of “test levels” as opposed to “service levels”.
- Changes to evaluation criteria.

AASHTO has recently released the ‘Manual for Assessing Safety Hardware’ (MASH) which will supersede the NCHRP Report 350. We have included a review of MASH (2009) in Section 4.2.2.

3.2 AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO)

3.2.1 AASHTO 1989 Guide Specifications for Bridge Rail Design

The American Association of State Highway and Transportation Officials (AASHTO) has produced several documents and standards pertaining to bridge railing design, spanning several decades, and design philosophies. The most recent, the 1989 “Guide Specifications for Bridge Rail Design”, calls for three performance levels, based on NCHRP 230 crash-test requirements. The performance level is selected from a table based on design speed, percentage truck traffic, number of lanes, and AADT. This document guided bridge railing design in the United States until 1994 and is the basis for the CHBDC barrier design philosophy.

3.2.2 Manual for Assessing Safety Hardware, First Edition

The AASHTO “Manual for Assessing Safety Hardware” (MASH) presents uniform guidelines for crash testing permanent and temporary highway safety features and recommends evaluation criteria to assess test results. MASH is an update to, and supersedes NCHRP Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features, for the purposes of evaluating new safety hardware devices. MASH does not supersede any guidelines for the design of roadside safety hardware, which are contained within the AASHTO Roadside Design Guide. Further, current recommended regulatory policy in the United States suggests all highway safety hardware accepted prior to the adoption of MASH – using criteria contained in NCHRP Report 350 – are considered acceptable and retesting is not required. However, new highway safety hardware not previously evaluated must utilize MASH for testing and evaluation.

MASH has increased the weight of all test vehicles by 13%, compared to NCHRP Report 350. In addition, MASH increased the impact angle for passenger cars from 20° to 25° for all test levels. The manual has not altered the 25° impact angle for pickup trucks or the 15° angle for single-unit trucks, tractor-van trailers, and tractor-tank trailers. It has been suggested that a 25° impact angle is unrealistic on a single lane bridge and as such, the US Forest Service is questioning the relevancy of MASH for the testing of TL-1 and TL-2 barriers for forestry roads. A summary of crash test requirements for TL-1 and TL-2 barriers is provided in **Table 3-1** below.

Table 3-1
MASH Crash Test Criteria for TL-1 and TL-2 Barriers

Test Level	Vehicle	Vehicle Mass, lb. (kg)	Impact Speed, mph (km/h)	Impact Angle, Degrees
TL-1	Intermediate Car	3300 (1500)	31 (50)	0 – 25
	Pickup Truck	5000 (2270)	31 (50)	0 – 25
TL-2	Intermediate Car	3300 (1500)	44 (70)	0 – 25
	Pickup Truck	5000 (2270)	44 (70)	0 – 25

3.2.3 Standard Specifications for Highway Bridges 17th Edition (2002)

The 2002 “Standard Specifications for Highway Bridges” presents the Allowable Stress and Load Factor Design methodologies for bridges. Since the Federal Highway Administration and States have established the goal that all new bridges conform to the Load and Resistance Factor Design (LRFD) by 2007, this Standard is no longer being updated. Notwithstanding, numerous authorities still accept and refer to this Standard.

This Standard requires the design of bridge barriers using a traditional static load method and specifies the magnitude and location of the transverse, horizontal and vertical static loads. The Standard specifies an un-factored transverse load of 45 kN (10 kip) and a factored load of 127 kN

Notwithstanding the static force design methodology included in this Code, the Federal Highway Administration has mandated the use of crash-tested bridge barriers since 1986 as described in [Section 2](#).

3.2.4 AASHTO LRFD Bridge Design Specifications

After several decades and numerous revisions to the “Standard Specifications for Highway Bridge Design”, AASHTO replaced the Allowable Stress Design and Load Factor Design methodology with a Load and Resistance Factored Design (LRFD) philosophy in 1994. With the publication of a new LRFD Bridge Design Specifications, AASHTO adopted the bridge barrier design methodology detailed in the 1989 AASHTO Guide Specifications for Bridge Rail Design including the mandated three performance levels and associated crash testing requirements. It also referenced the AASHTO Roadside Design Guide, and through the commentary refers readers to the NCHRP 350 Report, “Recommended Procedures for the Safety Performance Evaluation of Highway Features”.

In 1998, AASHTO released the first revision to the LRFD Bridge Design Specifications. Within this revision, AASHTO:

- Increased the number of barrier types to six barrier ‘Test Levels’, TL-1 through TL-6 to conform to NCHRP 350 Report requirements.
- Eliminated barrier selection based on AADT and tabulated modification factors and incorporated a methodology that requires Owners develop warrants for bridge sites and the chosen railing satisfy the concerns of the warrants as completely as possible and practical. A warrant would provide guidance for evaluating the potential safety and operational benefits of traffic control features and typically convey concerns over potential traffic hazards rather than mandating requirements.

Notwithstanding the deletion of barrier selection criteria, [Table 3-2](#) summarizes the AASHTO subjective descriptors for each test level.

Table 3-2
Summary of Subjective Selection Criteria for
various Barrier Test Levels

Test Level	Intended Use
TL-1	For work zones with low posted speeds and very low volume, low-speed local streets.
TL-2	For work zones with moderate speeds or with small number of heavy vehicles at reduced speeds and most local and collector roads with favourable site conditions
TL-3	For a wide range of high-speed arterial highways with low mixtures of heavy vehicles and with favourable site conditions.
TL-4	For the majority of applications on high-speed highways, freeways and expressways with a mixture of trucks and heavy vehicles
TL-5	For the same site conditions as TL-4 when the site conditions justify a higher level of rail resistance and for applications on freeways with high-speed, high-traffic volumes and where trucks make up a significant portion of the traffic or when unfavourable site conditions exist.
TL-6	For applications on freeways with high-speed, high-traffic volumes and a higher ratio of heavy vehicles and a highway with unfavourable site conditions.

Though AASHTO LRFD states that all barriers and barrier-deck connections must be crash-tested, it provides design loads and loading configurations to facilitate the preliminary design of test specimen barriers. **Table 3-3** summarizes the AASHTO LRFD barrier design forces.

Table 3-3
Summary of Barrier Design Forces in AASHTO LRFD

Design Forces	Railing Test Level					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
Transverse Load, kN	60	120	240	240	550	780
Longitudinal Load, kN	20	40	80	80	183	260
Vertical Load, kN	20	20	20	80	355	355

Note: These design forces also represent factored loads as the live load factor = 1.0 for the specified load combination.

The recently released 5th Edition of the AASHTO LRFD Bridge Design Specifications remains very similar to the 2nd Edition with respect to the design of bridge barriers.

3.2.4.1 Comparison of AASHTO LRFD Bridge Design Specifications and Canadian Highway Bridge Design Code Barrier Design Forces

Although CHBDC only includes three barrier performance levels, these performance levels correlate with the Test Levels described in AASHTO LRFD Bridge Design Specifications.

Table 3-4 compares the barrier loads mandated by the two Codes. As discussed in the previous Section, the barrier loads provided in AASHTO LRFD Bridge Design Specifications are intended to facilitate the design of test-specimen barriers and their connection to the bridge deck, prior to full-scale crash-testing.

**Table 3-4
Comparison of AASHTO LRFD and CHBDC
Barrier Design Forces**

Design Forces ¹	AASHTO LRFD Railing Test Level (Modified for Comparison)					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
Transverse Load, kN	35	70	141	141	323	458
Longitudinal Load, kN	12	24	47	47	108	153
Vertical Load, kN	12	12	12	48	209	209
Load Height, mm	685	685	685	810	1070	2290
Design Forces ²	CHBDC Railing Performance Level					
	PL-1	PL-2	PL-3			
Transverse Load, kN	50	100	210			
Longitudinal Load, kN	20	30	70			
Vertical Load, kN	10	30	90			
Load Height, mm	680	800	1050			

Note:

¹ The AASHTO LRFD load factor for barrier design forces is 1.0. To compare these to CHBDC unfactored loads, they must be divided by the CHBDC Live Load Factor of 1.7.

² The CHBDC load factor for barrier connection design forces is 1.7. The values shown are unfactored.

A review of **Table 3-4** suggests that:

- The PL-1 barrier is similar to the TL-2 for all design forces.
- The PL-2 barrier is similar to the TL-4 for all design forces.
- The PL-3 barrier is similar to the TL-5 for lateral and longitudinal design forces.

It should also be noted the AASHTO LRFD design forces are consistently 40% higher than the equivalent CHBDC design forces. The AASHTO LRFD design forces are based on peak forces observed during full-scale crash-testing at each of the test-levels, and are used to design prototype barriers for crash-testing. In comparison, CHBDC specifies that the design forces may be used to design the barriers' connection to the bridge deck. It has been observed that forces imparted on barrier anchorages are typically less than the observed peak forces, and CHBDC has therefore reduced the AASHTO LRFD forces by 40% to account for this observation.

3.2.5 AASHTO Roadside Design Guide

The AASHTO "Roadside Design Guide" presents a synthesis of information and operating practices related to roadside safety. This guide defines the roadside as that area beyond the traveled way (driving lanes) and the shoulder (if any) of the roadway itself, hence the focus of the guide is on safety treatments that minimize the likelihood of serious injuries when a driver runs off the road. When considering bridge barriers, the Guide draws heavily upon the AASHTO LRFD Bridge Design Specifications and the NCHRP 350 Report and provides a summary of best practices. The Guide does not specifically address low-volume roads except to state that the US Forest Services has designed and tested a number of barriers to the TL-1 criteria.

3.2.6 Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)

The AASHTO "Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)" addresses the unique needs of roads with low traffic volumes and reduced frequency of crashes where the traditional design approach use for higher traffic volumes results in less cost effective designs. It is intended that the Guide be used in lieu of the AASHTO "A Policy on Geometric Design of Highways and Streets" and AASHTO "Roadside Design Guide". This Guide highlights the following two unique characteristics of low-volume roads:

- Given the very low traffic volumes, encounters between vehicles that represent opportunities for crashes to occur are rare events and that multiple-vehicle collisions of any kind are extremely rare events.
- The local nature of the road means that most motorists using the road have traveled it before and are familiar with its features, i.e., geometric design features that might surprise an unfamiliar driver will be anticipated by the familiar driver.

Although the Guide focuses on providing geometric design guidelines, it does provide some useful definitions and summarizes a risk-based approach to the design of very low-volume roads. Of specific interest, is the definition of various types of rural low-volume roads including:

- Major access roads.
- Minor access roads.
- Industrial/commercial access roads.
- Agricultural access roads.
- Recreational and scenic roads.
- Resource recovery roads.

Where a rural resource recovery road is defined as "...local roads serving logging or mining operations". Such roads are typically found only in rural areas. Resource recovery roads are distinctly different from the other functional subclasses of very low-volume local roads in that they are used primarily by vehicles involved with the resource recovery activities and the driving population consists or exclusively of professional drivers with large vehicles. In some cases, traffic operations on resource recovery roads are enhanced through radio communication between drivers, enabling such roads to be built and to operate as single-lane roads. Most resource recovery roads are un-paved."

The Guide provides limited guidance with respect to bridges, but does address the design of single lane bridges, suggesting that they are a suitable solution where appropriate. The Guide recommends the adoption of a minimum bridge width of 4.5 m and provides some additional guidance for wider single lane bridges. No guidance is provided on the selection of bridge barriers and readers are referred to other AASHTO and NCHRP publications.

3.3 US DEPARTMENT OF AGRICULTURE – US FOREST SERVICE

The US Forest Service references two documents for the design of bridge barriers, regulatory requirements are defined within the "Forest Service Manual – Chapter 7720 – Transportation System – Development" (FSM) and design guidance is provided in the "Transportation Structures Handbook – Chapter 7 – Road Bridge Design" (TSH). Since the majority of the guidance is contained within the TSH, this is where we have focussed our review. The current version of the TSH was published in 2005; however, the FSM and TSH are currently being revised. We anticipate that the US Forest Service will publish the revised versions during 2011. To assist with our review, Rodney Dell 'Andréa, P.E., a US Forest Service Regional Engineer, based in Alaska, provided us with draft versions of the revised FSM and TSH. The following briefly summarises the existing and proposed requirements for bridge barriers on bridges falling under the jurisdiction of the US Forest Service.

3.3.1 2005 Transportation Structures Handbook

The 2005 TSH references the following AASHTO publications (described in [Section 3](#)):

- AASHTO Roadside Design Guide.
- AASHTO Standard Specifications for Highway Bridges.
- AASHTO LRFD Bridge Design Specifications.
- AASHTO Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT \leq 400).

In addition, the TSH also allows for the replacement of railings with a curb-only system for bridges at low-hazard sites that experience low volumes of low-speed traffic, and when object markers (bridge delineators) provide sufficient advance warning of the bridge. A warrant for a curb-only system is included in [Appendix D](#).

In summary, the approach to bridge barrier design is as follows:

Bridge Widths

Single lane bridges must have a minimum clear width between curbs of 4.3 m. The 2005 TSH allows wider single lane bridges but the designer must ensure that the single-lane bridge does not create the appearance of two traffic lanes.

Double-lane bridges must have a minimum clear width between curbs of 7.3 m. The 2005 TSH allows deviations from these guidelines if the widths are consistent with the intended use.

Bridge Railings

When designing bridge railings, the 2005 TSH mandates safety is the primary criterion for railing system selection. Further, a bridge railing is required to:

- Laterally restrain a collision with the design vehicle and limit deceleration to a tolerable level.
- Smoothly redirect any colliding vehicles.
- Remain intact during a collision.
- Protect vehicle occupants and pedestrians where sidewalks are used.
- Provide delineation of the bridge and increase visibility.
- Be simple to maintain.
- Meet the above requirements, giving emphasis first to safety, and secondly to aesthetics and costs.

Railing Design Loads

The TSH requires that all bridge railings be designed to AASHTO Standard Specifications for Highway Bridges or that the chosen system be fully crash-tested. The TSH does not provide any guidance on the selection of an appropriate Test Level.

For double-lane road bridges, the TSH requires the design of railings to the railing design load cited in AASHTO Standard Specifications for Highway Bridges. On single-lane road bridges, the 2005 TSH allows a 50% reduction in the AASHTO Standard Specifications for Highway Bridges design loads. However, the TSH recommends use of the full railing design load for the following:

- Where the design speed is relatively high and, in combination with poor approach alignment, creates a substantially increased risk of a vehicle colliding with the railing at a large impact angle.
- Where the potential hazard resulting from penetration of the railing system by a vehicle is high, such as those with decks 20' (6.1 m) or more above the stream.
- Where stream depth and/or current velocity are high, creating an extreme hazard to occupants of any vehicle that enters the water.

In addition, the TSH also allows for the replacement of railings with a curb-only system for bridges at low-hazard sites that experience low volumes of low-speed traffic, and when object markers (bridge delineators) provide sufficient advance warning of the bridge. A warrant for a curb-only system is included in [Appendix D](#).

In making this decision, engineers need to consider the following:

- Traffic speed.
- Traffic volume.
- Road alignment and bridge length.
- Bridge width (at least as wide as road travel-way and shoulders).
- Pedestrian traffic.
- Comparing the bridge as a hazard to the other hazards along the road.
- Curb design.

For curb-only systems, the top of the curbs are required to be a minimum of 15" (375 mm) above the travel surface, and for timber curbs, 6"x10" (150 mm x 250 mm) pieces are to be used, as a minimum.

Approach Railings

The TSH also requires the installation of approach railings on all new bridges with bridge barriers. To facilitate the design of the approach railings, the TSH provides guidance on the run-out length, lateral extent of area of concern, tangent length of barrier immediately upstream of area of concern and flare rate. Further, the TSH requires a minimum 11.43 m of approach railings where the design speed exceeds 50 km/hr.

For bridges with curbs and where appropriate bridge delineation is provided, approach barriers are not required.

3.3.2 Proposed Revisions to the Transportation Structures Handbook

The following briefly describes the proposed changes to the 2005 FSH that effect bridge barrier design. The 2011 TSH has removed all references to the AASHTO Standard Specifications for Highway Bridges and now refers to the current edition of AASHTO LRFD Bridge Design Specifications.

As included in the 2005 TSH, the 2011 TSH allows for a lowering of AASHTO bridge design standards as detailed within the TSH and outlined in the AASHTO Guidelines for Geometric Design of Very Low-Volume Local Roads ($ADT \leq 400$).

In summary, the proposed revised approach to bridge barrier design is as follows:

Bridge Railings

When designing bridge railings, the 2011 TSH continues to mandate safety as the primary criterion for railing system selection. The primary purpose of bridge barriers is to contain and redirect vehicles using the bridge. All new bridge barrier systems should be structurally and geometrically crashworthy with consideration given to:

- Protecting the occupants of the vehicle.
- Protecting other vehicles and users near the collision.
- Barrier Cost.
- Appearance and freedom of view.

These are similar requirements to what was previously included in the 2005 TSH.

Railing Design Loads

The 2011 TSH has removed all reference to railing design loads and requires that engineers adopt a railing system that has been shown to meet the desired Test Level through crash testing. The 2011 TSH further defines this crashworthy railing system as “a barrier *system* that has been successfully crash-tested to a currently acceptable test level or can be geometrically and structurally evaluated as equal to a crash-tested system.” This definition provides US Forest Service Engineers with the ability to evaluate systems that may not have been crash-tested but they can show are equivalent to crash-tested systems.

To assist with the selection of the appropriate Test Level, the 2011 TSH provides warrants, as detailed in **Table 3-5**. In developing these warrants, the 2011 TSH references the Test Levels included in the AASHTO LRFD Bridge Design Specifications and the NHCRP 350 Report. Further, the 2011 TSH suggests that the TL-1 barrier applies to most National Forest System road applications. In specifying the use of a crash-tested barrier, the 2011 TSH requires that both the barrier and its connection to the deck be crash-tested. To assist with the design of the deck, reference is made to the railing loads recommended in the AASHTO LRFD Bridge Design Specifications.

**Table 3-5
Minimum Bridge Barrier Requirements on
National Forest Service Road and Land**

Objective Maintenance Level (ML) ¹	Minimum Required Test Level (TL) ²
ML 1 and 2	TL-1
ML 3 Design Speed < 31 mph (50 km/hr)	TL-1
ML 3 Design Speed > 30 mph (50 km/hr)	TL-2
ML 4 and 5	TL-3

Note:

- **ML 1:** Closed and not maintained; only available for administrative use (sometimes considered to be in "storage").
- **ML 2:** Limited use; high clearance vehicles only and/or commercial logging or construction access.
- **ML 3:** All passenger and commercial vehicles; slow speed; single-lane native surface; usually low volume.
- **ML 4:** All passenger and commercial vehicles; moderate speed; single- or double-lane; native or paved surface; volume generally around 100 ADT.
- **ML 5:** All passenger and commercial vehicles; moderate to higher speed; generally double-lane; generally paved surface; access major points of interest in Forest Service and/or adjacent National or State parks and/or part of scenic loop; volumes generally exceeding 100 ADT.

As noted in **Table 3-5**, the Forest Service considers an 18" high timber curb an acceptable TL-1 bridge barrier, regardless of whether or not approach rails are used. In addition, the minimum concrete deck thickness for top mounted curbs is 8" (200 mm) and 12" (300 mm) for side mounted curbs. The minimum height for TL-2 and TL-3 railings is 27" (686 mm).

Approach Railings

The 2011 TSH also requires the installation of approach railings on all new bridges with bridge barriers using a similar design methodology to that stated in the 2005 TSH. Further, for bridges with curbs and where appropriate bridge delineation is provided, approach barriers are not required.

3.4 OCCUPATIONAL SAFETY AND HEALTH REGULATIONS

Similar to considering Occupational Health and Safety Requirements in British Columbia, we considered regulations in Alaska, Oregon and Washington State. The State of Alaska Occupational Safety and Health (AOSH) 'Additional Logging Standards' addresses minimum safety standards specific to logging operations. It requires the installation of shear rails on both outside edges of bridges. The rails are required to withstand the impact of the wheels of a loaded vehicle, and the top of the rails shall not be less than 15" (375 mm) above the bridge surface. The Washington State Department of Labour and Industries requires the same minimum shear rail specifications, while the Oregon Occupational Safety and Health Division requires that rails have a minimum height of 9" (225 mm), and that this height be achieved by using 6" by 6" timbers set on 4" by 6" blocks. No guidance is provided regarding the definition of "loaded vehicle" in these regulations. **Appendix C** includes the full passages from each of the State OSH regulations.

3.5 OTHER REGULATORY FRAMEWORK

Several other regulatory bodies in North America provide varying levels of guidance for low-volume bridge barrier design. The Maine Department of Transportation (DOT) states that if the ADT is less than or equal to half the maximum allowed for the TL-2 system, a bridge barrier may be designed based on AASHTO LRFD requirements for connection design forces TL-2 loads, rather than crash tested. No other regulatory body appears to offer a similar approach.

Several State DOT's have standard bridge railings for low-volume roads, all of which have been crash-tested to NCHRP Report 350 standards. These standard railings generally meet TL-2 requirements. Most DOT's, for which documentation could be found, do not have specific design philosophies for low-volume roads, and follow either AASHTO 'Standard Specifications for Highway Bridge Design' or AASHTO LRFD methodologies for barrier design and selection.

4 Research and Guidelines

We were unable to locate many research documents specific to low-volume bridge barrier design with the exception of actual barrier crash test reports that are included in [Appendix E](#). The Ontario Ministry of Transportation published a report, “Bridge Barriers: Analysis, Performance, and Selection”, comparing the CHBDC and AASHTO LRFD methodologies for highway barrier design. The report recommends the adoption of the six LRFD test levels, in lieu of the current three performance levels. It also recommends a modified methodology for calculating the ‘Barrier Exposure Index’, and the associated test level selection. A review of this revised selection criteria suggests that it would require the use of either TL-1 or TL-2 performance levels depending on percentage of trucks, height above water and whether the bridge lies on horizontal curve. The Ontario Ministry of Transportation has not adopted these revised selection criteria.

The University of British Columbia (UBC) recently published a report titled “Experimental Evaluation of Guard Rail System for Bridges” (Villiard, Khorasani, & Stierner, 2010). The report summarizes the results of the static testing of several MNRO standard bridge barriers, for the purpose of comparing the resulting barrier capacities to specified design loads included in the Canadian Highway Bridge Design Code. It should be noted that static testing does not meet the CHBDC requirements for crash testing.

UBC tested the following barrier systems:

- **System I:** Timber guardrail and risers on timber cross ties.
- **System II:** Timber rails and timber risers on a timber bracket.
- **System III:** Side-mounted all steel system.
- **System IV:** All steel system (post only).

[Table 4-1](#) tabulates the barrier resistances for each system.

**Table 4-1
UBC Barrier System Static Test Results**

System	System Type	Average Capacity (kN)	TL-1 ² Modified to S6-06 (kN)	TL-2 ² Modified to S6-06 (kN)	TL-3 ² Modified to S6-06 (kN)
I (D.Fir)	Timber guardrail and risers on timber cross ties	45	43	86	171
I (Hemlock)		38.5			
II (D.Fir)	Timber rail and risers on steel bracket	38			
III	Side-mounted HSS all steel system	118			
IV (2-25φ anchor bolts)	HSS all steel system (post only)	169			

Note:

¹ Barrier classification assigned by AE by comparing capacities against suggested demands specified by CHBDC and AASHTO LRFD.

² AASHTO LRFD Factored Transverse Barrier design forces are divided by 1.4 to reflect equivalent factored CHBDC forces.

Based on the testing it appears that the existing timber curb system does not meet the requirements for an AASHTO LRFD TL-1 barrier, while the side-mounted steel barrier may conform to the TL-2 requirements if the barrier-deck connection has sufficient capacity. The railing systems II, III and IV were tested using a steel W-flange beam and steel spacer plates for anchorage and do not reflect the actual connection to a precast concrete deck. Therefore, the ultimate capacities and failure modes of each system may not reflect the true behaviour of the same system when anchored to a precast concrete deck.

The Iowa State University Institute for Transportation (ISUIT) published a research report entitled "Bridge Rail and Approach Railing for Low-Volume Roads in Iowa". As part of the report, ISUIT surveyed and gathered literature from various State and County transportation authorities on their respective design methodologies for bridge and approach rails for low-volume roads.

The report states that while many jurisdictions have standard crash-tested railings, most do not have special provisions for low-volume bridges beyond what is offered in AASHTO LRFD or 'Standard Specifications for Highway Bridge Design'.

The report also notes that:

- Most low-volume bridges have bridge barriers that do not meet 'current acceptable standards'.
- Makes recommendations for the incorporation of TL-1, 2 or 3 barriers onto new low-volume bridges, as well as retrofitting existing bridges.
- Proper signage and approach railings are key factors for hazard reduction.

4.1 WEAK POST BARRIER SYSTEMS

As part of the literature review, we reviewed available literature on weak post or 'breakaway' railings. In theory, breakaway barriers have a weak post to deck connection that will, on vehicle impact, break away from the deck, and undergo significant lateral deflection as it contains the errant vehicle. The failure of the post should cause little or no damage to the deck, and be relatively inexpensive to repair or replace, when compared to the cost of concrete deck repairs. These barrier systems are typically designed to low test levels, typically TL-1, though several TL-2 and TL-3 barriers exist. Notwithstanding the theoretical intent that the post break fail without damaging the deck, observations suggest that decks are still subject to significant damage. As these barriers require large deflections to contain vehicles, additional deck is required outside of the barrier, to ensure the vehicles do not partially leave the bridge surface.

In reviewing standard barrier drawings, it appears that most North American jurisdictions have not adopted breakaway barriers as a standard low test-level system.

5 Standard Bridge Barriers

As discussed previously, several State DOTs and Provincial MOTs have developed crash-tested barriers for low-volume bridges. **Appendix F** contains sample drawings of crash-tested or approved bridge rails for use on low volume roads.

A review of the standard barrier drawings indicates the following:

- Most barriers incorporate either thrie-beam or HSS rails. Timber systems were uncommon, and typically used for curb-only systems.
- There are limited crash tested timber deck connections available and less demand for testing as timber decks are being replaced with concrete decks. The USFS has developed several barriers for timber decks, but the majority of standard barriers are designed for concrete decks.
- Top-mounted barriers are more common than side-mounted. Many DOT's have no standard side-mounted systems. Where side-mounted systems are used, the typical minimum concrete deck thickness ranges from 8"-12" (200-300 mm).

6 Summary

Based on our review, we can conclude the following:

- .1 All jurisdictions typically require the use of crash-tested bridge barriers.
- .2 Occupational Health and Safety Regulatory requirements in British Columbia and the Western United States mandate, as a minimum, the installation of a timber curb on single lane forestry bridges.
- .3 No jurisdictions specifically address the containment of heavy industrial traffic.
- .4 The US Forest Service, BC Ministry of Transportation and Infrastructure and Ontario Ministry of Transportation provide warrants for the use of crash tested TL-1 bridge barriers. These warrants vary in requirements but typically include a maximum allowable AADT and a maximum height above water.
- .5 Bridges that see a high volume of traffic or are higher than 5 m above water typically require the installation of TL-2 (CHBDC PL-1) crash-tested barriers.
- .6 Most jurisdictions prefer to use pre-approved standard barrier drawings.

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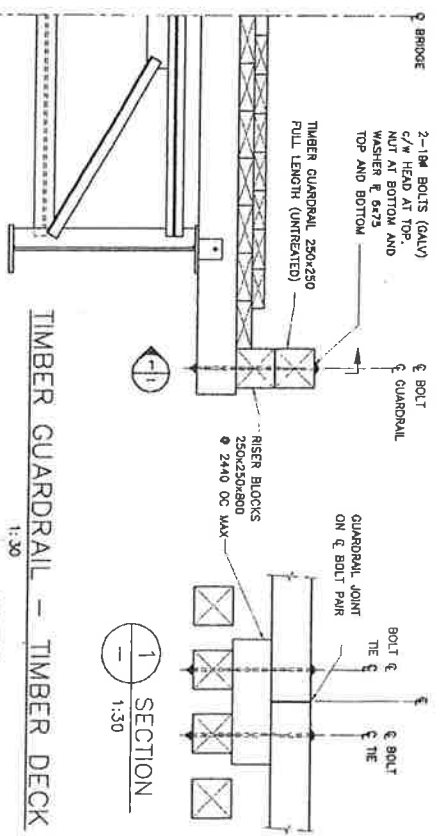
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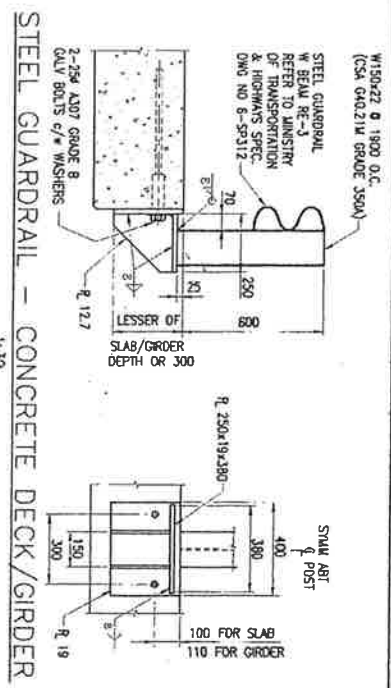
A

Appendix A – BC Ministry of Forests and Range Standard Drawings for Timber Curb, W-beam and HSS Bridge Barriers

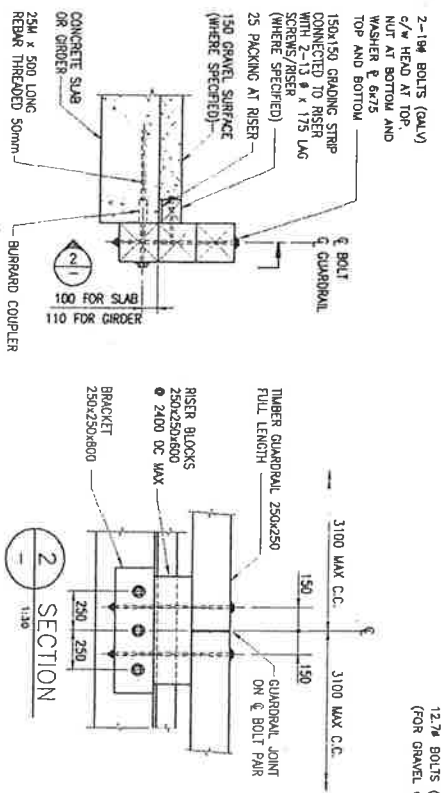




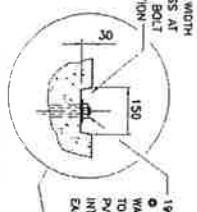
TIMBER GUARDRAIL - TIMBER DECK
1:30



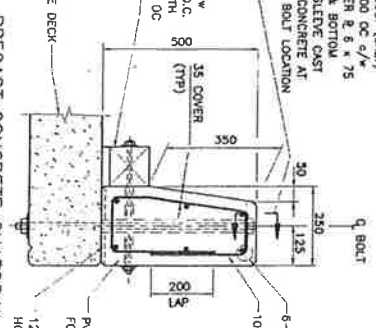
STEEL GUARDRAIL - CONCRETE DECK/GIRDER
1:30



TIMBER GUARDRAIL - CONCRETE SLAB/GIRDER
1:30



19M BOLT (GALV)
1000 OC c/w
TOP & BOTTOM
PVC SLEEVE CAST
INTO CONCRETE AT
EACH BOLT LOCATION



PRECAST CONCRETE GUARDRAIL
CONCRETE SLAB/GIRDER
1:30

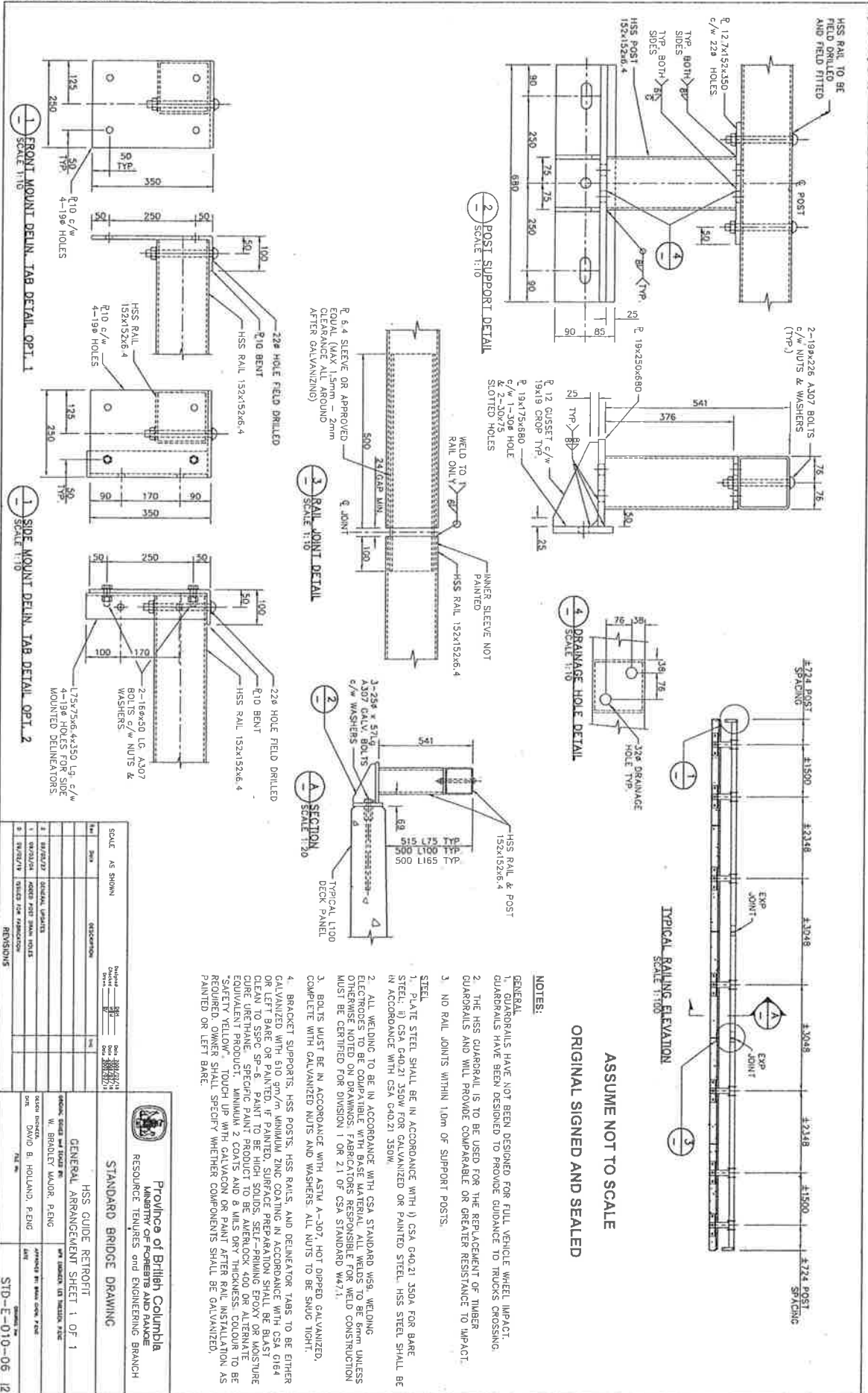
NOTES:

1. GENERAL
- 1.1 GUARDRAILS HAVE BEEN DESIGNED TO PROVIDE GUIDANCE TO TRUCKS CROSSING BRIDGE. GUARDRAILS HAVE NOT BEEN DESIGNED TO WITHSTAND FULL IMPACT OF TRUCK WHEEL AGAINST GUARDRAIL
2. TIMBER
- 2.1 ALL GUARDRAIL TIMBER IS TO BE ROUGH SAWN UNTREATED SPRUCE-PINE-FIR NO. 2 OR BETTER
- 2.2 ALL ASSOCIATED STEEL HARDWARE (BOLTS, WASHERS, NUTS) TO BE GALVANIZED WITH 610 gm/m² MINIMUM ZINC COATING IN ACCORDANCE WITH CSA G164
- 2.3 ALL BOLTS ARE TO BE A307
3. STEELWORK
- 3.1 STEEL SECTIONS AND PLATE TO CONFORM TO CSA C41.21-92 GRADE 350A
- 3.2 WELDING TO BE IN ACCORDANCE WITH CSA-W59
4. CONCRETE
- 4.1 GUARDRAIL CONCRETE TO BE 28MPa AT 28 DAYS.
- 4.2 ALL REINFORCING STEEL TO BE DEFORMED BARS IN ACCORDANCE WITH CSA C30.18 GRADE 400
- 4.3 ALL EXPOSED FACES TO HAVE 20 CHAMFER.
- 4.4 EXPOSED CORNERS AND FACES TO BE GIVEN A CLASS 2 FINISH AS DEFINED IN SECTION 2.11 OF THE PROVINCE OF BRITISH COLUMBIA STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION.

<p>ISSUE RECORD</p> <table border="1"> <tr> <th>NO.</th> <th>DATE</th> <th>DESCRIPTION</th> </tr> <tr> <td> </td> <td> </td> <td> </td> </tr> </table>	NO.	DATE	DESCRIPTION				<p>SCALE</p> <p>Author</p> <p>Checked</p> <p>Drawn</p> <p>Scale</p>	<p>Province of British Columbia Ministry of Forests Resource Tenures and Engineering Branch</p> <p>STANDARD BRIDGE DRAWING</p> <p>GUARDRAILS GUARDRAIL DETAILS</p> <p>DRAWN BY: [Signature] CHECKED BY: [Signature]</p> <p>DATE: [Date]</p> <p>PROJECT: [Project Name]</p> <p>SCALE: [Scale]</p>
NO.	DATE	DESCRIPTION						

ASSUME NOT TO SCALE

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ORIGINAL SIGNED AND SEALED

NOTES:

- GENERAL**
1. GUARDRAILS HAVE NOT BEEN DESIGNED FOR FULL VEHICLE WHEEL IMPACT. GUARDRAILS HAVE BEEN DESIGNED TO PROVIDE GUIDANCE TO TRUCKS CROSSING IN ACCORDANCE WITH CSA G40.21 350V.
 2. THE HSS GUARDRAIL IS TO BE USED FOR THE REPLACEMENT OF TIMBER GUARDRAILS AND WILL PROVIDE COMPARABLE OR GREATER RESISTANCE TO IMPACT.
 3. NO RAIL JOINTS WITHIN 1.0m OF SUPPORT POSTS.
- STEEL**
1. PLATE STEEL SHALL BE IN ACCORDANCE WITH 1) CSA G40.21 350A FOR BARE STEEL; 1) CSA G40.21 350V FOR GALVANIZED OR PAINTED STEEL. HSS STEEL SHALL BE IN ACCORDANCE WITH CSA G40.21 350V.
 2. ALL WELDING TO BE IN ACCORDANCE WITH CSA STANDARD W59. WELDING ELECTRODES TO BE COMPATIBLE WITH BASE MATERIAL. ALL WELDS TO BE 9mm UNLESS OTHERWISE NOTED ON DRAWINGS. FABRICATORS RESPONSIBLE FOR WELD CONSTRUCTION MUST BE CERTIFIED FOR DIVISION 1 OR 2.1 OF CSA STANDARD W47.1.
 3. BOLTS MUST BE IN ACCORDANCE WITH ASTM A-307, HOT DIPPED GALVANIZED, COMPLETE WITH GALVANIZED NUTS AND WASHERS. ALL NUTS TO BE SNUG TIGHT.
 4. BRACKET SUPPORTS, HSS POSTS, HSS RAILS, AND DELINEATOR TABS TO BE EITHER GALVANIZED WITH 610 gm/m² MINIMUM ZINC COATING IN ACCORDANCE WITH CSA G16.4 OR LEFT BARE OR PAINTED. IF PAINTED, SURFACE PREPARATION SHALL BE BLAST CLEAN TO SSPC SP-6. PAINT TO BE HIGH SOLIDS SELF-PRIMING EPOXY OR MOSTURE CURABLE URETHANE. SPECIFIC PAINT PRODUCTS TO BE AMERLOCK 400 OR ALTERNATE EQUIVALENT PRODUCT. MINIMUM 2 COATS AND B MILS DRY THICKNESS. COLOUR TO BE YELLOW. FLOUR SHOULD UP WITH GALVANON OR PAINT AFTER RAIL INSTALLATION AS REQUIRED. WELDER SHALL SPECIFY WHETHER COMPONENTS SHALL BE GALVANIZED, PAINTED OR LEFT BARE.

<p>Province of British Columbia MINISTRY OF FORESTS AND RANGE RESOURCE TENURES AND ENGINEERING BRANCH</p>											
<p>STANDARD BRIDGE DRAWING</p>											
<p>HSS GUIDE RETROFIT GENERAL ARRANGEMENT SHEET 1 OF 1</p>											
DESIGNED BY DAVID B. HOLLAND, P.ENG	CHECKED BY W. BRADLEY MAJOR, P.ENG	DRAWN BY DAVID B. HOLLAND, P.ENG									
<p>DATE: 11/10/10</p>											
<p>SCALE: AS SHOWN</p>											
<p>REVISIONS</p> <table border="1"> <thead> <tr> <th>NO.</th> <th>DATE</th> <th>DESCRIPTION</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>11/10/10</td> <td>ISSUED FOR PERMIT</td> </tr> <tr> <td>2</td> <td>11/10/10</td> <td>REVISED FOR GALVANIZING</td> </tr> </tbody> </table>			NO.	DATE	DESCRIPTION	1	11/10/10	ISSUED FOR PERMIT	2	11/10/10	REVISED FOR GALVANIZING
NO.	DATE	DESCRIPTION									
1	11/10/10	ISSUED FOR PERMIT									
2	11/10/10	REVISED FOR GALVANIZING									
<p>PROJECT NO. 109-PC-006623</p>		<p>SHEET NO. 12</p>									
<p>STD-E-010-06</p>											

B Appendix B - Ontario Ministry of Transportation Sample LVPL1 and LVPL2 Bridge Barriers and other Standard Bridge Barriers



**Exceptions to the Canadian Highway Bridge
Design Code CAN/CSA-S6-00 for Ontario
June 2002**

Excerpt From Structural Manual – Division 1

Ministry of Transportation
Engineering Standards Branch
Bridge Office
June 2002

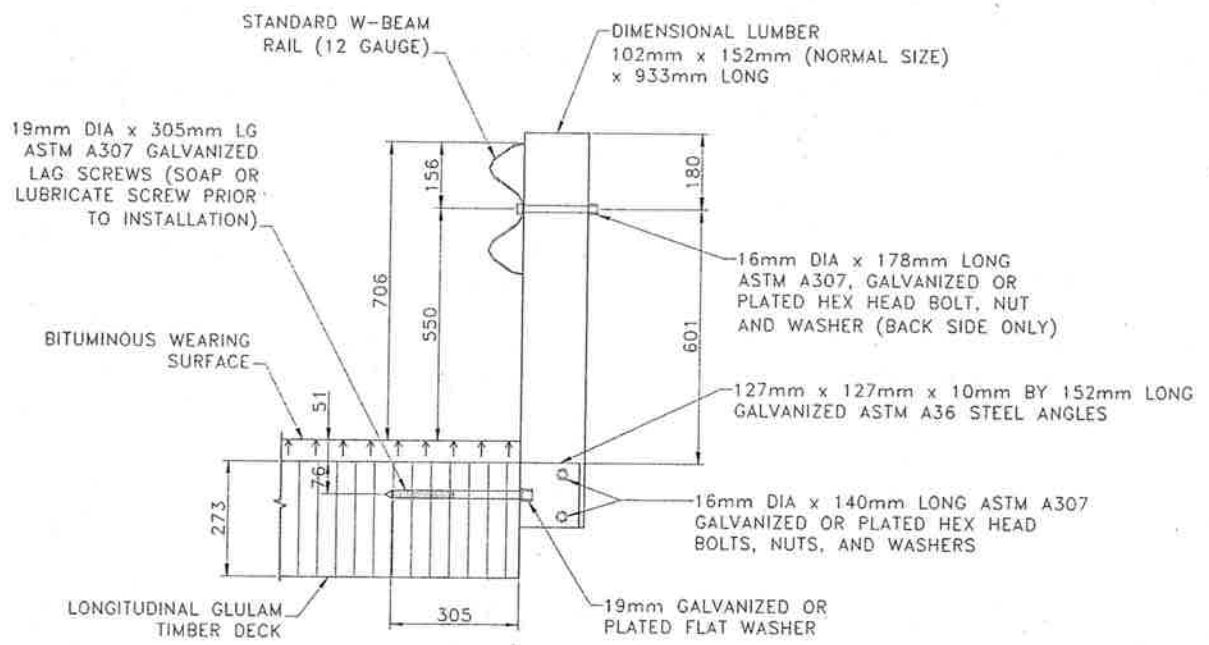


Figure 1: Approved LVPL2 barrier
(TL-1)

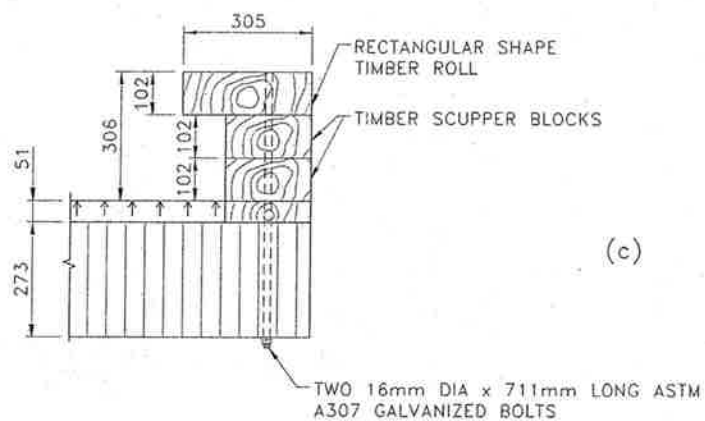
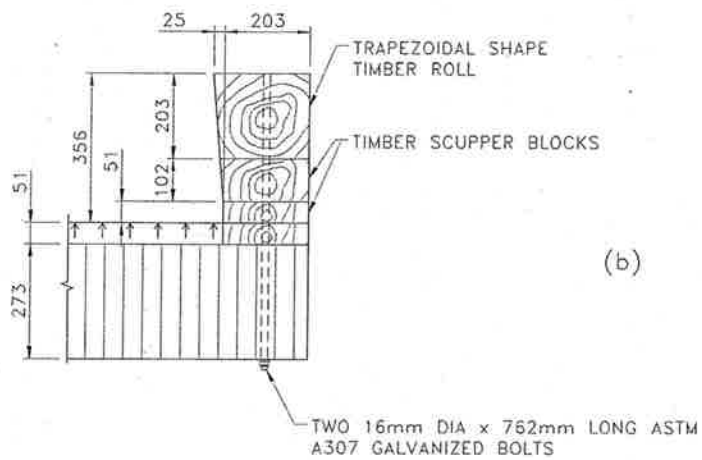
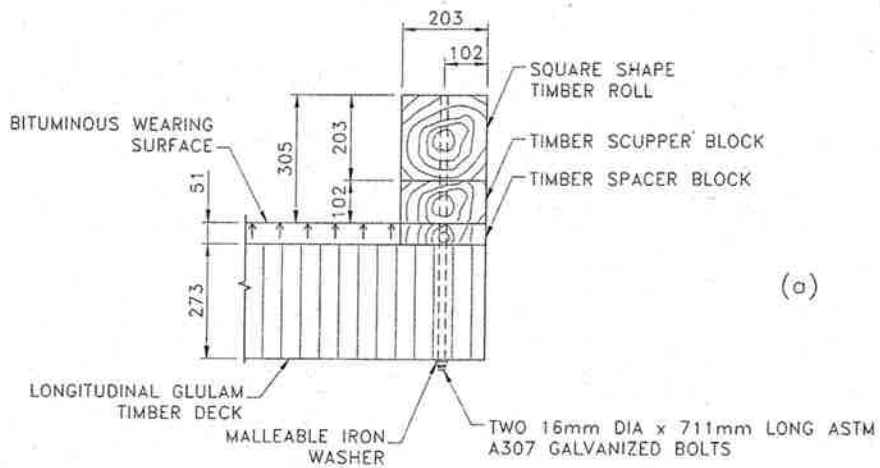
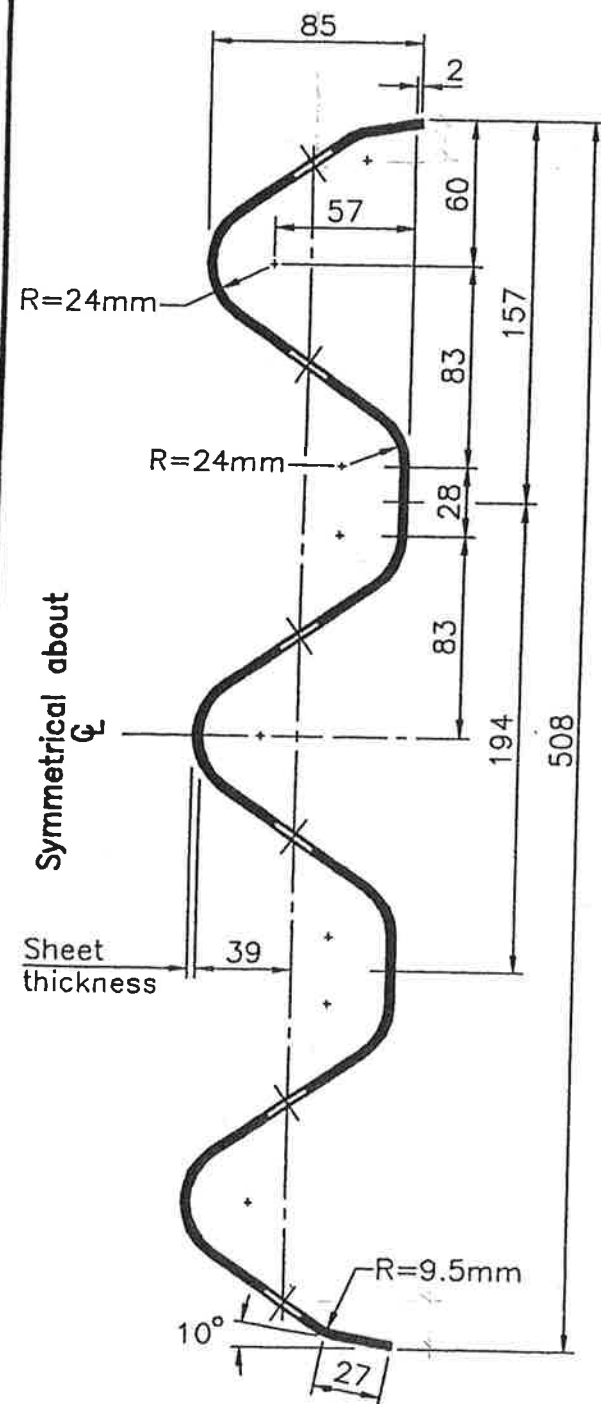
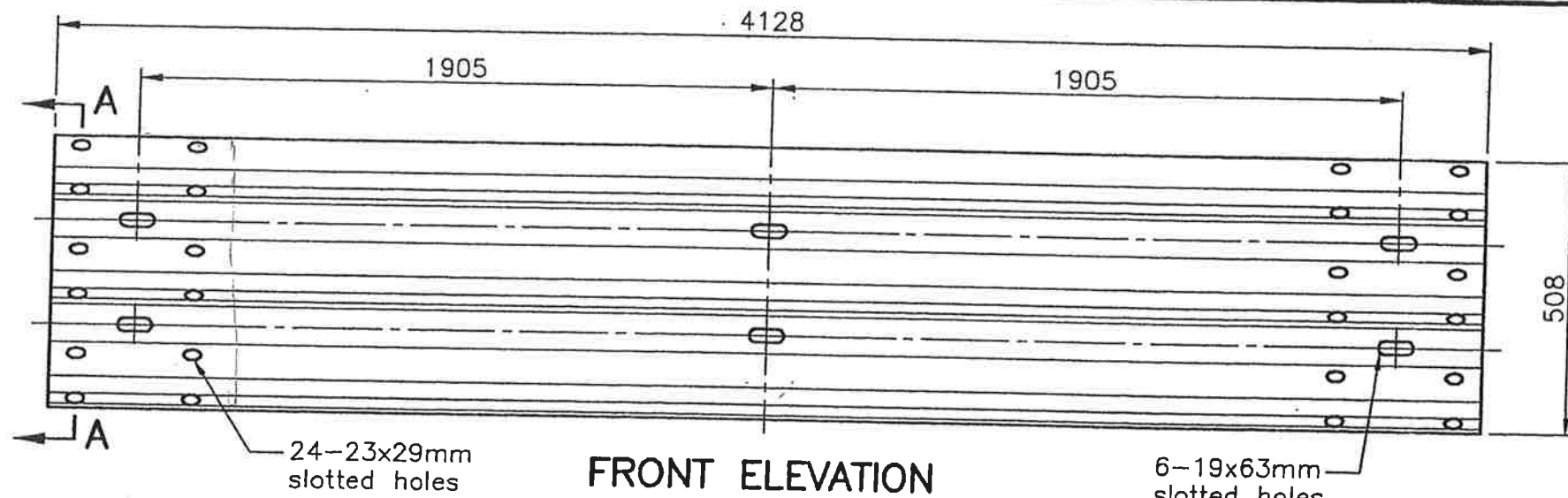


Figure 2: Approved LVPL1 barrier systems
(Sub-TL-1)

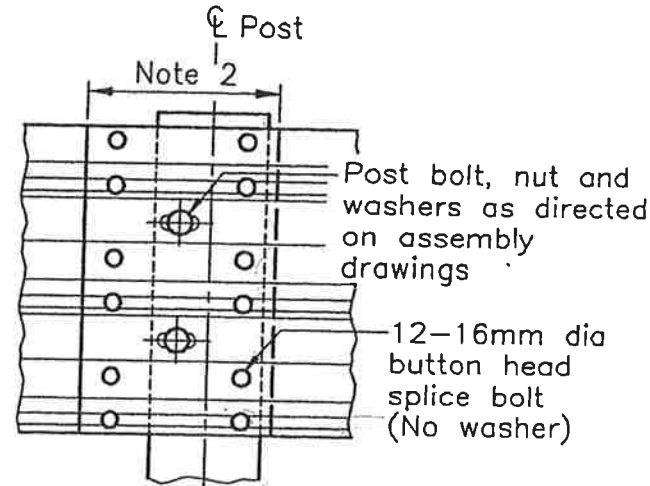


ENLARGED SECTION A-A

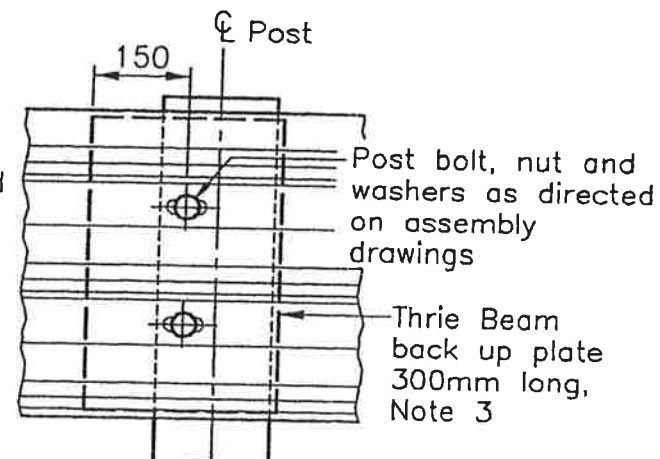
Note 1



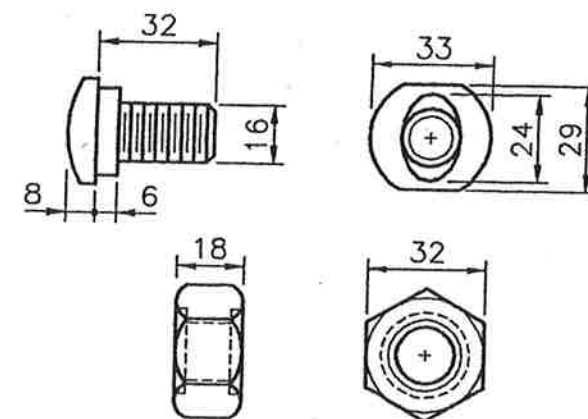
FRONT ELEVATION



SPLICE DETAIL



THRIE BEAM
BACK UP PLATE
DETAIL



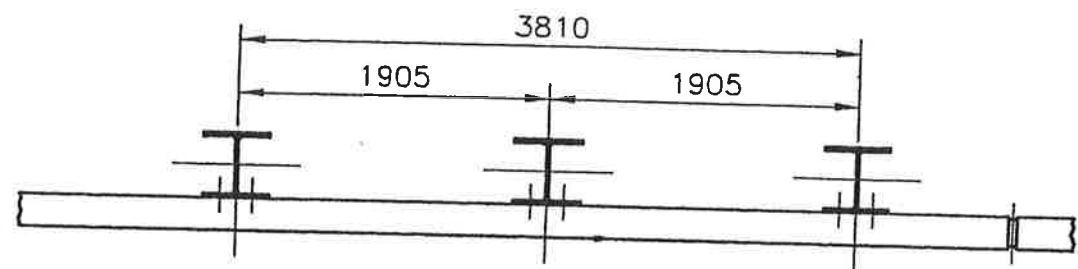
SPLICE BOLT AND NUT

NOTES:

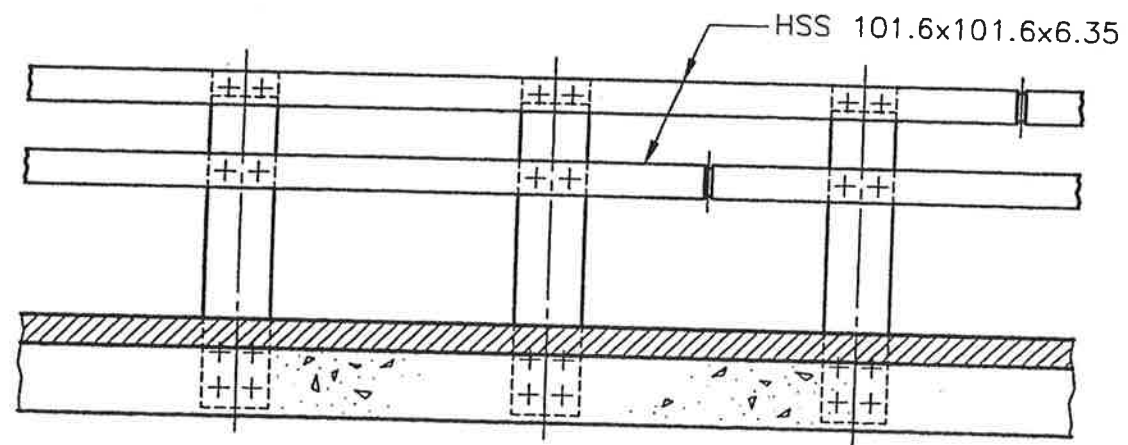
- 1 Thickness of rail shall be as shown on assembly drawings.
 - 2 Overlap guide rail elements in direction of traffic flow.
 - 3 The 300mm section of Thrie Beam (back up plate) shall be installed at intermediate posts only.
- A Thrie Beam shall conform to AASHTO Specification M180 Type 2.
 B All dimensions subject to manufacturing tolerances unless otherwise indicated.
 C All dimensions are in millimetres or metres unless otherwise shown.

THRIE BEAM GUIDE RAIL
RAIL DETAIL

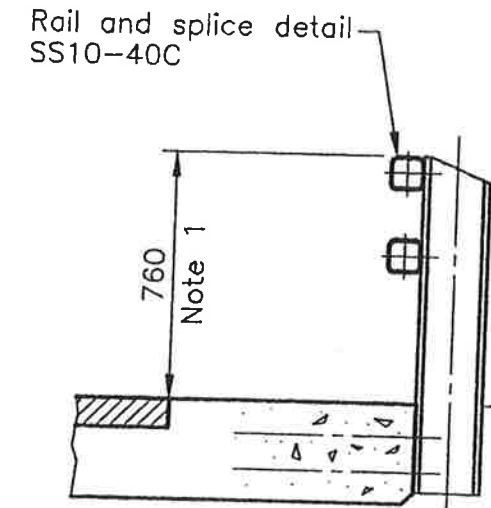
Date	JUNE 1993	Rev	
SS10-20			



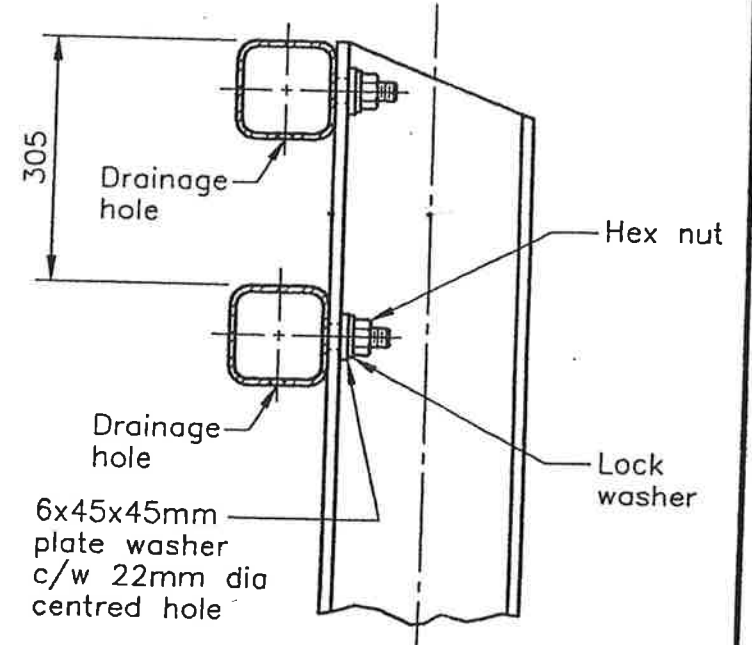
PLAN



ELEVATION



SIDE VIEW



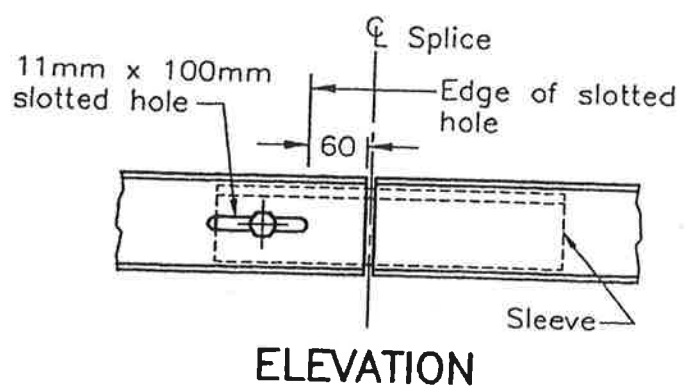
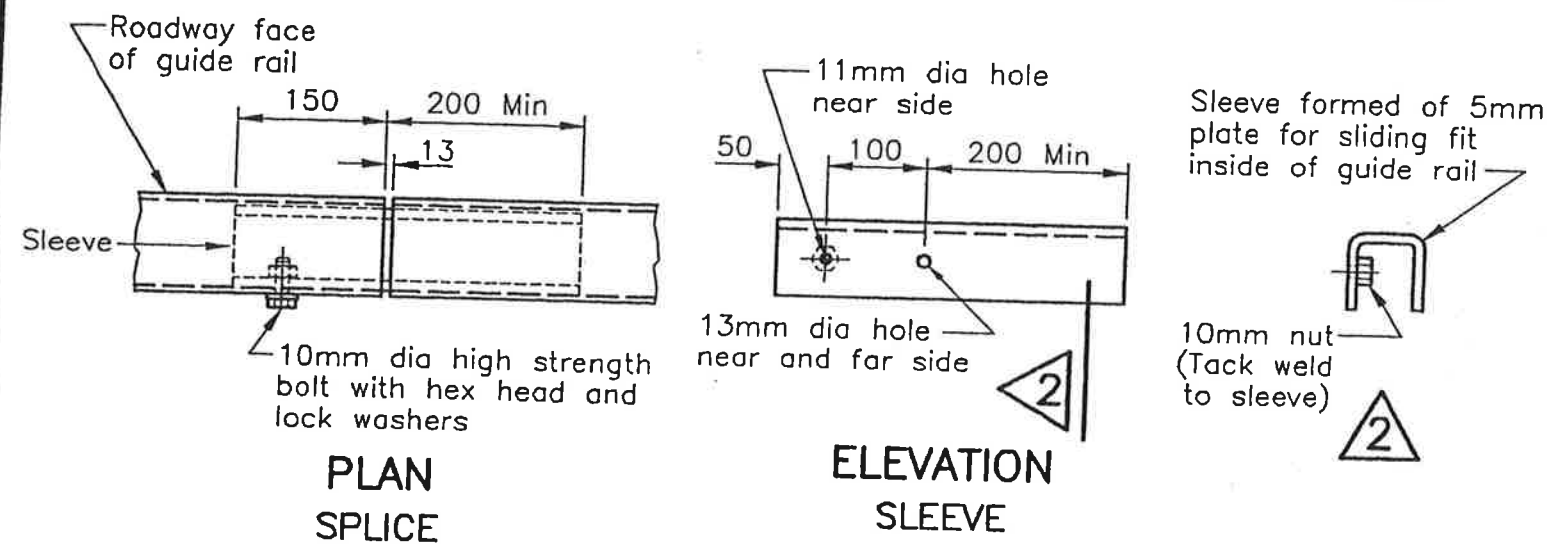
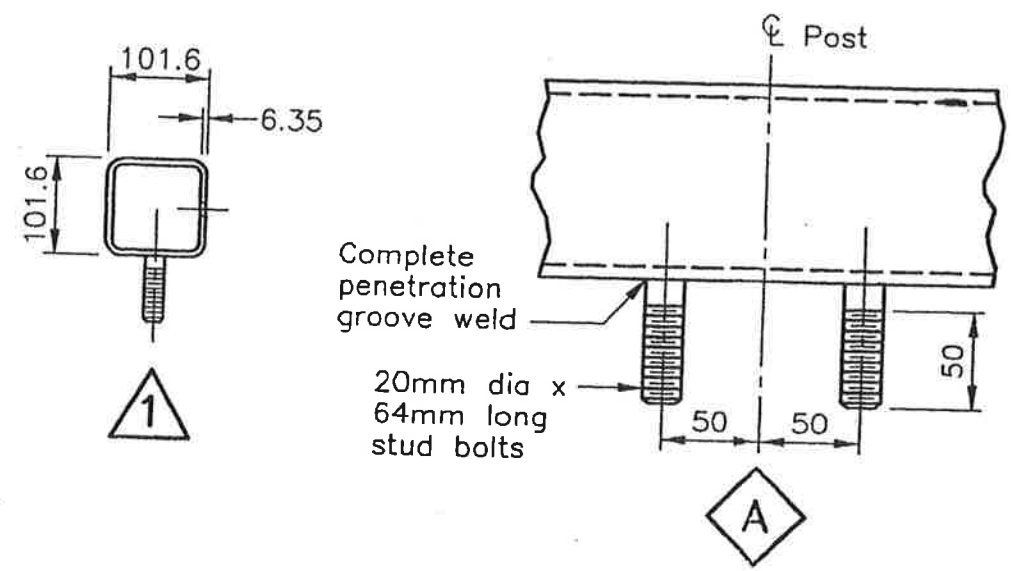
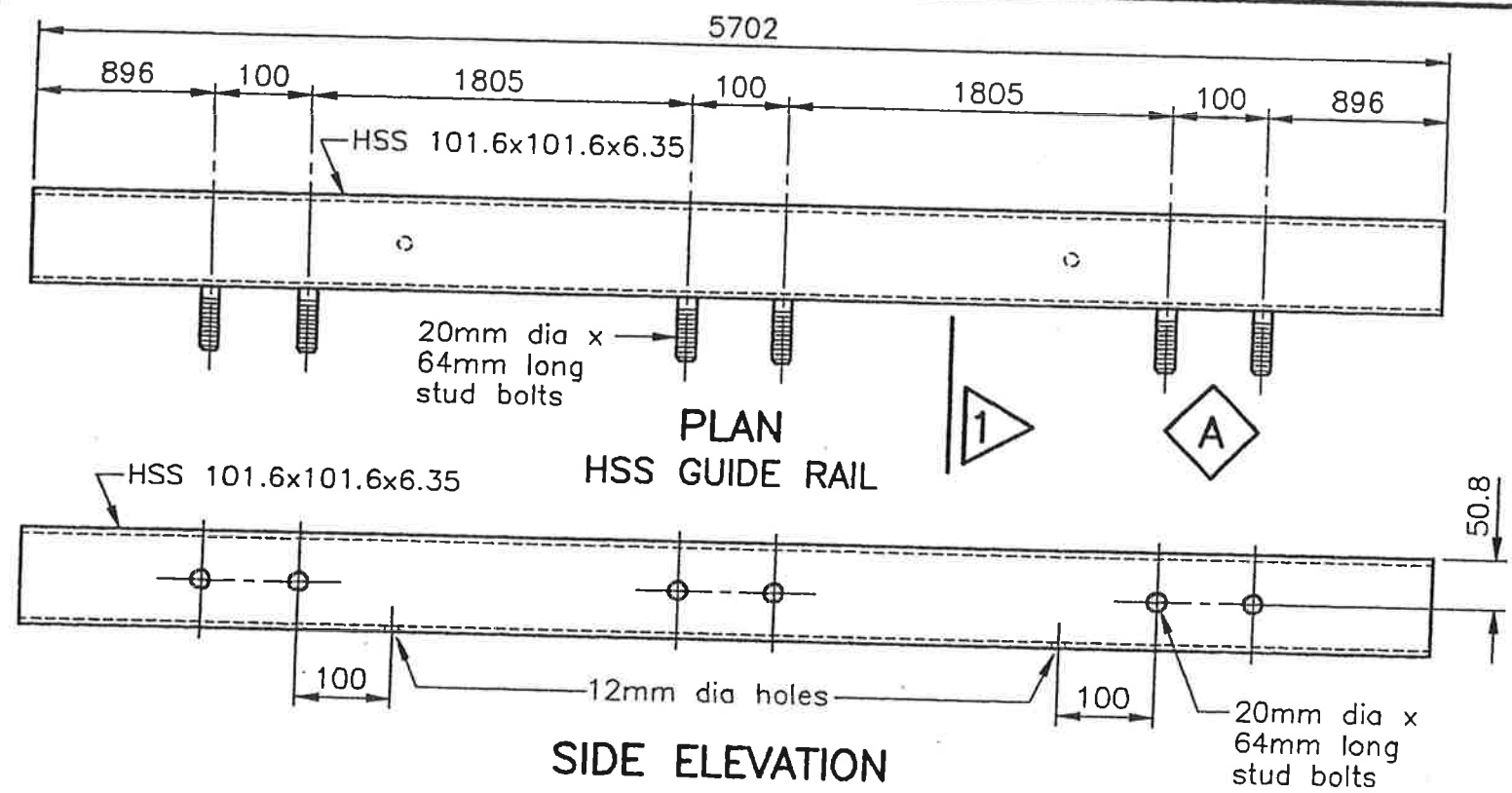
DETAIL

NOTES:

- 1 Mounting height shall be measured to top of guide rail.
- A This Standard to be read in conjunction with SS10-40B and SS10-40C.
- B Posts shall be vertical. Tubing shall be parallel to vertical alignment.
- C Tubing shall be shop bent to fit horizontal curve when curve radius is less than 290 m.
- D Guide rails shall be continuous over not less than 3 intermediate posts, with a minimum length of 3 panels, except as noted.
- E No more than one rail is to be spliced in each panel, except as noted.
- F Space posts to provide 500mm clear between expansion joints and C of post.
- G Rail joints in top and bottom beams at deck expansion joints shall provide allowance for movement equal to width of deck joint with corresponding increase of length of sleeve.
- H Stud bolt nuts shall be torqued to 237 N.m.
- J All dimensions are in millimetres or metres unless otherwise shown.

**BOX BEAM GUIDE RAIL – SIDE MOUNT
ASSEMBLY DETAIL
PERFORMANCE LEVEL 1**

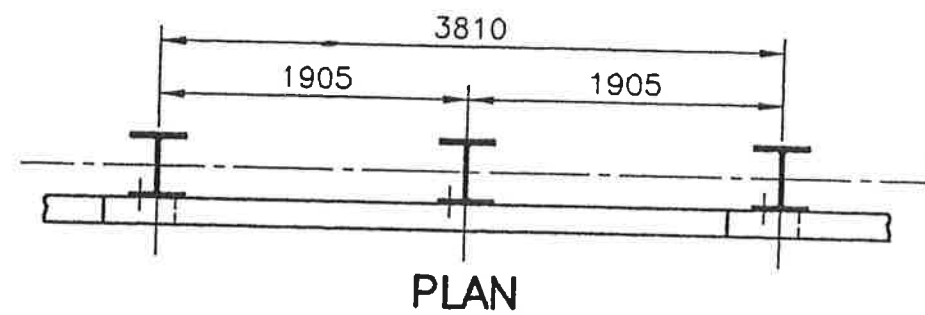
Date	JUNE 1993	Rev	
SS10-40A			



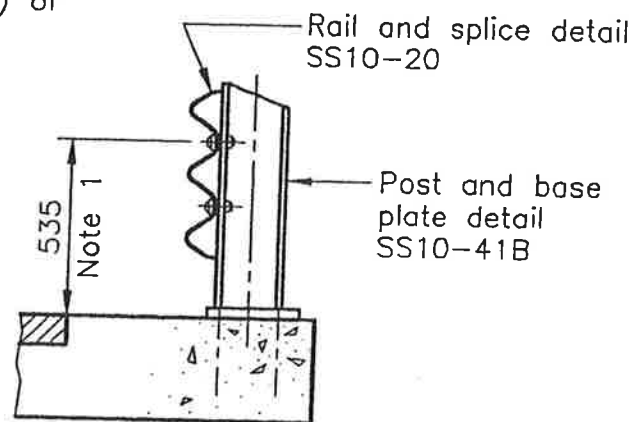
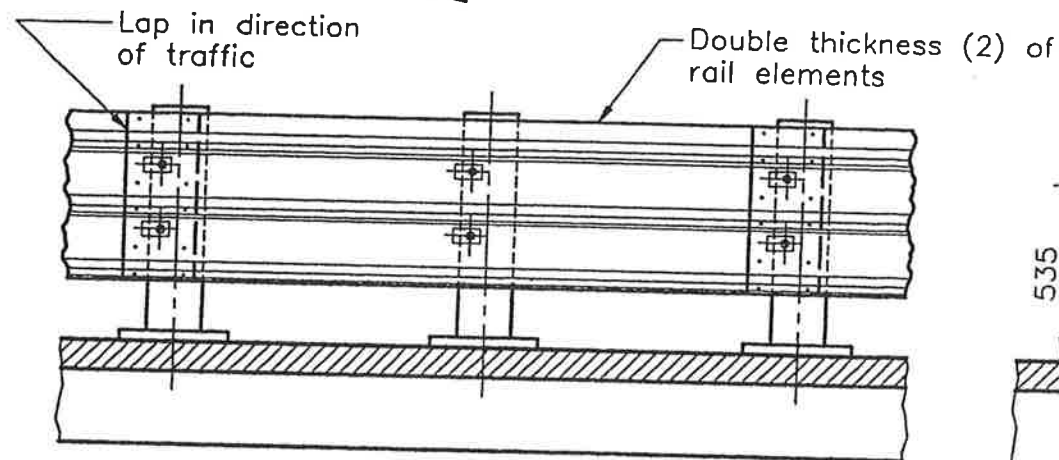
- NOTES:**
- A This Standard to be read in conjunction with SS10-40A and SS10-40B.
 - B HSS guide rail shall conform to CSA Standard CAN3-G40.21-M92 Grade 350W.
 - C Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
 - D Stud bolts shall conform to ASTM Specification A449.
 - E 10mm nut may be replaced with drilled and tapped hole in sleeve.
 - F Welding shall conform to CSA Standard W59-M89 and shall be done by a welder qualified under CSA Standard W47-M83.
 - G HSS guide rail, rail splices and fasteners shall be hot dip galvanized after shop fabrication in conformance to CSA Standard G164-M92.
 - H All dimensions are in millimetres or metres unless otherwise shown.

**BOX BEAM GUIDE RAIL – SIDE MOUNT
RAIL AND SPLICE DETAIL
PERFORMANCE LEVEL 1**

Date	JUNE 1993	Rev	
SS10-40C			



DIRECTION OF TRAFFIC

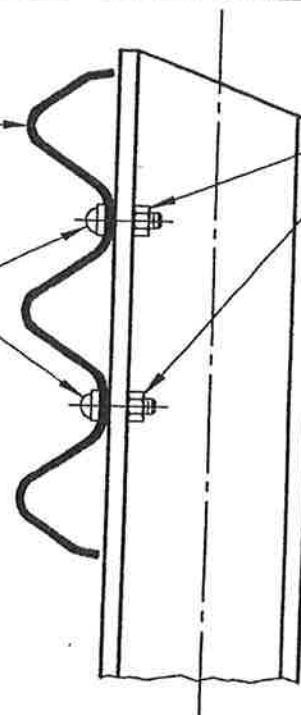


SIDE VIEW

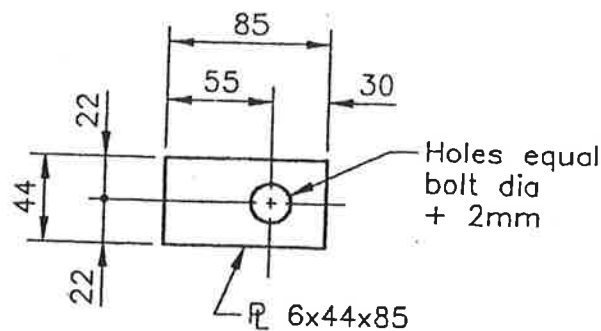
2-Thrie beam rail elements (12 Gauge)

5/8" dia ASTM A307 galvanized button head post bolts, nuts and washers (2 per post)

1-Plate washer "A" and 1-SAE lock washer



DETAIL



NOTE: Position washer to completely cover slotted hole

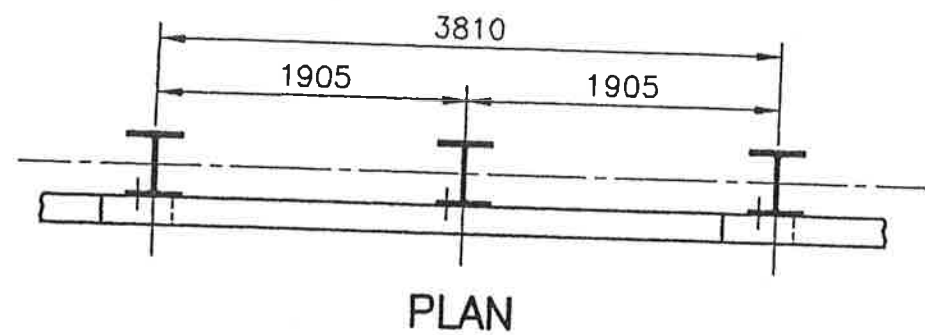
PLATE WASHER 'A'

NOTES:

- 1 Mounting height of thrie beam above deck shall be measured to centre line of top bolt.
- A This Standard to be read in conjunction with SS10-41B and SS10-20.
- B Railing shall be fabricated to the horizontal and vertical alignment of the structure. Posts shall be installed normal to grade.
- C Field ream bolt holes in double thickness rail at splice locations. Repair damaged galvanizing with zinc-rich paint.
- D All dimensions are in millimetres or metres unless otherwise shown.

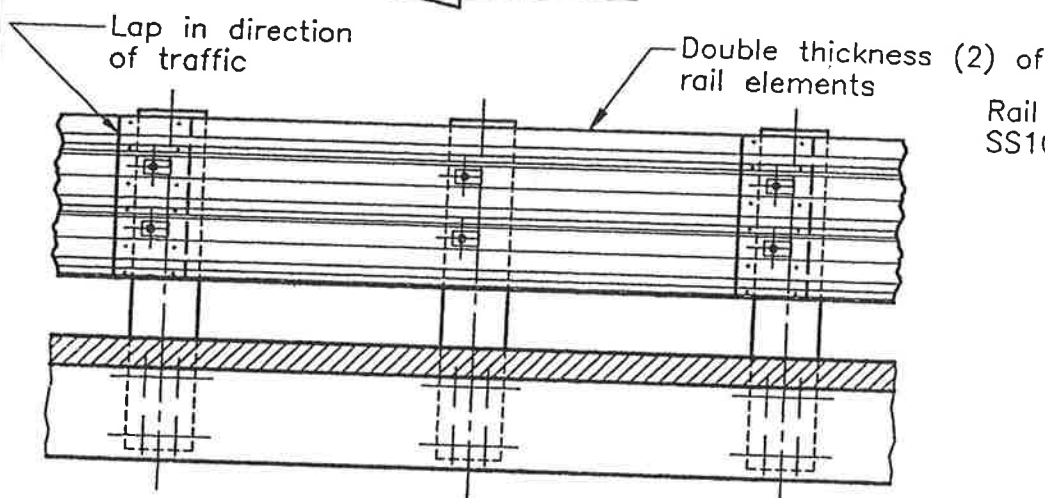
THRIE BEAM GUIDE RAIL - TOP MOUNT
ASSEMBLY DETAIL
 PERFORMANCE LEVEL 1

Date	JUNE 1993	Rev	
SS10-41A			



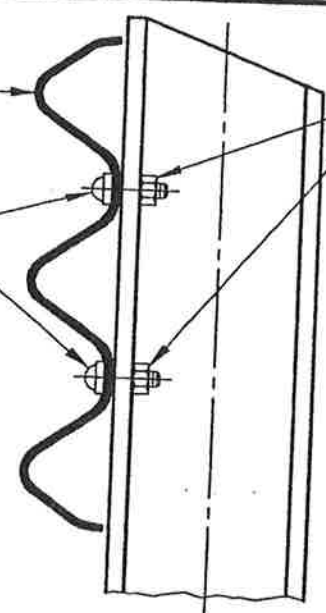
PLAN

DIRECTION OF TRAFFIC



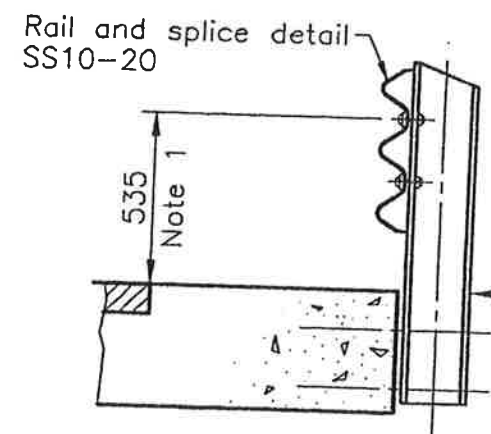
ELEVATION

2-Thrie beam rail elements (12 Gauge)
 5/8" dia ASTM A307 galvanized button head post bolts, nuts and washers (2 per post)



DETAIL

1-Plate Washer 'A' and 1 SAE lock washer



SIDE VIEW

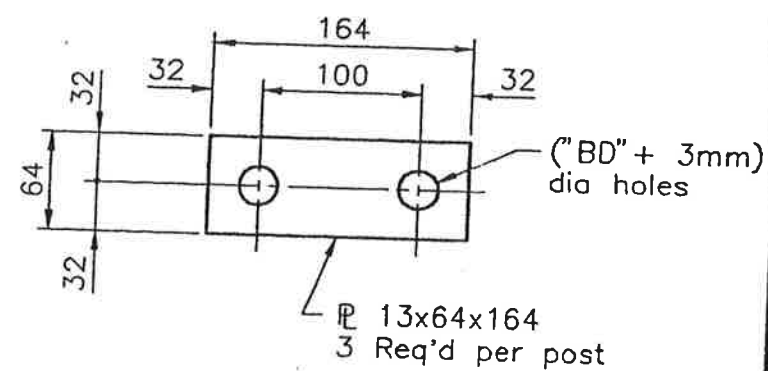
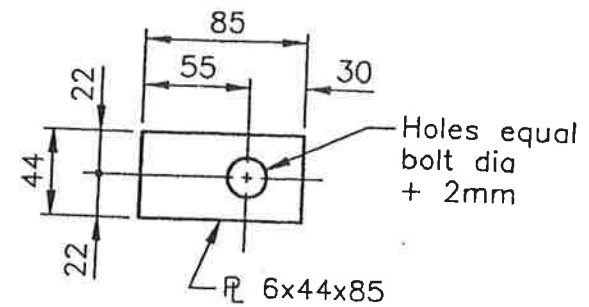


PLATE WASHER "B"



NOTE: Position washer to completely cover slotted hole

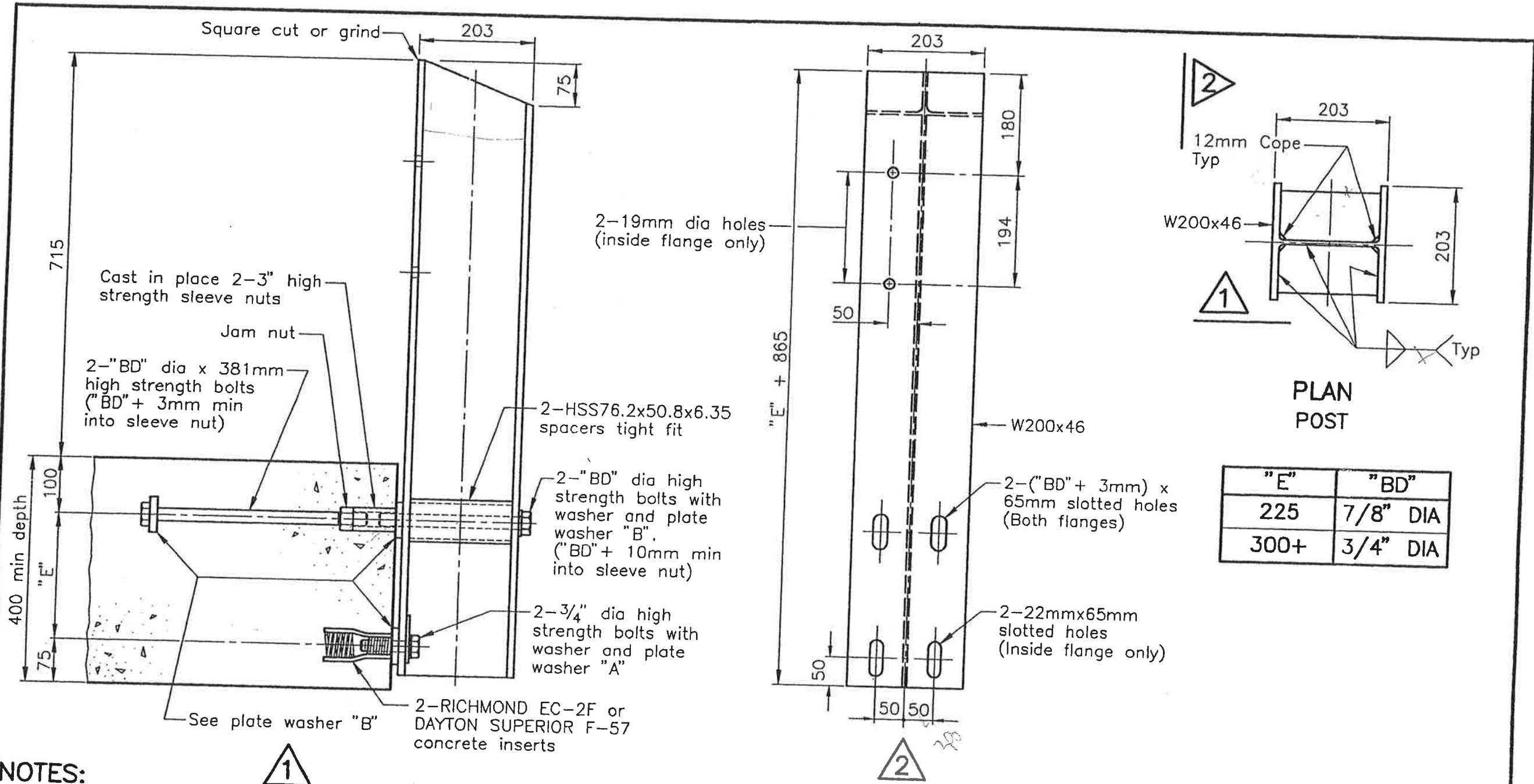
PLATE WASHER 'A'

NOTES:

- 1 Mounting height of thrie beam above deck shall be measured to centre line of top bolt.
- A This Standard to be read in conjunction with SS10-42B and SS10-20.
- B Railing shall be fabricated to the horizontal and vertical alignment of the structure. Posts shall be installed normal to grade.
- C Post shall be plumbed by shimming with washers for superelevations up to 6%. For superelevations greater than 6% post shall be set normal to roadway surface.
- D Field ream bolt holes in double thickness rail at splice locations. Repair damaged galvanizing with zinc-rich paint.
- E All dimensions are in millimetres or metres unless otherwise shown.

THRIE BEAM GUIDE RAIL - SIDE MOUNT
 ASSEMBLY DETAIL
 PERFORMANCE LEVEL 1

Date	JUNE 1993	Rev
SS10-42A		

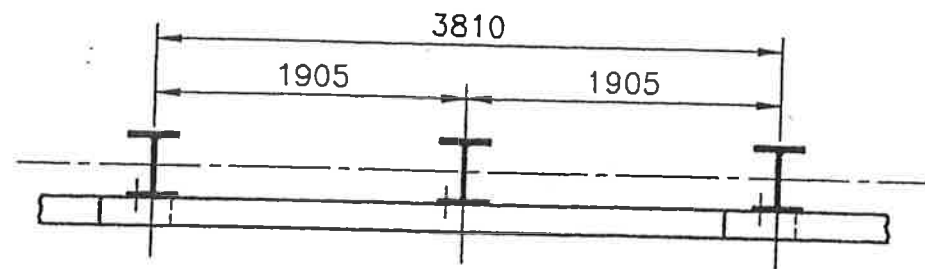


NOTES:

- A This Standard to be read in conjunction with SS10-42A and SS10-20.
- B W200x46 shall conform to CSA Standard G40.21-M92 Grade 300W.
- C Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
- D Anchor bolts shall conform to ASTM Specification A325. They shall be fitted with matching sleeve nuts, heavy hex nuts and washers.
- E Turn upper high strength post bolts 2/3 turn past snug tight condition.
- F Tighten lower high strength post bolts 1/3 turn past snug condition.
- G The anchorage assembly including bolts, nuts and washers shall be hot dip galvanized in conformance to CSA Standard G164-M92.
- H All dimensions are in millimetres or metres unless otherwise shown.

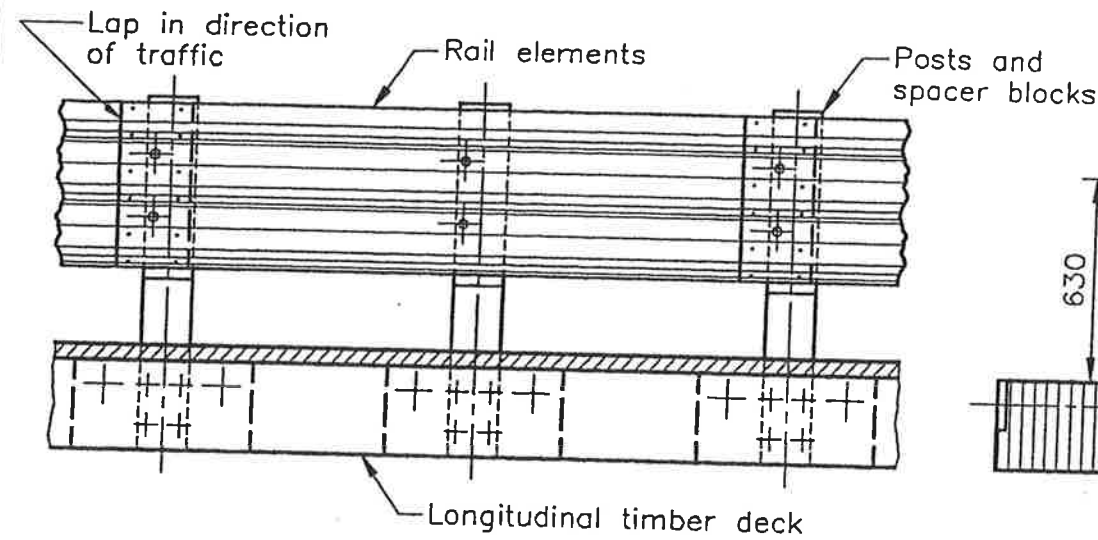
**THREE BEAM GUIDE RAIL - SIDE MOUNT
POST AND ATTACHMENT DETAILS
PERFORMANCE LEVEL 1**

Date	JUNE 1993	Rev	
SS10-42B			

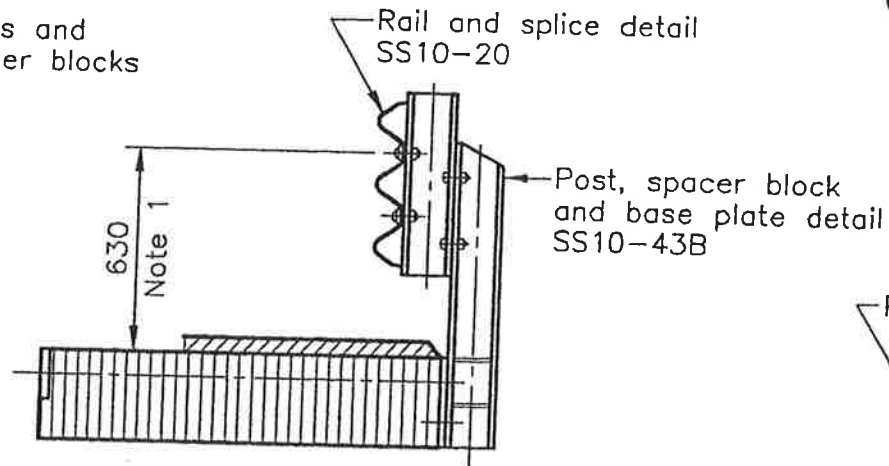


PLAN

DIRECTION OF TRAFFIC



ELEVATION

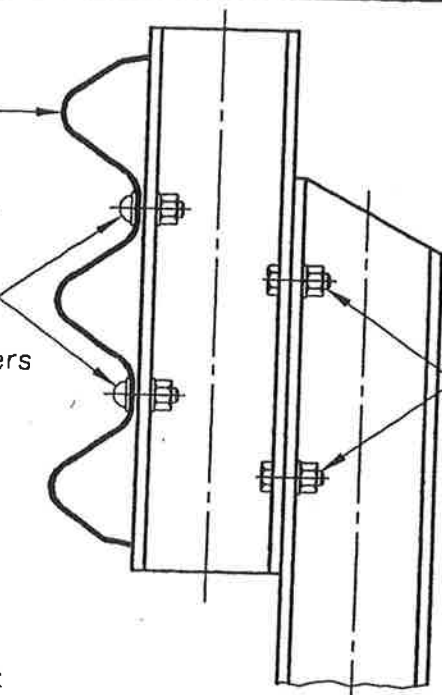


SIDE VIEW

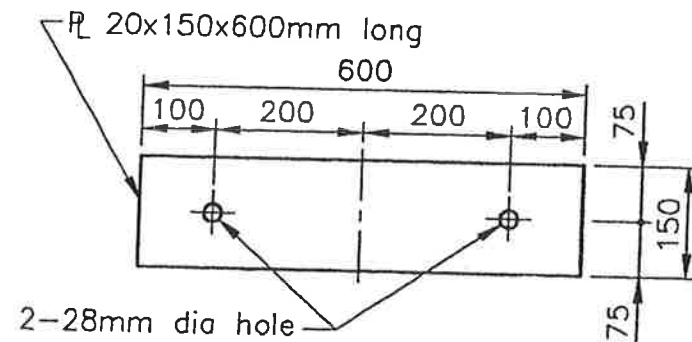
Thrie beam rail element
(10 Gauge)

5/8" dia ASTM A307
galvanized button head
post bolts, nuts and washers
(2 per post)

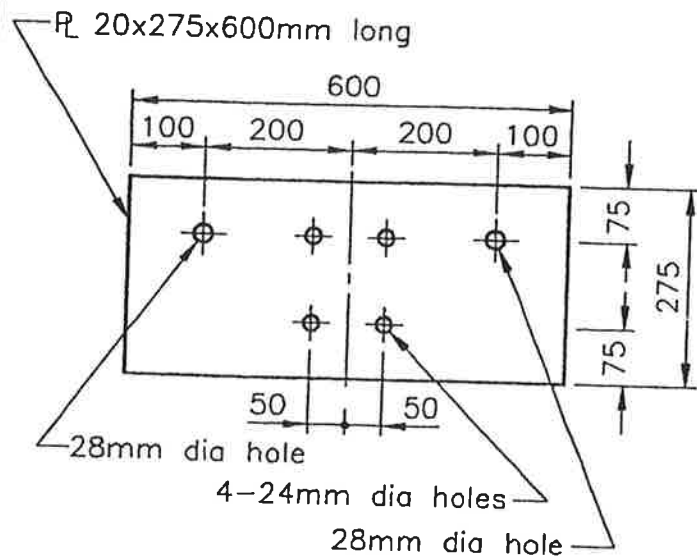
5/8" dia x 50mm long
ASTM A307 galvanized
hex bolts, nuts and
washers
(4 per post)



DETAIL



ANCHOR PLATE



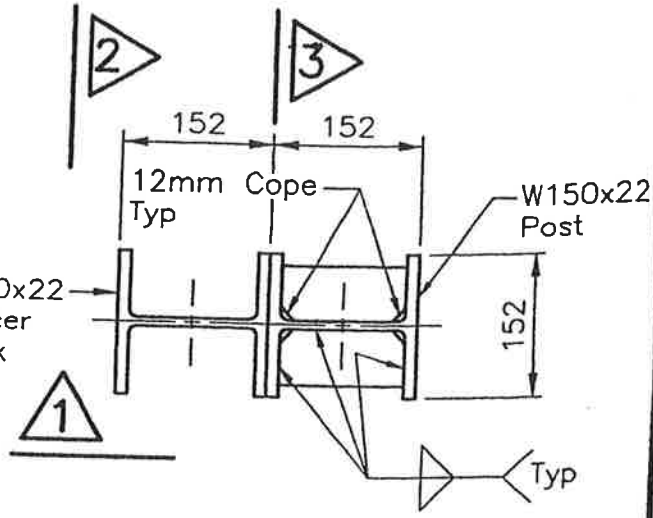
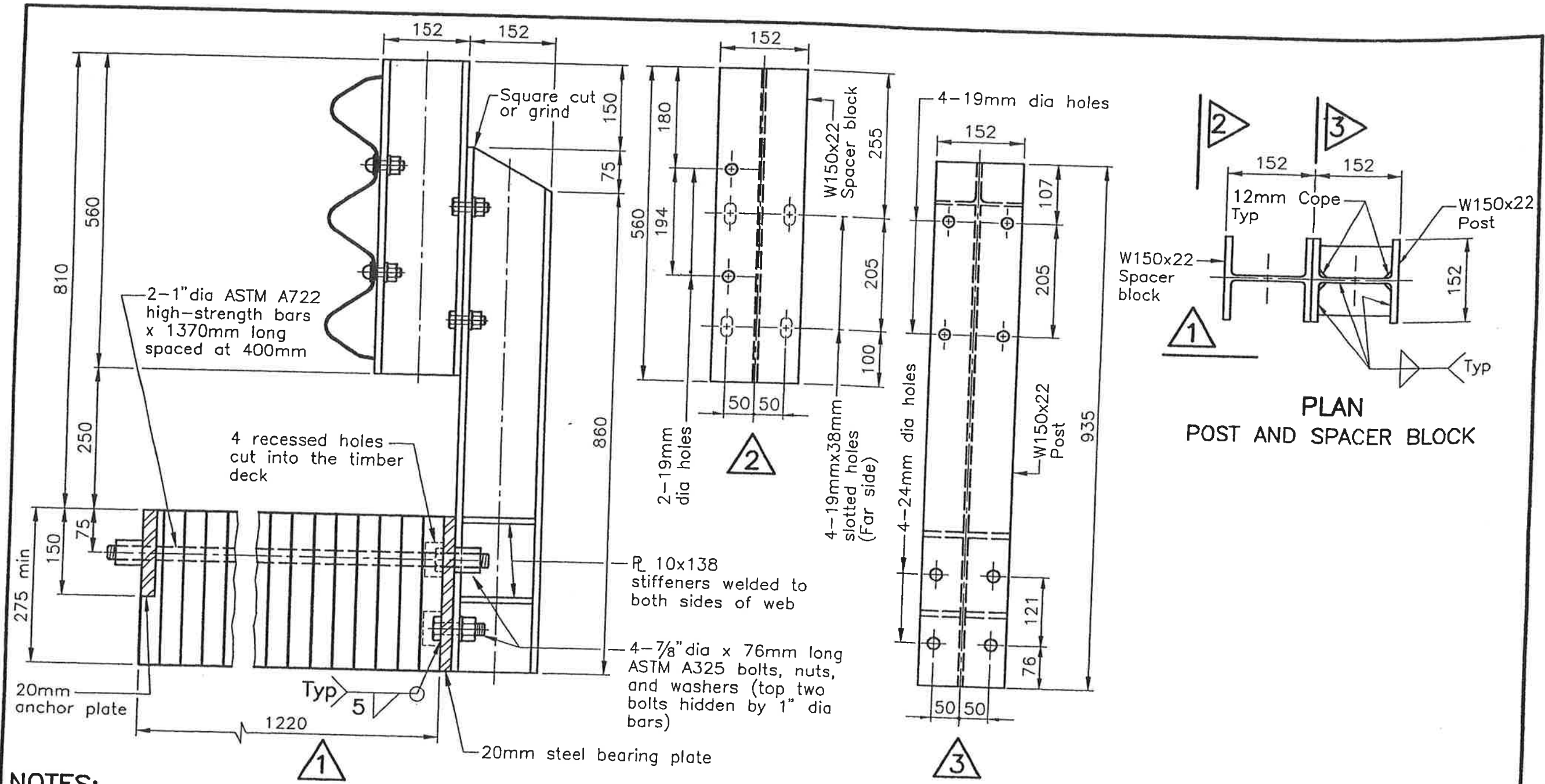
STEEL BEARING PLATE

NOTES:

- 1 Mounting height of thrie beam above deck shall be measured to centre line of top bolt.
- A This Standard to be read in conjunction with SS10-43B and SS10-20.
- B Railing shall be fabricated to the horizontal and vertical alignment of the structure. Posts and spacer blocks shall be installed normal to grade.
- C Field ream bolt holes in rail at splice locations. Repair damaged galvanizing with zinc-rich paint.
- D All dimensions are in millimetres or metres unless otherwise shown.

THRIE BEAM GUIDE RAIL - TIMBER DECK
ASSEMBLY DETAIL
PERFORMANCE LEVEL 1

Date	JUNE 1993	Rev	
SS10-43A			



PLAN
POST AND SPACER BLOCK

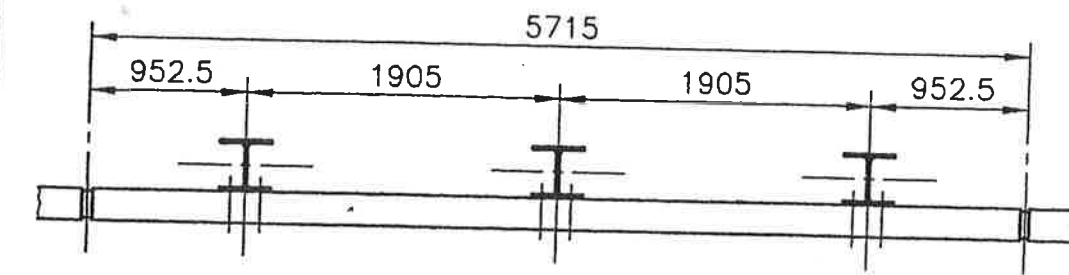
NOTES:

- A This Standard to be read in conjunction with SS10-43A and SS10-20.
- B W150x22 shall conform to CSA Standard G40.21-M92 Grade 300W. 357W
- C Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
- D High-Strength bars shall conform to ASTM Specification A722. They shall be fitted with matching nuts, heavy hex nuts and washers.
- E High-Strength bolts shall conform to ASTM Specification A325. They shall be fitted with matching heavy hex nuts and washers.

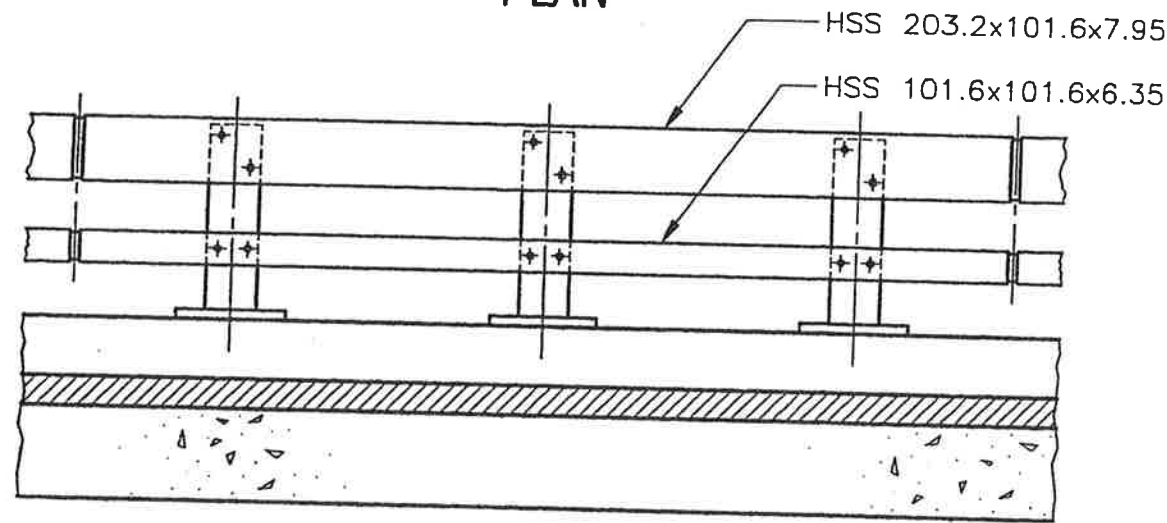
- F All components shall be hot dip galvanized in conformance to CSA Standard G164-M92.
- G All dimensions are in millimetres or metres unless otherwise shown.

**THREE BEAM GUIDE RAIL - TIMBER DECK
POST AND ATTACHMENT DETAILS
PERFORMANCE LEVEL 1**

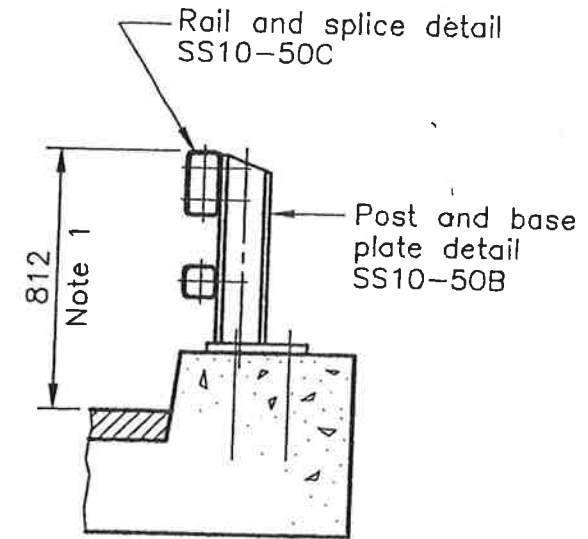
Date	JUNE 1993	Rev	
SS10-43B			



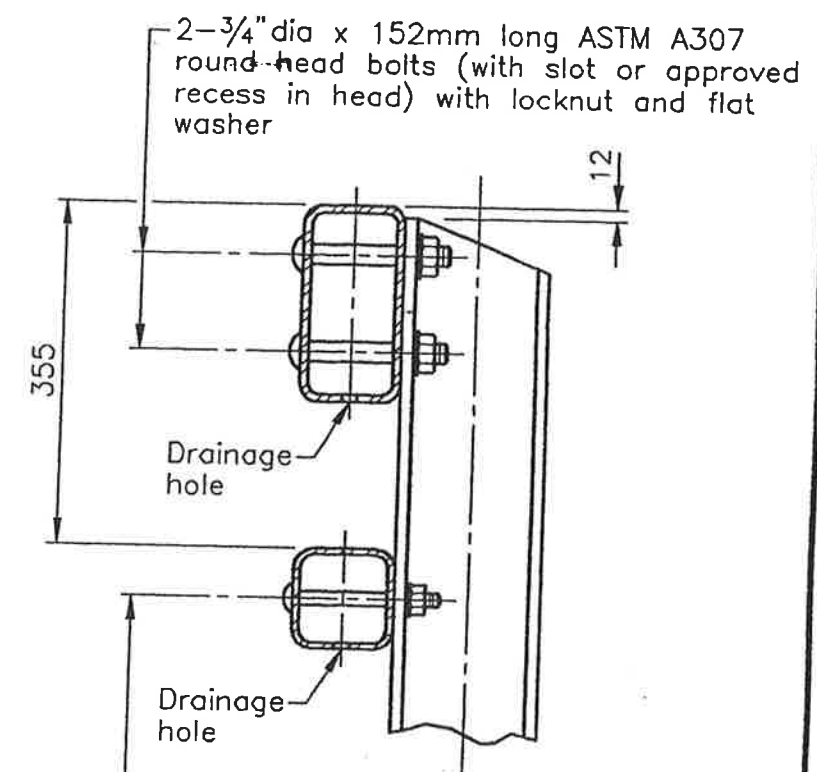
PLAN



ELEVATION



SIDE VIEW



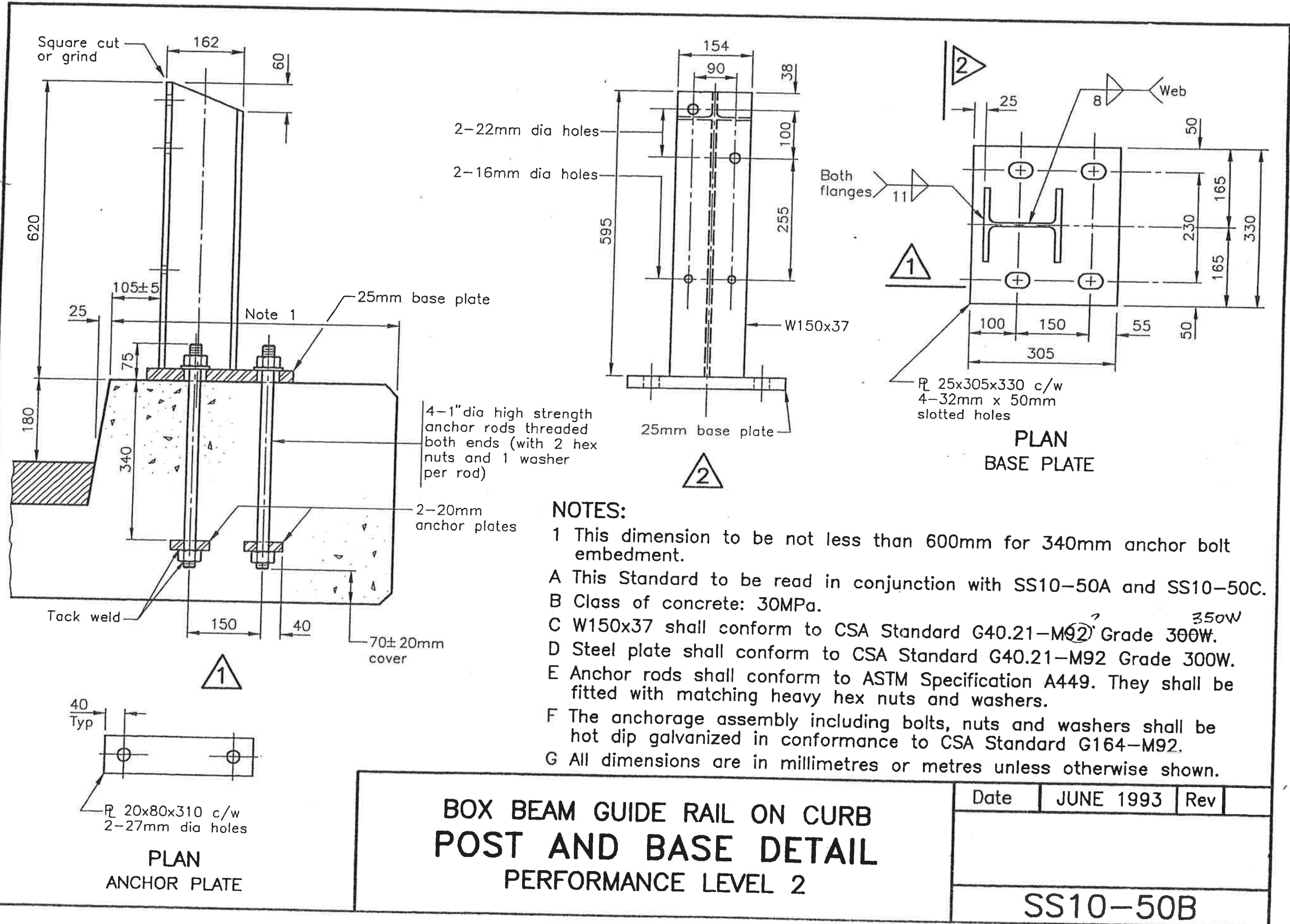
DETAIL

NOTES:

- 1 Mounting height shall be measured to top of guide rail.
- A This Standard to be read in conjunction with SS10-50B and SS10-50C.
- B Posts to be set vertical.
- C All dimensions are in millimetres or metres unless otherwise shown.

BOX BEAM GUIDE RAIL ON CURB
ASSEMBLY DETAIL
 PERFORMANCE LEVEL 2

Date	JUNE 1993	Rev	
SS10-50A			

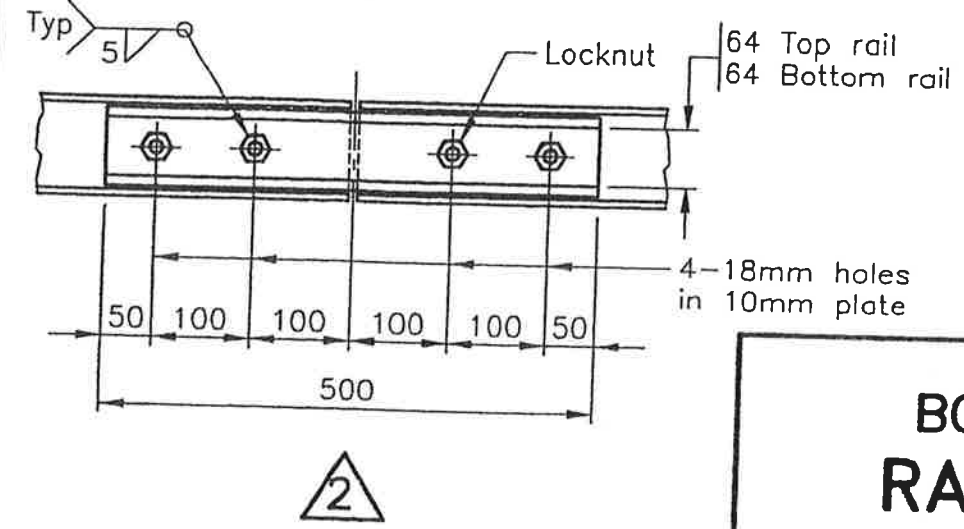
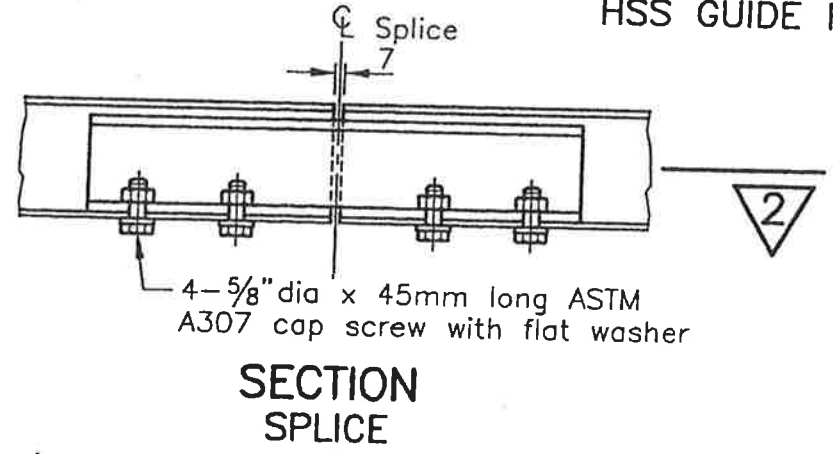
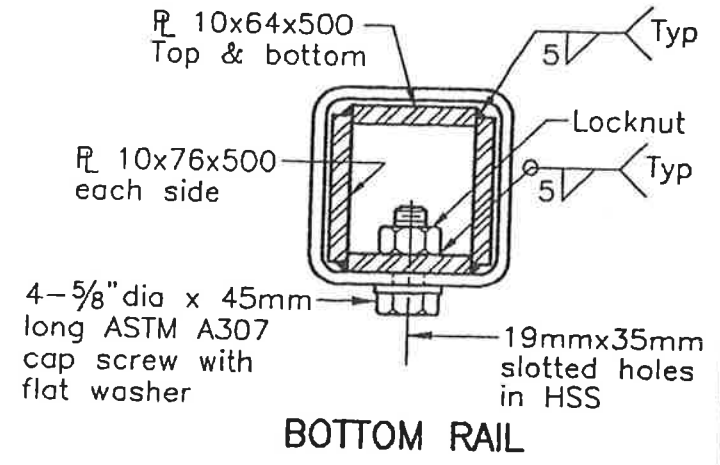
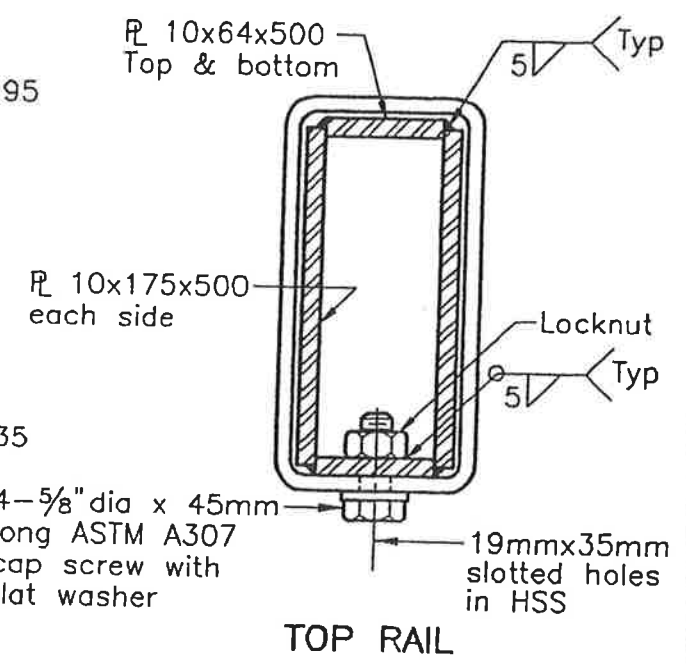
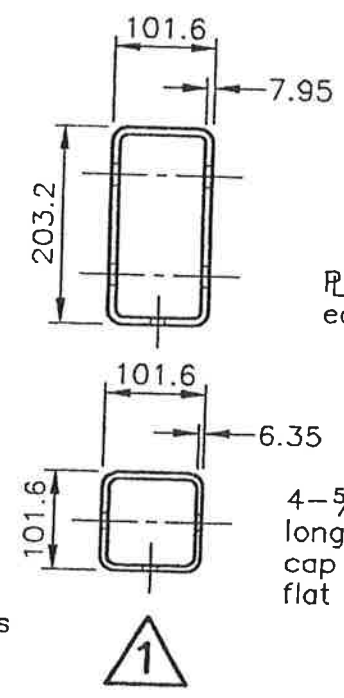
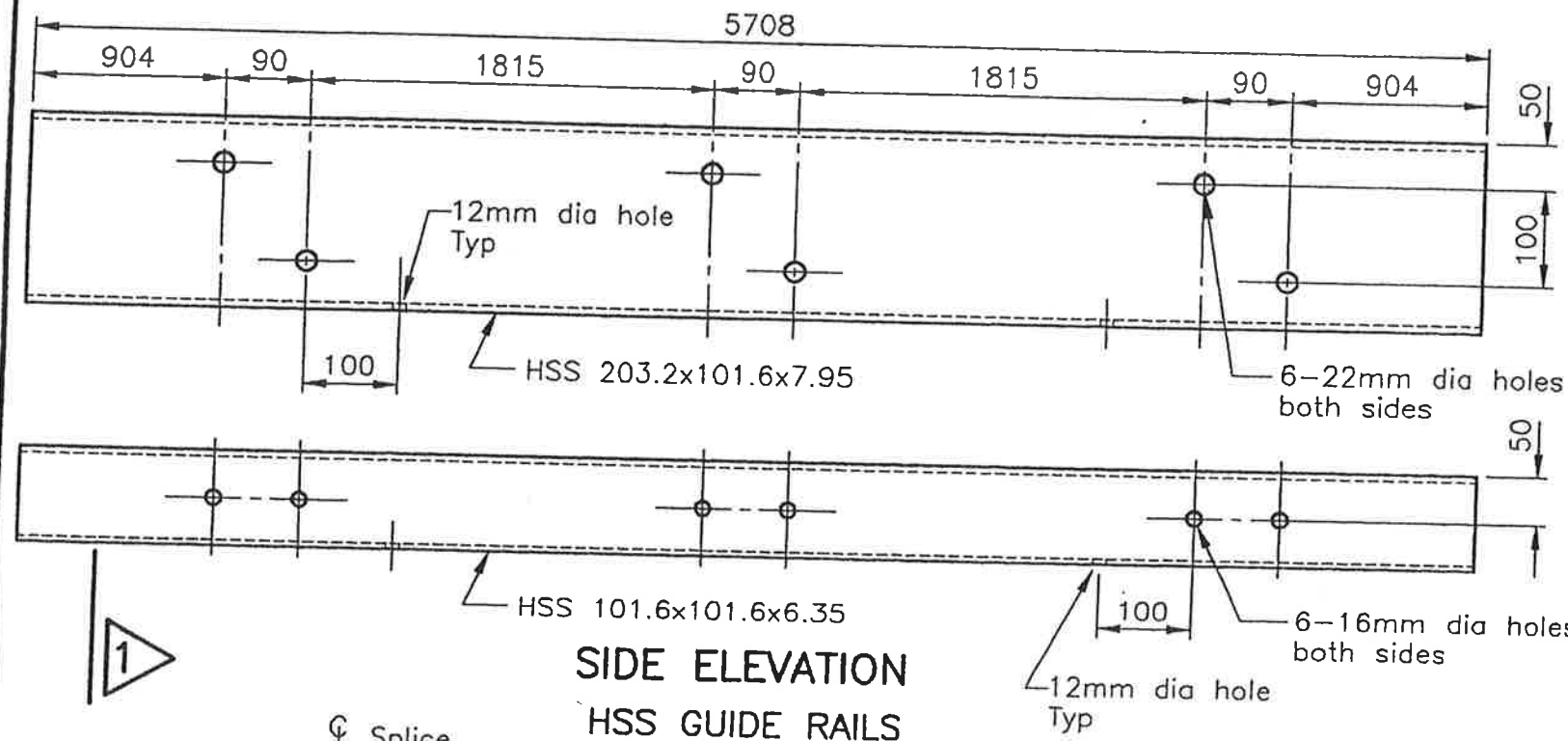


2-22mm dia holes
2-16mm dia holes

4-1" dia high strength anchor rods threaded both ends (with 2 hex nuts and 1 washer per rod)
2-20mm anchor plates

- NOTES:**
- 1 This dimension to be not less than 600mm for 340mm anchor bolt embedment.
 - A This Standard to be read in conjunction with SS10-50A and SS10-50C.
 - B Class of concrete: 30MPa.
 - C W150x37 shall conform to CSA Standard G40.21-M92 Grade 300W.
 - D Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
 - E Anchor rods shall conform to ASTM Specification A449. They shall be fitted with matching heavy hex nuts and washers.
 - F The anchorage assembly including bolts, nuts and washers shall be hot dip galvanized in conformance to CSA Standard G164-M92.
 - G All dimensions are in millimetres or metres unless otherwise shown.

BOX BEAM GUIDE RAIL ON CURB POST AND BASE DETAIL PERFORMANCE LEVEL 2		Date	JUNE 1993	Rev	
		SS10-50B			



NOTES:

- A This Standard to be read in conjunction with SS10-50A and SS10-50B.
- B HSS guide rails shall conform to CSA Standard CAN3-G40.21-M92 Grade 350W.
- C Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
- D Holes in HSS guide rails may be drilled in the field.
- E Welding shall conform to CSA Standard W59-M89 and shall be done by a welder qualified under CSA Standard W47-M83.
- F HSS guide rails, rail splices and fasteners shall be hot dip galvanized in conformance to CSA Standard G164-M92.
- G All dimensions are in millimetres or metres unless otherwise shown.

**BOX BEAM GUIDE RAIL ON CURB
RAIL AND SPLICE DETAIL
PERFORMANCE LEVEL 2**

Date	JUNE 1993	Rev
SS10-50C		

C Appendix C - WorkSafe BC, Ontario, Alaska, Washington and Oregon OHS Regulations for Forestry Operations



WorkSafeBC OHS Regulation Part 26: Forestry Operations and Similar Activities

Roads and Road Maintenance

26.79 Haul road standards

Roads, bridges, elevated platforms, and other structures used by vehicles transporting workers, logs or other forest products in forestry operations must be constructed and maintained to a standard which will permit safe transit.

26.80 Creating additional hazards

Road or skid trail construction, including any blasting activity, must be carried out in a manner that prevents hangups, hanging broken tops or limbs, leaners, sidebind of pushed trees, or similar hazards which could endanger fallers or other workers.

26.81 Bull rails

The open sides of bridges, elevated truck weigh scales and associated elevated ramp approaches, and other elevated structures used by logging trucks must be equipped with substantial and well secured continuous timber or log curbs or bull rails of sufficient height to prevent vehicles from running off the structure, but not less than 25 cm (10 in).

[Amended by B.C. Reg. 312/2003, effective October 29, 2003.]

26.82 Roadside hazards

(1) Dangerous trees, loose rocks, stumps, or other unstable materials that are hazardous to road users must be removed or cleared for a safe distance back from roadsides or roadside banks.

(2) Brush, foliage or debris which prevents an adequate view by a vehicle operator of traffic approaching at roadway intersections or on sharp curves must be cleared and all possible precautions must otherwise be taken to control the hazards created by limited sight distance.

26.83 Traffic control systems

(1) When 2 or more vehicles are using a section of a road which is too narrow to permit them to pass, an effective traffic control system must be used by all vehicles on the road.

(2) The traffic control system must include

(a) turnouts, where required,

(b) vehicles operating with their headlights and, if fitted, flashing beacons, turned on,

(c) warning signs where required, and

(d) instructional signs, including kilometre and road name/number signs, and the radio frequency for traffic control if one is being used.

26.84 Weigh scales

(1) Repealed. [B.C. Reg. 312/2003, effective October 29, 2003.]

(2) Weight recording house structures, forming part of a log transporter weigh scale unit, must

(a) be sufficiently offset from the scale balance platform to provide an adequate margin for log load clearance, or

(b) have an effective barrier erected between the weigh scale deck and the house.



**State of Alaska
Occupational Safety and Health**

Additional Logging Standards

Adopted by Reference Under
8 AAC 61.1060

Bridge Curb Minimum: Page 43



**ALASKA DEPARTMENT OF LABOR
& WORKFORCE DEVELOPMENT**

Good, Safe Jobs Are Alaska's Future

(G) Shear rails shall be installed on both outside edges of bridges. The shear rails shall be made of substantial material securely fastened, capable of withstanding the impact of the wheels of a loaded vehicle. The top of the shear rails shall be not less than 15 inches above the bridge surface.

(H) Control measures shall be instituted to minimize the generation of dust on logging roads so that visibility will permit safe operation of vehicles.

(8) Drivers. No person shall move a truck into a landing except on signal from a designated member of the loading crew who shall assure that all persons are in a safe position.

(p) Log Dumps, Booms and Rafts. (1) General

(A) In operations where regular logging machinery, rigging, etc., is used, the applicable subsections of this section and subsections of 29 C.F.R. 1910.266 shall apply.

(B) Where a single or multiple log dump approach extends into open water, a plank walk no less than three feet wide and hand rail shall be provided on one side of the trestle for its entire length and kept in good repair.

(C) Employees shall not attach lines for dumping or detach binders while the truck is in motion.

(D) Unloading lines shall be so arranged that it is not necessary for employees to attach them on the pond or dump side of the load.

(E) All decks and plankways on log dumps shall be kept in good repair and free from bark and other debris. Roadways shall not be inclined more than one inch to 12 inches.

(F) Unless they will support the weight of at least three persons, the use of small bridge-over logs, planking or timbers between regular foot logs or walkways is prohibited. All regular foot logs shall be barked on the upper side.

(G) One employee working alone on any log dump while logs are being unloaded is prohibited.

WAC 296-54-531 (Cont.)

- (e) A record maintained of each inspection must be available to a representative of the department on request.
- (5) Shear rails must be installed on both outside edges of bridges. The shear rails must be securely fastened and made of material able to withstand the impact generated by contact with the wheels of a loaded vehicle. The top of shear rails must be at least fifteen inches above the bridge surface. Bridges in use before 1980 with outside shear rails a minimum of ten inches high or center shear rails at least five inches high are permissible until repairs are needed.
- (6) The employer must implement measures that minimize dust to the degree that visibility is sufficient to allow an operator to safely operate a vehicle. Vehicle operators must travel at a speed consistent with road conditions.
- (7) Pneumatic-tired equipment must have fenders as described in the Society of Automotive Engineers Technical Report J321a.
- (8) Employee(s) must be assigned to flag on roads or provide other equivalent protection where hazardous conditions are created from logging such as but not limited to:
 - (a) Running wire rope lines or rigging across road grades, excluding guylines and standing skylines if lines remain a safe distance above the road to allow a vehicle to pass under; or
 - (b) The movement of logs, chunks, or debris across or suspended over road grades.

Exception: Where there is no through traffic, such as on a dead end road or where the property owner's permission or proper authority is granted to close a section of road, warning signs and barricades may be used instead of flagger(s).

[Statutory Authority: RCW 49.17.010, .040, .050 99-17-117, (Order 97-09), § 296-54-531, filed 08/18/99, effective 12/01/99. Statutory Authority: RCW 49.17.040, .050, .060 96-22-013, § 296-54-531, filed 10/28/96, effective 1/1/97. Statutory Authority: RCW 49.17.040, .050, and .240, chapters 43.22 and 42.30 RCW. 80-11-057 (Order 80-15), § 296-54-531, filed 8/20/80. Statutory Authority: RCW 49.17.040, .150, and .240. 79-10-081 (Order 79-14), § 296-54-531, filed 9/21/79.]

WAC 296-54-533 Road pioneering and earthwork.

- (1) Banks at the borrow area must be sloped to prevent slides.
- (2) Backfill must be firmly compacted.
- (3) Roadside banks must be sloped or stabilized to prevent slides.
- (4) Overhanging banks, large rocks and debris must be removed or secured.
- (5) Where riprap is used, the material and design must ensure containment of material.
- (6) Trees or snags that may fall into the road must be fell.
- (7) Root wads, logs, and other unstable debris must not be placed against standing timber or otherwise placed so as to create a hazard for timber falling or other logging operations.

[Statutory Authority: RCW 49.17.010, .040, .050 99-17-117, (Order 97-09), § 296-54-533, filed 08/18/99, effective 12/01/99. Statutory Authority: RCW 49.17.040, .150, and .240. 79-10-081 (Order 79-14), § 296-54-533, filed 9/21/79.]

WAC 296-54-535 Hand and portable powered tools.

- (1) Each hand and portable powered tool, including any tool provided by an employee, must be maintained in serviceable condition.

437-007-0505 Bridges.

(1) All bridge structures and surfaces must be:

- (a) Adequate to support the maximum imposed loads.
- (b) Maintained in good repair.

(2) All bridges must have rub rails constructed of wood, concrete or equivalent materials that:

- (a) Have a minimum height of 9 inches (6-inch by 6-inch timbers set on 4-inch by 6-inch blocks).
- (b) Are secured to the bridge deck.

Stat. Auth.: ORS 654.025(2) and 656.726(4).

Stats. Implemented: ORS 654.001 through 654.295.

Hist: OR-OSHA Admin. Order 5-2003, f. 6/02/03, ef. 12/01/03.

437-007-0510 Flagging.

(1) Warning signs and a flagger(s) must be placed in advance of active operations, or other equivalent protection must be used on roads to control traffic where hazardous conditions are created from forest activities, such as, but not limited to:

- (a) Skylines and running lines or rigging across road grades, excluding tightened guylines.
- (b) The movement of logs, chunks or debris across or suspended over road grades.
- (c) Timber cutting operations.
- (d) Helicopter logging operations.

NOTE: Where there is no through traffic, such as on a dead end road or where the property owner's permission or proper authority is granted to close a section of road, warning signs and barricades may be used instead of flagger(s).

(2) Flaggers must wear vests of a high-visibility color and use a minimum 18-inch x 18-inch "STOP/SLOW" paddle to control traffic.

(3) Warning signs and flagging activities along state and county roads must comply with the requirements of the Millennium Edition of the (FHWA) Manual of Uniform Traffic Control Devices (MUTCD), December 2000.

Stat. Auth.: ORS 654.025(2) and 656.726(4).

Stats. Implemented: ORS 654.001 through 654.295.

Hist: OR-OSHA Admin. Order 5-2003, f. 6/02/03, ef. 12/01/03.



Occupational Health and Safety Act

R.R.O. 1990, REGULATION 851

INDUSTRIAL ESTABLISHMENTS

A bridge on a haul road shall,

- (a) be structurally adequate to support any load likely to be applied to it;
- (b) have curbs of a height of not less than fifteen centimetres on each side of the travelled portion of the bridge;
- (c) be of sufficient width between curbs to permit the passage of vehicles using the bridge; and
- (d) have markers which clearly indicate the width and ends of the bridge.

D Appendix D - US Forest Service Warrant for a Curb-only Barrier System Form

ROAD NUMBER - MILEPOST
BRIDGE NAME

Proposed structure length	xx m	Height above channel	xx m
Traffic Service Level	D	Road Maintenance Level – Operating	2
Bridge meets NBIS Standards	Yes/No	Road Maintenance Level – Objective	2

From FSH 7709.56b Transportation Structures Handbook:

7.24-5. Exceptions to using bridge railings. Consider omitting railing and installing a curb-only system for bridges at low hazard sites that experience low-volumes of low-speed traffic, and when object markers can provide sufficient advance warning of the bridge. Base the decision to install a protective railing system on an analysis that evaluates the specific site conditions, such as the amount, type, and speed of traffic, and the hazards involved.

Consider omitting the bridge railing under the following conditions:

- a. The road is used at low speeds by drivers generally familiar with the road.
 - o *Describe traffic speed and type of users. For example: Traffic is primarily low speed. The majority is administrative, logging, or logging associated or residents of nearby communities.*
- b. The road is not heavily used.
 - o *Describe traffic patterns. For example: Traffic is light. It is not a thru road, and does not provide access to any special recreation attractors. Primary recreation use is hunting and firewood gathering.*
- c. The bridge is relatively short and located on a tangent.
 - o *Describe bridge and approach geometry. For example: Expected length is approximately xx meters. It is located on a tangent, no curve widening is necessary.*
- d. The bridge width is equal to the traveled way plus shoulders.
 - o *Yes or no, the bridge width provides for the traveled way plus required curve widening.*
- e. Separation is provided for vehicles and sidewalk pedestrians.
 - o *Describe pedestrian accommodations, if needed. For example: There will not be a sidewalk. Pedestrian use will be almost non-existent.*
- f. Hazards created by the unrailed bridge are not unusual in comparison to the exposure presented by the rest of the road.
 - o *Describe road hazards along the rest of the road, such as side slopes, distance downhill to a slope break, etc. For example: A 380 mm (15-inch) high curb would result in a bridge hazard similar to hazards associated with driving the remainder of the road. Estimated height of the roadway above the channel is less than 3 meters. Approach fill slopes will be approximately 1.25 to 1. Typical fill slopes on forest roads are 1.25 to 1 and often higher than 3 meters.*
- g. Curbs are provided for loadings that conform to AASHTO specifications.
 - o *Yes or no, design is in accordance with AASHTO.*
- h. Object markers outline the bridge and mark the curbs. Place object markers in accordance with Forest Service Guide for Traffic Control Devices and the Manual of Uniform Traffic Control Devices.
 - o *Describe object markers. For example: Object markers will be installed as part of the bridge construction contract in accordance with Forest Service and AASHTO requirements.*

Based on these conditions, it is my decision to not require bridge and approach guardrail.

Forest Engineer, _____ National Forest

Date

E Appendix E - Barrier Crash Test Reports



Railing Systems for Longitudinal Timber Deck Bridges

Ronald K. Faller and Barry T. Rosson, Midwest Roadside Safety Facility,
University of Nebraska-Lincoln

Michael A. Ritter and Paula D. Hilbrich Lee, USDA Forest Service,
Forest Products Laboratory

Sheila R. Duwadi, Federal Highway Administration

Abstract

Bridge railing systems in the United States have historically been designed based on static load criteria given in the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges*. In the past decade, full-scale vehicle crash testing has been recognized as a more appropriate and reliable method of evaluating bridge railing acceptability. In 1989, AASHTO published the *Guide Specifications for Bridge Railings* which gives recommendations and procedures to evaluate railings by full-scale vehicle crash testing. In 1993, the National Cooperative Highway Research Program (NCHRP) published *Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features*, which provides criteria for evaluating longitudinal barriers. Based on these specifications, a cooperative research program was initiated between the University of Nebraska-Lincoln and the Forest Products Laboratory, and later the Federal Highway Administration, to develop and crash test several bridge railings for longitudinal wood decks. This paper describes the successful development and testing of nine resulting railing systems in accordance with the AASHTO Performance Level 1 and 2 (PL-1 and PL-2) requirements, and the Test Level 1 and 4 (TL-1 and TL-4) requirements of *NCHRP Report 350*.

Keywords: Timber Bridges, Bridge Rail, Crash Testing, Roadside Safety, and Longitudinal Barrier.

Introduction

The primary purpose of a bridge railing is to safely contain errant vehicles crossing a bridge. To meet this objective, railings must be designed to withstand the force of an impacting vehicle. In designing railing systems for highway bridges, engineers have traditionally assumed that vehicle impact forces can be approximated by equivalent static loads that are applied to railing elements. Although rail loads are actually dynamic, the equivalent static load method has been used for many years as a simplified approach to standardized railing design. Until recently, the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (1) required that rail posts be designed to resist an outward transverse static load of 44.5 kN (10,000 lb). A portion of this load was also applied to posts in the inward transverse, longitudinal, and vertical directions and to the rail elements. These requirements were identical for all bridges regardless of bridge geometry or traffic conditions. Thus, a railing for a single-lane bridge located on a low-volume road was required to meet the same loading requirements as a railing for a bridge located on a major highway.

Despite the widespread use of design requirements based primarily on static load criteria, the need for more appropriate full-scale vehicle crash test criteria has long been recognized. The first U.S. guidelines for full-scale vehicle crash testing were published in 1962 (2). This initial 1-page document provided basic guidelines for the test vehicle mass, approach speed, and impact angle and served to provide a degree of uniformity to the traffic barrier research in progress at the time. Through subsequent use of this document, the need for more comprehensive guidelines became apparent, and several reports were published during the 1970s through the National Cooperative Highway Research Program (NCHRP). In 1981, NCHRP published *Report 230, Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances* (3). This comprehensive report provided recommendations relative to crash testing and evaluation of longitudinal barriers and served as the basis for future bridge rail crash testing requirements.

Although crash test criteria have been available for many years, the requirement to implement crash testing as a means of evaluating bridge railings in the United States has been jurisdiction dependent. Some states implemented extensive bridge rail crash testing programs, while others continued to exclusively use static load design. The first recognition of full-scale crash testing in a national bridge specification came in 1989 when AASHTO published *Guide Specifications for Bridge Railings* (4). This specification presents recommendations for the development, testing, and use of crash-tested bridge railings and refers extensively to *NCHRP 230* for crash testing procedures and requirements.

A primary concept of the *AASHTO Guide Specifications* is that bridge railing performance needs differ greatly from site to site, and railing designs and costs should match site needs. Thus, recommended requirements for rail testing are based on three performance levels: Performance Level 1 (PL-1), Performance Level 2 (PL-2), and Performance Level 3 (PL-3). The PL-1 requirements represent the "weakest" system, and the PL-3 the "strongest" system. The relationship between the railing performance level and requirements for a specific bridge depend on a number of factors, such as the type of roadway, design speed, average daily traffic, and percentage of trucks in the traffic mix.

The recently published *NCHRP Report 350, Recommended Procedure for the Safety Performance Evaluation of Highway Features* (5), provides for six test

levels for evaluating longitudinal barriers; Test Level 1 (TL-1) through Test Level 6 (TL-6). Although this document does not include objective criteria for relating a Test Level to a specific roadway type, the lower test levels are generally intended for use on lower service level roadways and certain types of work zones while the higher test levels are intended for use on higher service level roadways.

In 1994, AASHTO published the *LRFD Bridge Design Specifications* (6) as an update to the *Standard Specifications for Highway Bridges* (1) and the *Guide Specifications for Bridge Railings* (4). For crash testing bridge railings, three performance levels were provided similar to those provided in the *Guide Specifications for Bridge Railings* (4). Guidelines for crash testing bridge railings followed procedures provided in both the *AASHTO Guide Specifications* and *NCHRP Report 350*. Yield line and inelastic analysis and design procedures, as originally developed by Hirsch (7), were also provided for bridge railings as a replacement to the 44.5 kN (10,000 lb) equivalent static load design procedures.

Emphasis on the use of crash-tested rails for new Federally funded projects has significantly increased the role of full-scale crash testing as a means of evaluating railing performance. Recently, the Federal Highway Administration (FHWA) officially adopted *NCHRP 350* as a replacement for *NCHRP 230* and has strongly suggested that AASHTO also adopt the test level definitions contained in *NCHRP 350*, thus making crash-tested railings mandatory for most bridges. Most highways on which wood bridges are installed will require railings that meet either the AASHTO PL-1 or PL-2 requirements, or the *NCHRP 350* TL-1 through TL-4 requirements. A railing that meets either PL-3, TL-5, or TL-6 requirements currently has a very limited application for wood bridges because of the high traffic volumes and speeds associated with these levels.

As of August 1990, 25 bridge rails had been successfully crash tested in accordance with the requirements of the *AASHTO Guide Specifications* and approved for use on Federal-aid projects by the FHWA (8). Of these crash-tested railings, 24 are for concrete bridge decks and one is for a wood deck. For wood bridges to be viable and competitive with other bridges in the future, a range of crash-tested bridge railings for different wood bridge types was required. Based on this need, National emphasis was placed on developing a limited number of crash-tested railings for wood bridges.

Background

To meet the need for crashworthy railings for wood bridges, the USDA Forest Service, Forest Products Laboratory (FPL), in cooperation with the Midwest Roadside Safety Facility (MwRSF) of the University of Nebraska-Lincoln, the FHWA, and the wood products industry initiated a program to develop crash-tested bridge rails for longitudinal wood decks. The program objectives were to develop a total of nine crashworthy rails: three to meet AASHTO PL-1, one to meet AASHTO PL-2, three to meet NCHRP 350 TL-1, one to meet NCHRP 350 TL-4, and one intended for very low performance conditions. The scope of the project was limited to railings for longitudinal wood decks, 252 mm (10 in.) or greater in thickness, and constructed of glued-laminated (glulam) timber, spike-laminated lumber, or stress-laminated lumber. In each system, the lumber laminations are placed edgewise and oriented with the lumber length parallel to the direction of traffic. A brief description of each longitudinal deck bridge type is provided in *Timber Bridges: Design, Construction, Inspection, and Maintenance* (9).

Longitudinal glulam timber decks are constructed of panels that consist of individual lumber laminations glued together with waterproof structural adhesives. The panels are 1.07 to 1.38 m (3.5 to 4.5 ft) wide and effectively function as a large, solid block of wood. To form the bridge deck, panels are placed side by side and are interconnected by transverse distributor beams bolted to the deck underside at intervals of 2.4 m (8 ft) or less. These distributor beams are designed to transfer vertical loads between adjacent panels. They are not designed to resist lateral loads.

Spike-laminated decks are constructed of sawn lumber laminations 102 mm (4 in.) in nominal thickness. The individual laminations are interconnected with spikes that are typically 8 or 9.5 mm (0.3125 or 0.375 in.) in diameter and 356 to 406 mm (14 to 16 in.) long. The decks are commonly manufactured in panels that are 1.5 to 2.1 m (5 to 7 ft) wide and interconnected with transverse distributor beams in a manner similar to longitudinal glulam timber decks.

Stress-laminated decks are constructed of sawn lumber laminations that are typically 51 to 102 mm (2 to 4 in.) in nominal thickness. The laminations are stressed together with high strength steel bars that are placed through holes drilled through the center of the wide faces of the laminations. When tensioned, the bars create compression between the laminations, and the entire deck effectively acts as a solid, orthotropic wood plate.

Test Requirements and Evaluation Criteria

The test requirements and evaluation criteria for this project followed procedures defined in the *AASHTO Guide Specifications* (including applicable references to *NCHRP 230*) and the *NCHRP 350* criteria. These procedures establish a uniform methodology for testing and evaluating railings so that the safety performance of different railing designs, tested and evaluated by different agencies, can be compared. It is impractical and impossible to test all railings for all possible vehicle and impact conditions. Therefore, the procedures specify a limited number of tests using severe vehicle impact conditions and a set of evaluation criteria against which test results may be evaluated.

Test Requirements

Vehicle impact requirements for rail crash testing depend on the railing performance level/test level and are specified as requirements for vehicle type and weight, impact speed, and impact angle relative to the longitudinal rail axis. Testing for PL-1 and TL-1 requires two vehicle impact tests while testing for PL-2 and TL-4 requires three vehicle impact tests. A summary of the requirements for PL-1, PL-2, TL-1, and TL-4 are shown in Table 1. In some cases, all tests for a given level may not be required if a railing with similar geometry and strength was previously tested and found to be satisfactory.

In addition to vehicle impact requirements, the *AASHTO Guide Specifications* and the *NCHRP 350* criteria also specify requirements for data acquisition and construction of the bridge railing. Requirements for data acquisition are referenced to *NCHRP 230* and *NCHRP 350* and include specific data collection parameters and techniques that must be completed before, during, and after the crash test. Construction requirements specify that the bridge rail be designed, constructed, erected, and tested in a manner representative of actual installations. To properly assess the performance of most bridge rails, they must also be evaluated as a system in combination with the bridge superstructure for which it is intended. This is very important when considering rails for wood bridges because the attachment of the rail to the bridge deck and the ability of the wood superstructure to resist applied loads may often be the controlling parameters.

Evaluation Criteria

Evaluation criteria for full-scale crash testing is based on three appraisal areas: structural adequacy, occupant risk, and vehicle trajectory after the collision. Criteria for structural adequacy are intended to evaluate the ability of the railing to contain, redirect, or permit

Table 1 – Vehicle impact requirements for AASHTO PL-1 and PL-2 and NCHRP 350 TL-4 bridge railings.

AASHTO Performance Level (4)	Impact Conditions		
	Small Car (816 kg)	Pickup Truck (2,449 kg)	Single-Unit Van Truck (8,165 kg)
1	80.5 km/h 20 deg	72.4 km/h 20 deg	
2	96.6 km/h 20 deg	96.6 km/h 20 deg	80.5 km/h 15 deg

NCHRP 350 Test Level (5)	Impact Conditions		
	Small Car (820 kg)	Pickup Truck (2,000 kg)	Single-Unit Van Truck (8,000 kg)
1	50 km/h 20 deg	50 km/h 25 deg	
4	100 km/h 20 deg	100 km/h 25 deg	80 km/h 15 deg

controlled vehicle penetration in a predictable manner. Occupant risk evaluates the degree of hazard to occupants of the impacting vehicle. Vehicle trajectory after the collision is concerned with the path and final position of the impacting vehicle and the probable involvement of the impacting vehicle with other traffic. Note that these criteria address only the safety and dynamic performance of the railing and do not include service criteria such as aesthetics, economics, bridge damage, or post-impact maintenance requirements. The following evacuation criteria are summarized from the *AASHTO Guide Specifications* for PL-1 and PL-2 testing (similar evaluation criteria are provided in *NCHRP 350*):

1. The railing shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the railing is acceptable.
2. Detached elements, fragments, or other debris from the railing shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
3. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.

4. The vehicle shall remain upright during and after collision.

5. The railing shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle does not yaw more than 5 degrees away from the railing from time of impact until the vehicle separates from the railing.

6. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction where = 0.0 to 0.25 is good, = 0.26 to 0.35 is fair, and > 0.36 is marginal. Requirements for computing are given in the *AASHTO Guide Specifications*.

7. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 610-mm (2-ft) longitudinal and 305-mm (1-ft) lateral displacements, shall be less than 9.15 m/s (30 ft/s) in the longitudinal direction and 7.63 m/s (25 ft/s) in the lateral direction. In addition, the vehicle highest 10-ms average accelerations subsequent to the instant of hypothetical passenger impact should be less than 147 m/s² (483 ft/s²) in the longitudinal and lateral directions.

8. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 30.5m (100 ft) plus the length of the test vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 6.1 m (20 ft) from the line of the traffic face of the railing.

Development Phase

Based on a fundamental understanding of the performance characteristics of each deck type, development work was initiated to formulate a methodology for the railing tests. From the standpoint of economics and time, it was considered impractical to develop and test different rail systems for each longitudinal deck type. Rather, a more feasible approach was undertaken to develop several railing systems that could be adapted to each of the three longitudinal deck types without modifications that would result in reduced performance. To accomplish this, it was determined that railing development and testing should utilize the weakest deck type. This conclusion was based on the premise that if successful tests could be completed on the weakest deck, the railing could be adapted for use on stronger decks without adversely affecting performance.

In assessing the potential resistance of each deck type to transverse railing impact forces, consideration was

given to the strength of the wood and mechanical reinforcement. Of primary concern was loading that could introduce tension perpendicular to grain stress in the wood deck.

Of the three deck types, the stress-laminated deck was considered the strongest for transverse railing loads, because the high strength steel bars are continuous across the deck width. Loads developed at vehicle impact can be effectively distributed across the deck by the bars, making the entire deck width effective in resisting the applied loads.

The spike-laminated deck was considered to be of intermediate strength. If rail loads are applied transverse to the panel length, the loads are resisted by the spikes in withdrawal. Because of this, tension perpendicular to grain in the lumber laminations is not a concern; however, the spikes could be pulled from the deck resulting in longitudinal separations between the laminations, and additional reinforcement could be required.

The glulam timber deck was considered to be the weakest in resisting railing loads, because the glulam timber panels act as solid pieces of wood, and loads applied transverse to the panel length are most likely to introduce tension perpendicular to grain and failure in the upper panel section. Mechanical reinforcement was considered necessary for longitudinal glulam timber decks to resist railing loads without damage. Thus, the glulam timber deck was considered the weakest deck for transverse railing loads and was selected for full-scale crash testing. If bridge railings performed acceptably on the glulam timber system, it was rationalized that the railings could be adapted to other longitudinal wood bridge decks with no reduction in railing performance.

The primary emphasis of the railing design process was to develop rails that would meet the requirements of the *AASHTO Guide Specifications* and *NCHRP 350*. Additionally, it was determined that consideration should be given to (1) extent of probable damage to the structure after vehicle impact and the difficulty and cost of required repairs; (2) adaptability of the railing to different wood deck types; (3) rail system cost to the user, including material, fabrication, and construction; (4) ease of railing construction and maintenance; and (5) aesthetics.

The conclusion of the development phase involved the design of several railing systems and preparation of plans and specifications for testing. The selection and

design of these final systems were based on a review of other railings that had been successfully crash tested, as well as those that are currently used on wood bridges but had not been crash tested. To the extent possible, feasible designs were evaluated using computer simulation models. Although several proven computer models were used, it was difficult to adapt the programs for wood components because the behavior and properties of the wood systems at ultimate loading were unknown. Data collected during the crash testing were used to refine input parameters and to more accurately predict railing performance in subsequent tests.

Test Methodology

Testing of all bridge rails was conducted at the Midwest Roadside Safety Facility in Lincoln, Nebraska. The site is located at an airport and was formerly a taxiway and parking area for military aircraft. It includes approximately 11 ha (27 acres) of concrete pavement and 1.6 ha (4 acres) of soil surface. To perform the rail testing, a test bridge was constructed that measured approximately 2.4 m (8 ft) wide and 28.6 m (93.75 ft) long, in five simply-supported spans measuring 5.72 m (18.75 ft) each. The deck was constructed of 273-mm (10.75 -in.) thick glulam timber panels, 1.2 m (4 ft) wide. The glulam timber for the deck was Combination No. 2 Douglas Fir given in the *AASHTO Standard Specifications for Highway Bridges* (1) and was treated with pentachlorophenol in heavy oil in accordance with American Wood Preservers' Association (AWPA) Standard C14 (10). Two glulam timber panels were placed side by side to achieve the 2.4-m (8-ft) width, and transverse distributor beams were attached to the deck underside per AASHTO requirements (1). The test bridge was supported by concrete footings that were placed in excavations so that the top of the test bridge was level with the concrete surface at the site.

Vehicle propulsion and guidance were provided by steel cable configurations. For propulsion, a reverse cable tow with a 1:2 mechanical advantage was used. A cable was attached to the front of the vehicle, routed through a series of pulleys, and was connected to a tow vehicle that traveled in a direction opposite to the test vehicle. The unoccupied test vehicle was then pulled by the tow vehicle and released from the tow cable approximately 9.2 m (30 ft) prior to impact. A vehicle guidance system developed by Hinch was used to steer the test vehicle (11). Using this system, the the left front wheel hub is attached to a tensioned steel cable that maintains the vehicle's direction along a designated straight path. Approximately 9.2 m (30

ft) from impact, the guidance connection is sheared off and the vehicle separates from the guidance cable.

Data acquisition parameters and techniques for the crash testing program were based on requirements of the *AASHTO Guide Specifications* and *NCHRP 350* and followed three testing phases: pretest, test, and post-test. In the pretest phase, the as-built bridge rail and vehicle were documented using photography and drawings that indicated the applicable configuration, dimensions, and vehicle weight. During the test phase, data regarding the vehicle impact speed, impact angle, trajectory, and accelerations were collected primarily through the use of high-speed motion picture photography and accelerometers mounted on the vehicle. In the post-test phase, the condition of both the railing, bridge superstructure, and vehicle were documented using photography and standardized damage assessment methods, including the Traffic Accident Data Scale (12) and Vehicle Damage Index (13). Additional instrumentation was placed on some railings to assess vehicle impact forces transmitted to the bridge rail and superstructure (14).

Successfully Tested Railings

As a result of the development and testing program, nine bridge railings were successfully developed and tested for longitudinal wood decks. Three of the railings were tested at PL-1, one was tested at PL-2, three were tested to TL-1, one was tested at TL-4, and one was tested primarily for low-volume forest road applications at impact conditions less than TL-1. Each railing was tested on the glulam timber deck and is adaptable to the spike-laminated and stress-laminated decks. All the PL-1, PL-2, and TL-4 designs employed posts spaced 1.9 m (6.25 ft) on-center and utilized high strength steel bars through a portion of the bridge deck to act as reinforcement in distributing railing loads without damage to the bridge. Two of the three TL-1 designs employed posts spaced 1.9 m (6.25 ft) on-center, while the third TL-1 design was a curb railing configured with scuppers spaced 3.0 m (10 ft) on-center. The railing design for conditions less than TL-1 consisted of a curb railing with scuppers spaced 2.9 m (9.5 ft) on-center. Glulam timber for the rail members was Combination No. 2 Douglas Fir as given in the *AASHTO Standard Specifications for Highway Bridges* (1), treated with pentachlorophenol in heavy oil to AWWA Standard C14 requirements (10). Sawn lumber for posts, curbs, scuppers, and spacer blocks was No. 1 Douglas Fir (1), treated with creosote to AWWA Standard C14 requirements (10).

A detailed discussion of the testing and results for each railing system is beyond the scope of this paper and, for most of the railing systems, is presented in detail in previous publications (14-18). Overall, no significant damage to the test bridge was evident from any of the vehicle impact tests. For the railing systems with glulam timber rails, damage to the railing was primarily gouging and scraping resulting from the vehicle impact. All glulam timber railing remained intact and serviceable after the tests and replacement of the railing was not considered necessary. For the steel thrie beam railings, there was permanent deformation in the rail and post in the vicinity of the impact location. This would necessitate replacement of specific railing and post members, but damage was relatively minor considering the severity of the impact. A brief description of each railing that met all test criteria follows.

PL-1 Railings

The three tested PL-1 railings included a glulam timber rail with curb, a glulam timber rail without curb, and a steel rail. Photographs and drawings of the PL-1 railings are shown in Figures 1 and 2, respectively.

The glulam timber rail with curb consisted of a single glulam timber rail mounted on a sawn lumber post. The post was connected with a single bolt to a lumber curb that was supported by scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts and timber connectors.

The glulam timber rail without curb consisted of a single glulam timber rail mounted on a sawn lumber post. The lower portion of the post was placed in a steel box that was attached to the bridge deck with high strength steel bars.

The steel rail consisted of a 10-gauge steel thrie beam rail mounted to a steel wide flange post. The lower end of the post was bolted to a steel plate that was connected to the bridge deck with high strength steel bars.

PL-2 Railing

The PL-2 railing included a steel rail and steel channel section, as shown in Figure 3. The steel rail was a modified version of that tested at PL-1. Minor changes in the railing geometry and the addition of a steel channel section above the rail were necessary to resist the increased loads at PL-2.

TL-1 Railings

The three tested TL-1 railings included a flexible steel rail, a semi-rigid steel rail, and a curb-type timber rail.

Photographs and drawings of the TL-1 railings are shown in Figures 4 and 5, respectively.

The flexible steel rail consisted of a 12-gauge W-beam rail mounted to a breakaway sawn lumber post. The lower end of the post was placed between two steel angles that were connected to the vertical edge of the bridge deck with lag screws.

The semi-rigid steel rail consisted of a 12-gauge W-beam rail mounted to a steel wide flange post. The post was bolted to a steel plate which was bolted to the bridge deck surface.

The low-height, curb-type timber rail was constructed with a glulam timber rail and supported with scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts and timber connectors.

TL-4 Railing

The TL-4 railing included a glulam timber rail with curb, as shown in Figure 6. The glulam timber rail with curb consisted of a single glulam timber rail mounted on a sawn lumber post and was a modification of the curb system tested at PL-1. Because of the greater loads at TL-4, rail and post sizes were increased and bolts and timber connectors attaching the curb and scupper to the bridge deck were increased.

Low Performance Railing

The low performance railing developed for impact conditions less than TL-1 included a low-height timber curb rail, as shown in Figure 7. Three geometries were considered for the curb rail — a square shape, a rectangular shape, and a trapezoidal shape. The curb rail was constructed with sawn lumber and supported with scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts.

Concluding Remarks

This program clearly demonstrates that crashworthy railing systems are feasible for longitudinal wood decks. Even at high-impact conditions required by AASHTO PL-2 and NCHRP 350 TL-4, the railing systems performed well with no significant damage to the bridge superstructure. With the development of crashworthy railing systems, a significant barrier to the use of longitudinal deck wood bridges has been overcome.

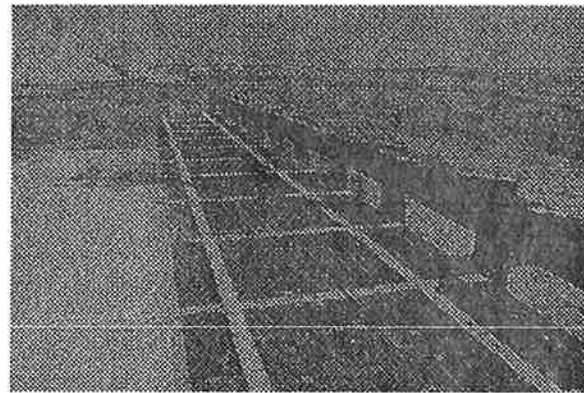
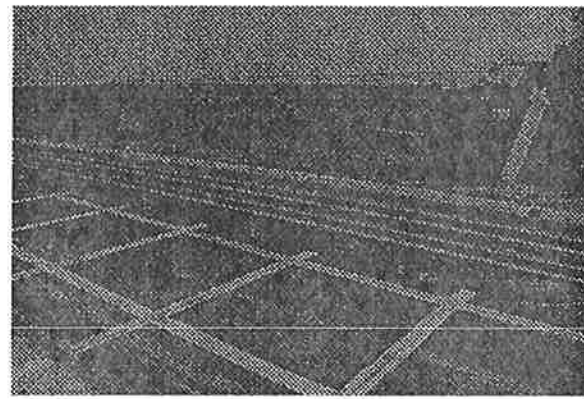
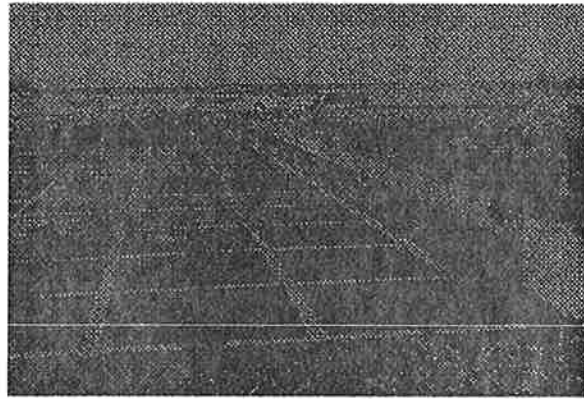
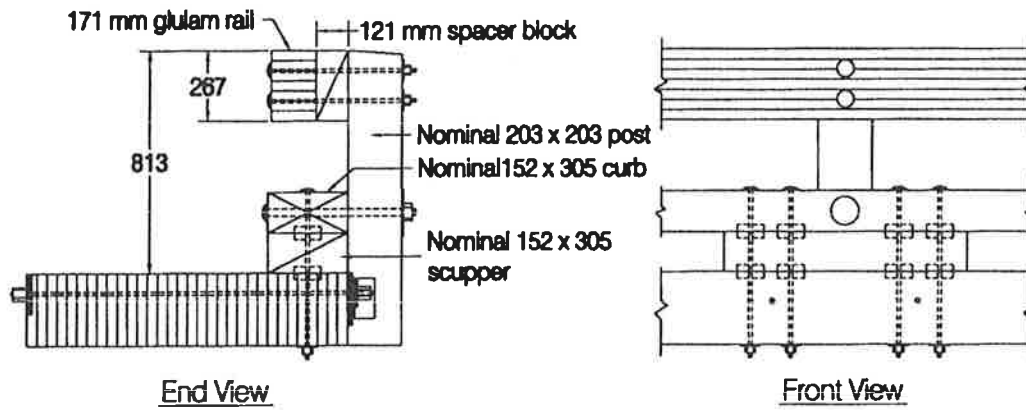
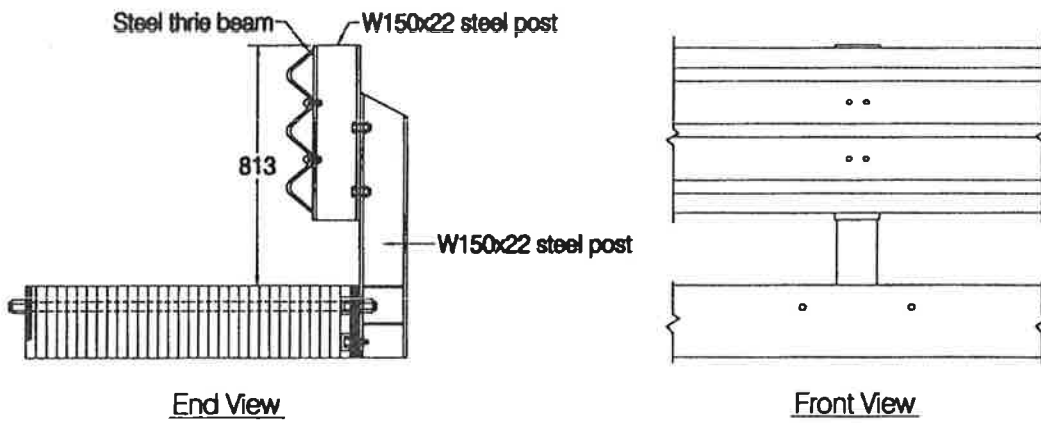


Figure 1 — Photographs of bridge railings successfully crash tested to AASHTO PL-1 (photographs taken prior to testing).

Glulam Timber Rail with Curb



Steel Rail



Glulam Timber Rail without Curb

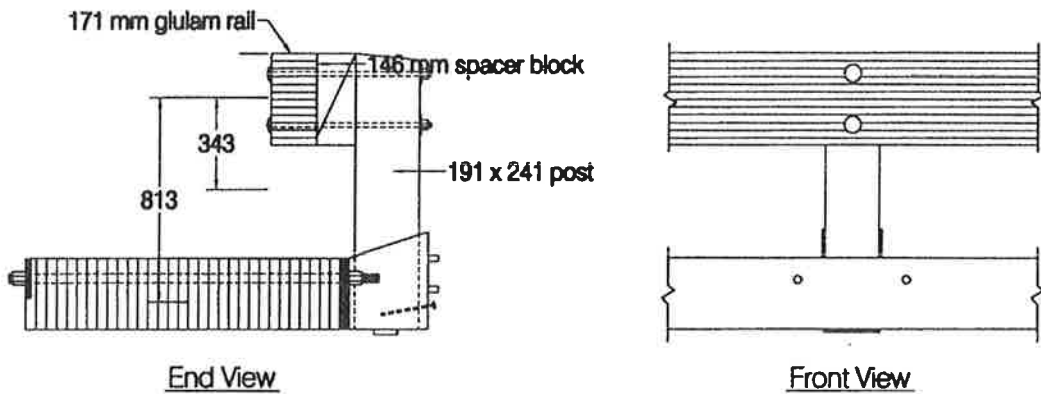


Figure 2 – Drawings of bridge railings successfully crash tested to AASHTO PL-1. Drawing units are millimeters

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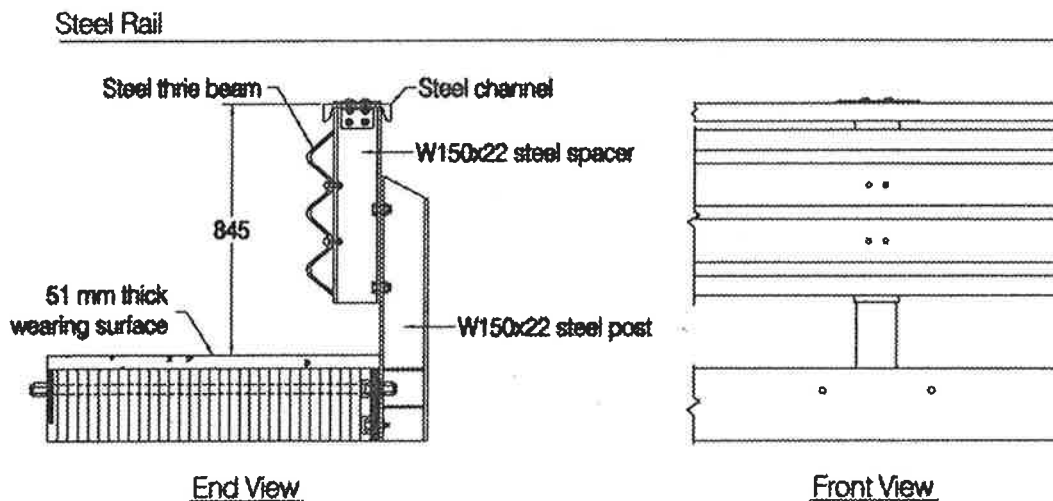
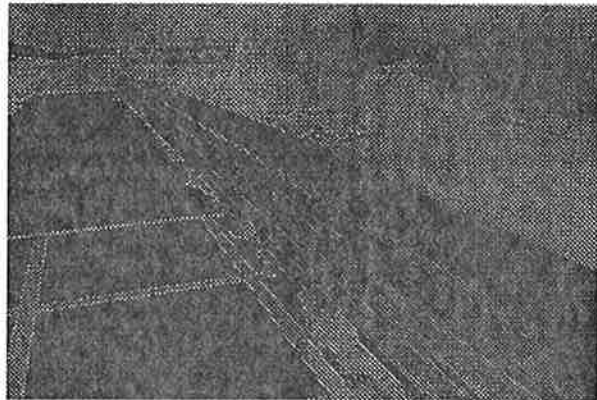


Figure 3 – Steel thrie beam bridge railing successfully crash tested to AASHTO PL-2 (photograph taken prior to testing). Drawing units are millimeters.

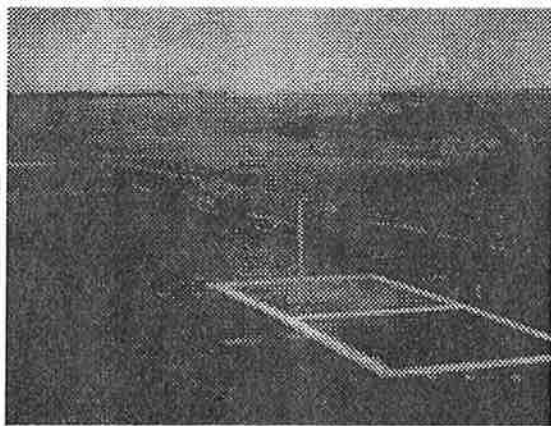
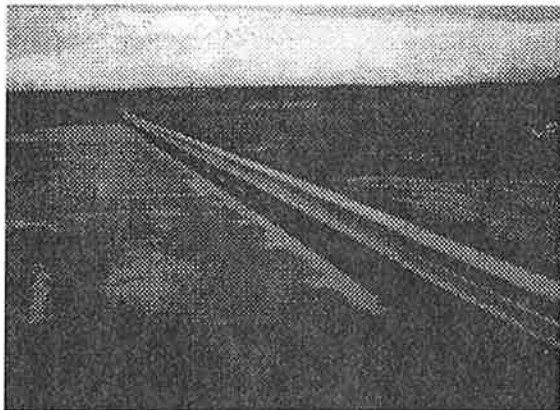


Figure 4 – Photographs of bridge railings successfully crash tested to NCHRP 350 TL-1 (photographs taken prior to testing).

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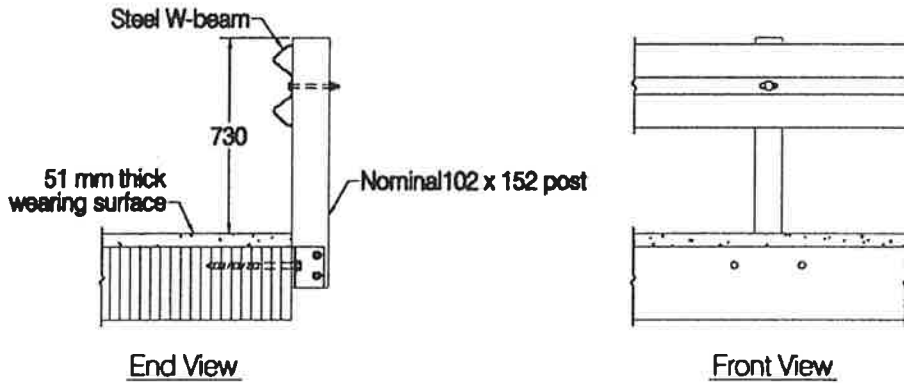
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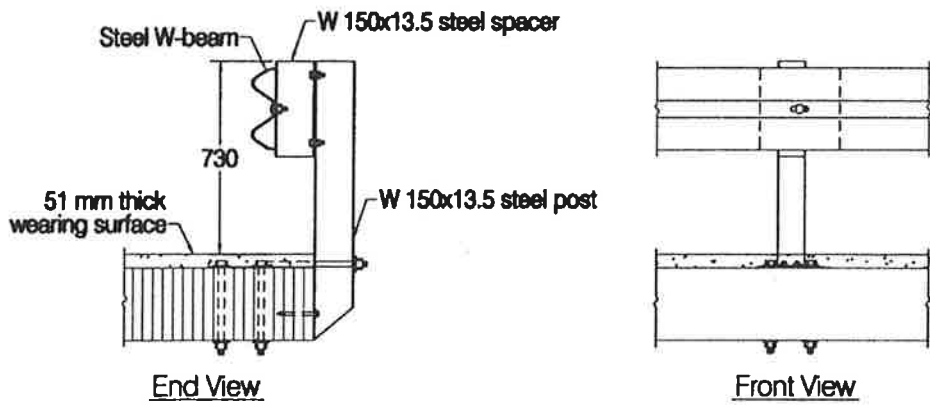
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Flexible Steel Rail



Semi-Rigid Steel Rail



Curb-type Rail

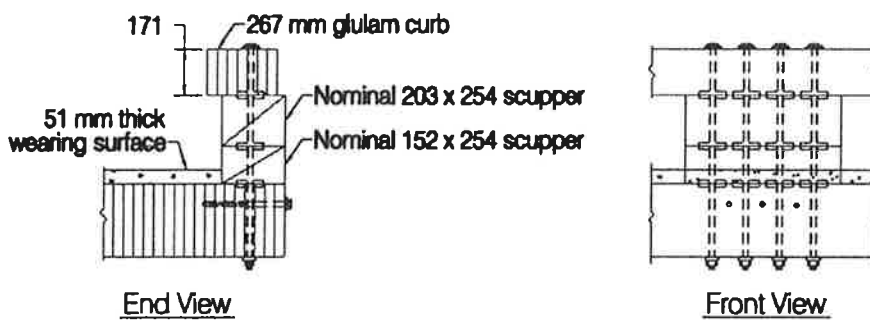


Figure 5 – Drawings of bridge railings successfully crash tested to NCHRP 350 TL-1. Drawing units are millimeters.



Glulam Timber Rail with Curb

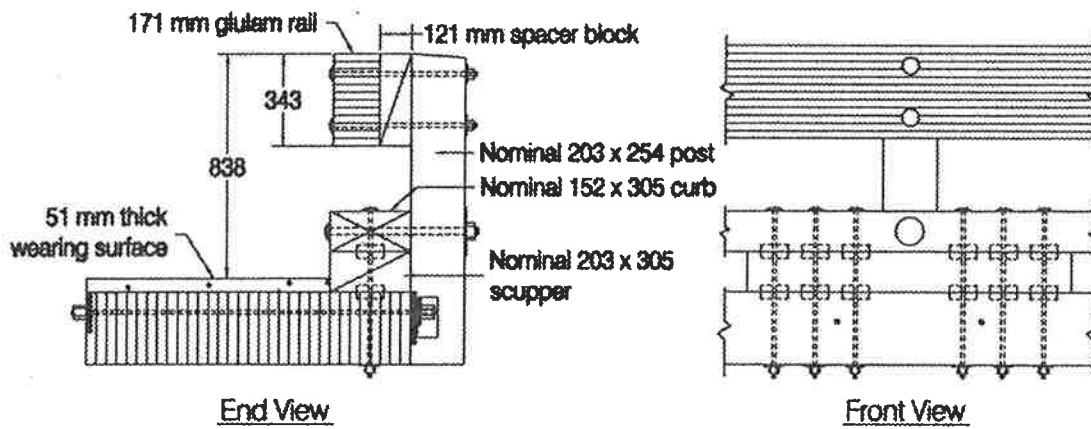


Figure 6 – Glulam timber bridge railing successfully crash tested to NCHRP 350 TL-4 (photograph taken prior to testing). Drawing units are millimeters.

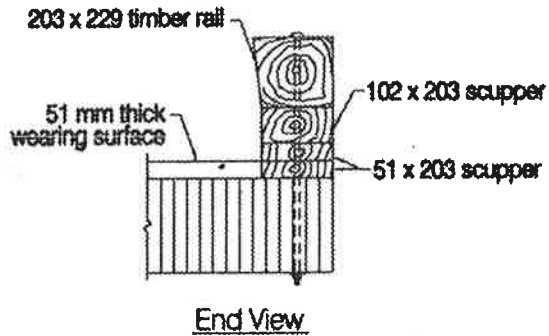
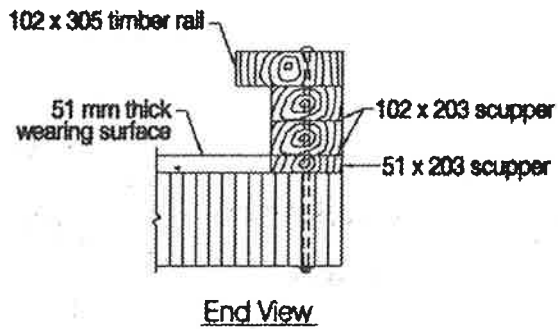
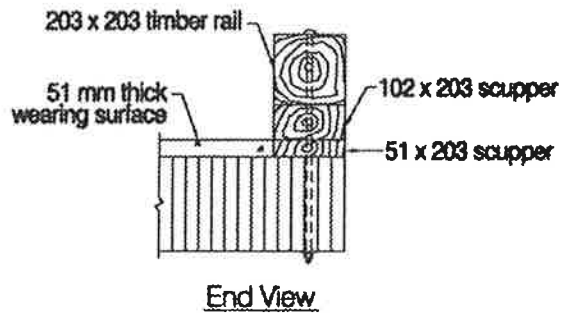


Figure 7 – Low performance bridge railing successfully crash tested with a 3/4-ton pickup truck at 15 mph and 15 degrees (photograph taken prior to testing). Drawing units are millimeters.

In: Ritter, M.A.; Duwadi, S.R.; Lee, P.D.H., ed(s). National conference on wood transportation structures; 1996 October 23-25; Madison, WI. Gen. Tech. Rep. FPL- GTR-94. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

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1995 June 25-29; Minneapolis, MN. Washington, DC: National Academy
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Design and Evaluation of Two Bridge Railings for Low-Volume Roads

Ronald K. Faller, Barry T. Rosson, and Dean L. Sicking, *University of Nebraska-Lincoln*

Michael A. Ritter, *Forest Products Laboratory, USDA Forest Service*
Steve Bunnell, *USDA Forest Service, Washington, D.C.*

The U.S. Department of Agriculture (USDA) Forest Service, Forest Products Laboratory (FPL) and Headquarters Engineering Staff, in cooperation with the Midwest Roadside Safety Facility undertook the task of developing bridge railing systems for roads with low traffic volumes and low speeds. Two low-cost bridge railing systems were developed and successful full-scale crash tests were conducted for their use on timber bridge decks using longitudinal lumber laminations. A curb-type timber railing system was designed to redirect a 3/4-ton pickup truck hitting at a speed of 24 km/hr (15 mph) and an angle of 15 degrees. The curb-type rail system used square, trapezoidal, or rectangular rail shapes. A flexible railing system consisting of steel W-beam supported by breakaway timber posts was designed to redirect a 3/4-ton pickup truck hitting at a speed of 50 km/hr (31 mph) and an angle of 25 degrees. The flexible railing system was developed according to Test Level 1 of NCHRP Report 350, *Recommended Procedure for the Safety Performance Evaluation of Highway Features*.

Historically, bridge railing systems have not been developed for use on low-speed, low-volume roads; however, many U.S. Forest Service and National Forest utility and service roads often carry very low traffic volumes at operating speeds of 24

to 32 km/hr (15 to 20 mph) or less. These roads are often narrow, generally incorporating one- or two-lane timber bridges with span lengths between 4.6 and 10.7 m (15 and 35 ft). The bridge rails that have been designed for high-speed facilities may be too expensive for low-volume roads. In recognition of the need to develop bridge railings for this very low service level, the U.S. Department of Agriculture (USDA) Forest Service, Forest Products Laboratory (FPL) and Headquarters Engineering Staff, in cooperation with the Midwest Roadside Safety Facility (MwRSF), undertook the task of developing two bridge railing systems.

OBJECTIVE

The objective of this research project was to develop two low-cost bridge railing systems for use on longitudinal timber bridge decks with low traffic volumes and speeds. A longitudinal glulam timber deck was selected for use in the development of the bridge railings because it is the weakest type of longitudinal timber deck for resisting transverse railing loads currently in use. Thus, any bridge railing not damaging the longitudinal glulam deck could be easily adapted to other, stronger, timber deck systems.

Curb-type railing systems were chosen as the basic design for the first bridge railing. A top-mounted curb type railing is shown in Figure 1 (a). Although curb barriers generally offer limited redirective capability at higher impact speeds, curb barriers can be very effective during low-speed impacts. A flexible railing with a breakaway post system was selected as the basic design for the second bridge railing. A side-mounted flexible railing is shown in Figure 1 (b).

EVALUATION CRITERIA

Background

Currently, bridge railings are usually designed to satisfy the requirements provided in AASHTO's *Guide Specifications for Bridge Railings (1)*. More specifically, bridge railings should be designed according to the ap-

propriate performance level of the roadway, based upon a number of factors such as design speed, average daily traffic (ADT), percentage of trucks, bridge rail offset, and number of lanes. These guide specifications include three performance levels, shown in Table 1, which provide criteria for evaluating the safety performance of bridge railings.

The recently published NCHRP Report 350, *Recommended Procedure for the Safety Performance Evaluation of Highway Features (2)*, provides for six test levels, shown in Table 1, for evaluating longitudinal barriers. Although this document does not contain objective criteria for selecting test level, safety hardware developed to meet the lower test levels is generally intended for use on lower-service-level roadways, and higher-test-level hardware is intended for use on higher-service-level roadways. The lowest performance level, Test Level 1, is suitable for applications on low-volume, low-speed facilities such as residential streets. However, operating speeds on these facilities are typically in the range of 48 km/hr (30 mph) or approximately twice as high as operating speeds on Forest Service utility roads. Thus, test impact conditions from Test Level 1 were deemed too severe for the low-cost curb-type bridge railing system envisioned. The second bridge railing, or flexible railing, was designed to meet Test Level 1 impact conditions because the increase in performance level could be achieved with little increase in cost.

Crash Test Conditions

Design impact conditions for narrow, low-volume utility roads were selected by the Forest Products Laboratory (FPL) of the USDA Forest Service in consultation with engineers of the Headquarters Engineering Staff. Reasonable design impact conditions for the curb-type bridge railings were estimated to involve a 3/4-ton pickup truck hitting at a speed of 24 km/hr (15 mph) and an angle of 15 degrees. The design impact conditions for the flexible bridge railing involved a 3/4-ton pickup truck hitting at a speed of 50 km/hr (31 mph) and an angle of 25 degrees according to Test Level 1 in NCHRP Report 350 (2). It is noted that a research study is in progress to develop a curb-type bridge railing to meet Test Level 1 of NCHRP Report 350 (2).

CURB-TYPE BRIDGE RAILINGS

Design Considerations

Timber was selected for use in the curb-type bridge railing designs on the basis of aesthetics and material availability. Further, curb railings were identified as a low-

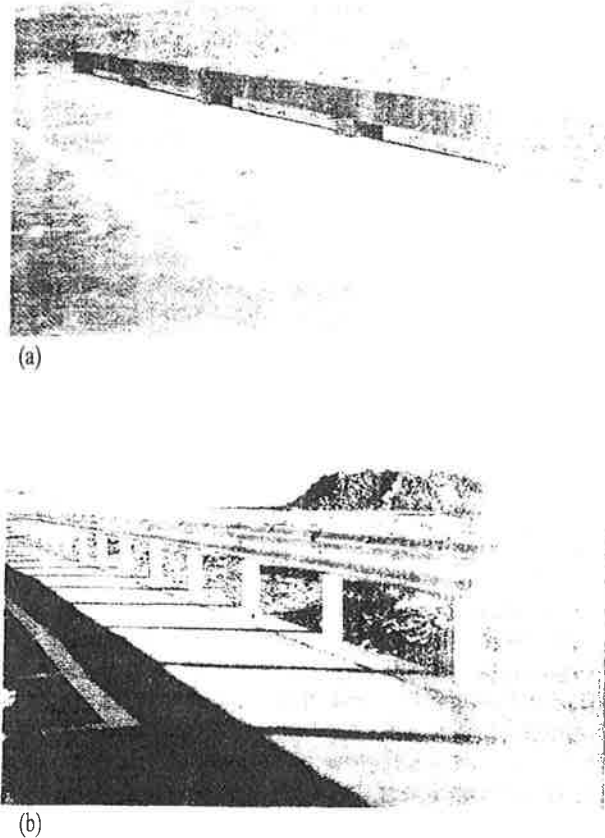


FIGURE 1 (a) Curb-type bridge railing and (b) flexible bridge railing.

TABLE 1 AASHTO Crash Test Conditions for Bridge Railings and NCHRP Report 350 Crash Test Conditions for Longitudinal Barriers

AASHTO Performance Level (1)	Impact Conditions				
	Small Car (816 kg)	Pickup Truck (2,449 kg)	Medium Single-Unit Truck (8,165 kg)	Van-Type Tractor-Trailer (22,680 kg)	
1	80.5 km/h and 20 deg	72.4 km/h and 20 deg			
2	96.6 km/h and 20 deg	96.6 km/h and 20 deg	80.5 km/h and 15 deg		
3	96.6 km/h and 20 deg	96.6 km/h and 20 deg		80.5 km/h and 15 deg	
NCHRP 350 Test Level (2)	Impact Conditions				
	Small Car (820 kg)	Pickup Truck (2,000 kg)	Single-Unit Van Truck (8,000 kg)	Tractor/Van Trailer (36,000 kg)	Tractor/Tank Trailer (36,000 kg)
1	50 km/h & 20 deg	50 km/h & 25 deg			
2	70 km/h & 20 deg	70 km/h & 25 deg			
3 (Basic Level)	100 km/h & 20 deg	100 km/h & 25 deg			
4	100 km/h & 20 deg	100 km/h & 25 deg	80 km/h & 15 deg		
5	100 km/h & 20 deg	100 km/h & 25 deg		80 km/h & 15 deg	
6	100 km/h & 20 deg	100 km/h & 25 deg			80 km/h & 15 deg

cost railing system and the most easily constructed design alternative for these low-service-level applications. Since most economical timber curb systems incorporate top-mounted single-railing designs, this type of structure was used for the new bridge rails.

Analysis of vehicular impacts with concrete and timber curbs revealed that the shape of the curb face could affect the redirective capacity of curb systems. A number of curb shape configurations were included in the design process. Each curb configuration was evaluated at different heights in order to determine the minimum height required to meet the selected performance criteria. Based on full-scale vehicle crash tests of curb systems 50.8 cm (20 in.) high (6) and a limited study of impacts with shorter curbs (unpublished research) using HVOSM computer simulation modeling (7), the researchers estimated that curbs 20.3 to 35.6 cm (8 to 14 in.) high should be able to meet the desired performance standard.

Peak lateral forces imparted to the curb railing were estimated to be approximately 9.5 kN (2.1 kips) using the procedures described by the NCHRP report, the AASHTO Guide, and Ritter et al. (3-5). Based on these findings, it was concluded that timber curb railings may be capable of withstanding design impact conditions without significant damage to the barrier or the timber

deck. Each railing was analyzed as a simply supported beam with pin connections at each end. Three rail shapes and sizes—a 20.3-cm (8-in.) by 20.3-cm (8-in.) square, a 20.3-cm (8-in.) by 22.9-cm (9-in.) trapezoid with a negative slope on the traffic-side face, and a 10.2-cm (4-in.) by 30.5-cm (12-in.) rectangle—were selected for a preliminary evaluation. A developmental testing program was then undertaken to evaluate the safety performance and height requirements for each of these curb rails.

Design Details

The basic curb design incorporated 6.10-m (20-ft) long rail sections mounted on scupper blocks. The rail elements, scupper blocks, and bridge deck were attached to each other with two 1.6-cm (5/8 in.) diameter ASTM A307 galvanized bolts placed 15.2 cm (6 in.) apart at each end and in the middle of each rail element. A bolted lap splice was also incorporated to attach the ends of adjacent rail elements. The 11.9-m (39-ft) long curb rails were constructed from two 6.10-m (20-ft) long rail sections and a 0.30-m (1-ft) long lap splice. Two sizes of timber scupper blocks were used to mount the curb rail elements on the timber deck. The curb rail

sections and scupper blocks were constructed from No. 1 Grade Douglas fir using rough-sawn and SIS specifications, respectively. Timber curb rail and scupper materials were treated to meet AWPAC Standard C14 with 192.22 kg/m^3 (12 pcf) creosote (8). Schematics of both a typical curb rail section mounted on the deck surface and a curb railing splice are shown in Figure 2.

The curb railings were attached to a longitudinal glulam timber deck supported by concrete abutments. The concrete abutments and the longitudinal glulam timber deck were the same as those used in the development of previously tested AASHTO PL-1 and PL-2 railing

systems (9-11). In addition, a 5.1-cm (2-in.) asphalt surface was placed on the top of the timber deck in order to represent actual field conditions.

Developmental Testing, Phase I

Developmental testing was used to determine critical heights for the three different curb shapes. The developmental testing used a 1985 Ford F-250 3/4-ton pickup truck with test inertial and gross static weights of 1999 kg (4,406 lb) and 2078 kg (4,581 lb), respec-

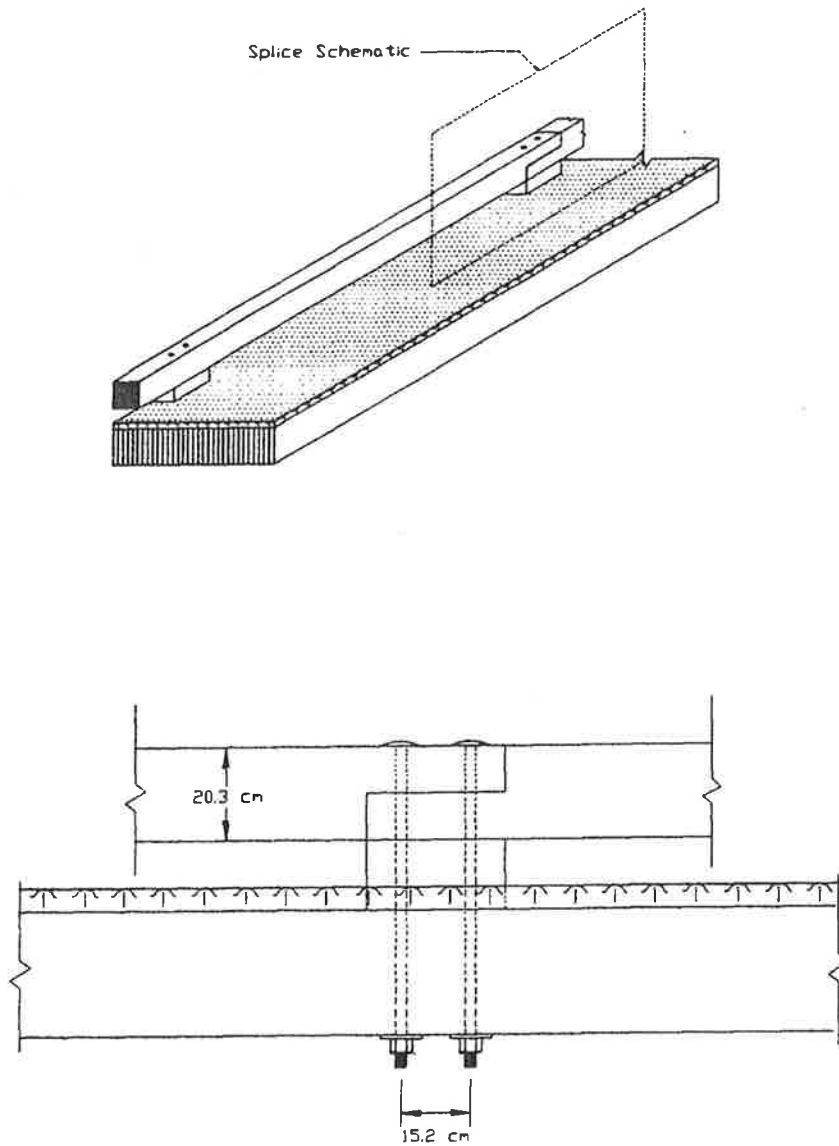


FIGURE 2 Typical curb section mounted to deck surface and curb railing splice.

tively. A pickup truck was driven into the rails at speeds of 24 and 32 km/hr (15 and 20 mph) and an angle of 15 degrees. No steering or braking inputs were applied to the vehicle during impact or until the vehicle had traveled an adequate distance downstream from the end of the rails.

The curb shapes were attached to a continuous concrete slab, as shown in Figure 3, with two 1.6-cm (5/8 in.) diameter ASTM A307 bolts spaced on 2.90-m (9-ft 6-in.) centers. If necessary, timber scupper blocks were placed below the rail shapes in order to mount the curb rails 20.3, 25.4, and 30.5 cm (8, 10, and 12 in.) above the surface.

Impact tests were performed on the three curb shapes mounted at three different heights for a total of nine curb configurations. The developmental testing phase consisted of 19 impact tests on the rail attached to the concrete slab, as shown in Table 2. For impacts at 24 km/hr (15 mph) and 15 degrees, the trapezoidal and rectangular shapes with a 20.3-cm (8-in.) mounting height successfully redirected the test vehicle with no tendency for the vehicle to climb. However, for the same impact conditions, the square shape with a 20.3-cm (8-in.) mounting height allowed the vehicle to climb over the top of the rail. Following these tests, it was determined that one full-scale vehicle crash test would be performed on one of the two successful curb shapes attached to the longitudinal timber deck. The trapezoidal shape with a 20.3-cm (8-in.) mounting height was selected for this crash testing because it appeared to provide a higher redirective capacity than the rectangular shape.

Full-Scale Crash Testing, Phase I

Full-scale crash testing used the same 3/4-ton pickup truck but with a test inertial and gross static weight of 1999 kg (4,406 lb), an impact speed of 24 km/hr (15 mph), and an angle of 15 degrees. The test vehicle was towed using a cable tow and guidance system and struck the rails attached to the longitudinal timber deck.

Originally, only one full-scale crash test was to be conducted on a 20.3-cm (8-in.) by 22.9-cm (9-in.) trapezoidal shape with 20.3-cm (8-in.) mounting height. However, because this test failed, two additional tests were conducted on the trapezoidal shape, one at the 20.3-cm (8-in.) mounting height and one at the 25.4-cm (10-in.) mounting height.

In Test LVCT-1a the vehicle struck the curb rail approximately 3.35 m (11 ft) from the upstream end of the 11.9-m (39-ft) long installation. During impact, the vehicle's right front tire climbed over the top of the curb. The vehicle came to rest on top of the curb at the end of the installation. In Test LVCT-1b the vehicle

struck the curb rail at the same location as in Test LVCT-1a. The vehicle's right front tire again climbed over the curb with little or no vehicle redirection. Following the two unsuccessful tests on the trapezoidal shape with a 20.3-cm (8-in.) mounting height, a third test was conducted on the trapezoidal shape with a 25.4-cm (10-in.) mounting height. The impact point for Test LVCT-1c was the same as that for the previous two tests. The vehicle's right front tire again climbed over



FIGURE 3 Square curb rail attached to concrete apron, three views.

TABLE 2 Summary of Curb-Type Bridge Railing Development, Phase I

Rail Type (cm x cm)	Rail Height (cm)	Test No.	Speed (km/h)	Results
Square - 20.3 x 20.3	20.3	1	24	Failed - vehicle over top of curb
Square - 20.3 x 20.3	25.4	2a 2b	24 24	Passed - right front tire briefly hopped onto curb Passed - right front tire popped into air
Trapezoid - 20.3 x 22.9	25.4	3a 3b 3c 3d	24 24 32 32	Passed - no climbing tendency Passed - no climbing tendency Passed - right-front tire popped into air Passed - right-front tire briefly hopped onto curb
Trapezoid - 20.3 x 22.9	20.3	4a 4b 4c	24 24 32	Passed - no climbing tendency Passed - no climbing tendency Failed - vehicle over top of curb
Trapezoid - 20.3 x 22.9	30.5	5	24	Passed - no climbing tendency
Square - 20.3 x 20.3	30.5	6a 6b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Rectangle - 10.2 x 30.5	30.5	7a 7b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Rectangle - 10.2 x 30.5	25.4	8a 8b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Rectangle - 10.2 x 30.5	20.3	9a 9b	24 24	Passed - no climbing tendency Passed - no climbing tendency

the top of the curb, which allowed the tire to go over the side of the bridge rail. The vehicle came to rest on top of the curb at the end of the installation.

Results of these tests were inconsistent with the previous findings from the developmental testing program. Factors that may have affected the results include the following: (a) air temperatures were much warmer when testing on the timber deck than during developmental testing on the concrete slab (average daily temperatures during developmental testing, Phase I, and full-scale crash testing, Phase I, were -2.2°C (28°F) and 17°C (63°F), respectively); (b) the trapezoidal curb rail was coated with a latex water-based paint to aid in photography and documentation of tests; and (c) creosote on the surface of the treated timber may have dried and increased friction levels between the tires and timber rail.

Developmental Testing, Phase II

Following three unsuccessful full-scale vehicle crash tests on the longitudinal deck with the trapezoidal curb rail, developmental testing was once again conducted on the concrete slab to determine the critical mounting heights for the three different curb shapes. The curb shapes were attached to the concrete in the same manner as during the first phase of the developmental testing program. The 3/4-ton pickup truck was again

driven into the curb railings at a speed of 24 km/hr (15 mph) and an angle of 15 degrees. The trapezoidal shape rail was tested with the same coating of paint used during the full-scale crash tests and creosote that may have dried on the timber rail surface.

Impact tests were performed on the three curb shapes mounted at heights ranging from 20.3 to 35.6 cm (8 to 14 in.) A total of eight curb configurations were evaluated with 15 crash tests, as shown in Table 3. For impacts at 24 km/hr (15 mph) and 15 degrees, a 30.5-cm (12-in.) mounting height successfully redirected the test vehicle for both the square and rectangular shapes with no tendency for vehicle climbing. However, for the same impact conditions, a 35.6-cm (14-in.) mounting height was required to successfully redirect the vehicle for the trapezoidal shape. The trapezoidal shape with a 30.5-cm (12-in.) mounting height allowed the tire to climb up and over the curb. These tests indicated that inconsistencies in the previous testing were not caused by paint applied to the trapezoidal rail but may have been a result of the drying creosote or the temperature changes mentioned previously. Following these tests, it was determined that one full-scale vehicle crash test would be performed on one of the successful curb shapes. The square shape with a 30.5-cm (12-in.) mounting height was selected for full-scale vehicle crash testing because it offered the most cost-effective design alternative.

TABLE 3 Summary of Curb-Type Bridge Railing Development, Phase II

Rail Type (cm x cm)	Rail Height (cm)	Test No.	Speed (km/h)	Results
Trapezoid - 20.3 x 22.9	20.3	10a 10b 10c	24 24 24	Failed - vehicle over top of curb Failed - vehicle over top of curb Failed - vehicle over top of curb
Rectangle - 10.2 x 30.5	20.3	11	24	Failed - vehicle over top of curb
Rectangle - 10.2 x 30.5	25.4	12	24	Failed - vehicle over top of curb
Rectangle - 10.2 x 30.5	30.5	13a 13b	24 24	Passed - right-front tire briefly popped into air Passed - right-front tire briefly popped into air
Trapezoid - 20.3 x 22.9	30.5	14a 14b 14c	24 24 24	Passed - minor vehicle uplift action Passed - right-front tire climbed onto curb Failed - vehicle over top of curb
Trapezoid - 20.3 x 22.9	35.6	15a 15b	24 24	Passed - no climbing tendency Passed - no climbing tendency
Square - 20.3 x 20.3	35.6	16	24	Passed - no climbing tendency
Square - 20.3 x 20.3	30.5	17a 17b	24 24	Passed - no climbing tendency Passed - no climbing tendency

Full-Scale Crash Testing, Phase II

One full-scale crash test (LVCS-4) was conducted on the 20.3- by 20.3-cm (8- by 8-in.) square shape with a 30.5-cm (12-in.) mounting height attached to the longitudinal timber deck. In Test LVCS-4 the vehicle hit the curb rail at a speed of 23.2 km/hr (14.4 mph) and an angle of 15 degrees. Impact occurred approximately 3.35 m (11 ft) from the upstream end of the 11.9-m (39-ft) long installation, as shown in Figure 4. The square shape with a 30.5-cm (12-in.) mounting height successfully redirected the vehicle, which came to rest approximately 22.0 m (72 ft) downstream from the impact, as shown in Figure 4. A summary of the test results and the sequential photographs are presented in Figure 5.

Except for minor scuff marks on the right-side tires, there was no visible vehicle damage, as shown in Figure 4. No damage occurred to the curb rail or steel hardware. In addition, the glulam timber deck was not damaged.

The curb-type bridge rail contained and redirected the test vehicle without penetrating or overriding the bridge rail. Detached elements, fragments, or other debris from the bridge rail did not penetrate or show potential for penetrating the occupant compartment and would not present any hazard to other traffic or pedestrians. The integrity of the occupant compartment was maintained with no intrusion or deformation. The vehicle remained upright during and after collision, and the vehicle's trajectory did not intrude into adjacent

traffic lanes. The vehicle exit angle of approximately 0 degrees was less than 60 percent of the impact angle or 9 degrees.

The curb bridge railing successfully redirected a 1999-kg (4,406-lb) pickup truck driven at a speed of 23.2 km/hr (14.4 mph) and an angle of 15 degrees. The curb bridge railing met the performance evaluation criteria (i.e., structural adequacy, occupant risk, and vehicle trajectory) for crash testing bridge railings (1,2) but at the reduced impact conditions of 24 km/hr (15 mph) and 15 degrees.

BREAKAWAY BRIDGE RAILING

Design Considerations

A steel W-beam railing with timber bridge posts was selected for use in the flexible bridge railing design based on previously crash-tested metal beam bridge railings (12-14), economics, and material availability. Breakaway posts rather than stiff posts were chosen in order to keep material costs below \$33/m (\$10/ft) by reducing the required structural capacity of the post-to-deck attachment. The post-to-deck attachment was designed so that no damage would occur to the timber deck or connection hardware. A side-mounted post-to-deck attachment with no rail or post blockouts was selected in order to reduce the required minimum width of timber deck.

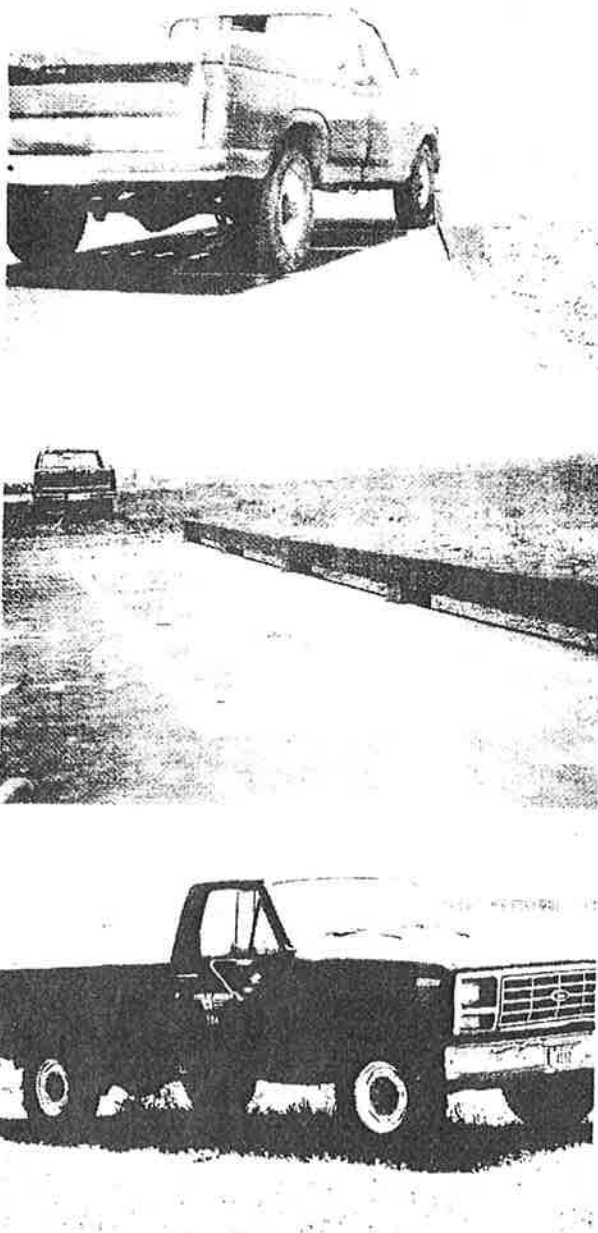


FIGURE 4 Impact location, vehicle trajectory, and vehicle damage, Test LVCS-4.

Static Post Testing

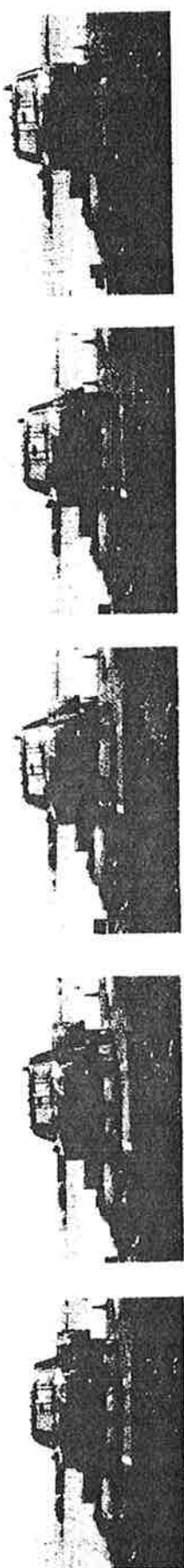
Static post testing was used to determine the force-deflection characteristics of two dimensions of lumber post sizes, 10.2-cm (4-in.) by 10.2-cm (4-in.) and 10.2-cm (4-in.) by 15.2-cm (6-in.) nominal. The cantilevered posts were bolted between two steel angles and attached to a rigid plate. Various angle sizes were used during

the testing in order to determine the optimum angle dimensions. Thirteen static tests were performed. A 10.2-cm (4-in.) by 15.2-cm (6-in.) lumber post measuring 83.8 cm (33 in.) long with steel angles measuring 12.7 cm (5 in.) by 12.7 cm (5 in.) by 1.0 cm (3/8 in.) was selected for the original design. The maximum static force for this post size was 10.7 kN (2.4 kips). The post and angle sizes were selected based on a maximum force level that would not damage the post-to-deck attachment hardware or the deck.

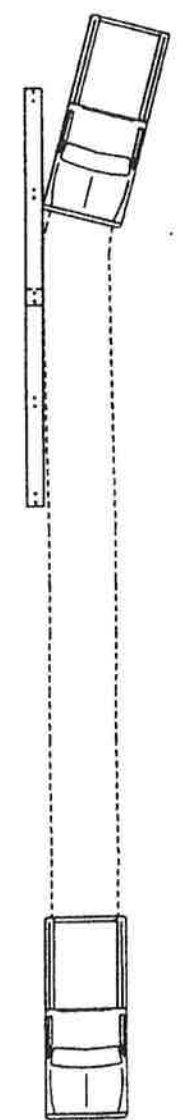
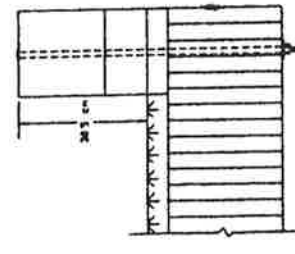
Following the failure of the first full-scale crash test, 24 additional static tests that included increasing the post height and placing saw cuts in the compression zone, tension zone, and combinations thereof were performed. A 10.2-cm (4-in.) by 15.2-cm (6-in.) lumber post measuring 93.3 cm (36.75 in.) long with steel angles measuring 12.7 cm (5 in.) by 12.7 cm (5 in.) by 1.0 cm (3/8 in.) was selected for the modified design. The modified posts also included a 2.5-cm (1-in.) horizontal saw cut placed on the tension side of the post 7.6 cm (3 in.) from the base of the post. The maximum static force for this post size was 5.8 kN (1.3 kips). Ritter et al. (15) provide additional details for the static post testing.

Design Details

A standard 12-gauge W-beam rail was selected for the rail element with a 61.0-cm (24-in.) top mounting height. However, after failure of the first full-scale crash test, the rail height was modified to 55.0 cm (21.65 in.) as measured from the top of the asphalt surface to the center of the rail. This provided a new rail top mounting height of approximately 70.6 cm (27.78 in.). In addition, the flat washer located under the head of the W-beam bolt was removed. The bridge rail was supported by 15 posts spaced on 1.90-m (6-ft 3-in.) centers. The chromated copper arsenate (CCA) treated lumber posts measured 10.2-cm (4-in.) by 15.2-cm (6-in.) nominal or 8.9-cm (3.5-in.) by 14.0-cm (5.5-in.) actual dressed size. The lumber posts were manufactured using Douglas fir Grade No. 2 or better. A 1.6-cm (5/8 -in.) diameter by 17.8-cm (7-in.) long ASTM A307 galvanized hex head bolt attached the rail to each post. Each post was placed between two 12.7-cm (5-in.) by 12.7-cm (5-in.) by 1.0-cm (3/8-in.) by 15.2-cm (6-in.) long ASTM A36 galvanized steel angles. Two 1.6-cm (5/8 -in.) diameter by 14.0-cm (5 1/2-in.) long ASTM A325 galvanized hex head bolts attached the post between the angles. Each post with attached angles was rigidly fixed to the outside vertical surface of the timber deck with two 1.9-cm (3/4-in.) diameter by 30.5-cm (12-in.) long ASTM A307 galvanized lag screws. A



0.000 sec 0.064 sec 0.275 sec 0.437 sec 1.229 sec



Test Number LVCS-4
 Date 5/12/93
 Bridge Rail Installation Low-Volume Curb Bridge Rail
 Length 11.89 m
 Timber Curb Rail
 Size Square 20.3 cm x 20.3 cm
 Top Mounting Height 30.5 cm
 Material Douglas Fir
 Grade No. 1
 Preservative Treatment Creosote
 Timber Scupper Block
 Size 10.2 cm x 20.3 cm x 30.5 cm
 Material Douglas Fir
 Grade No. 1
 Preservative Treatment Creosote
 Anchorage Bolts
 Type ASTM A307, Galvanized
 Size Two 1.6-cm ϕ Bolts Per Location
 Length 71.1 cm
 Spacing 2.90-m Centers

Bridge Deck Installation Longitudinal Ghilam Timber
 Panel Size 27.3 cm x 1.22 m x 5.72 m
 Material Ghilam Timber Deck Comb. No. 2
 Vehicle Model 1985 Ford F-250 Pickup
 Test Inertial Mass 1,999 kg
 Gross Static Mass 1,999 kg
 Vehicle Speed
 Impact 23.2 km/h
 Exit Not Available
 Vehicle Angle
 Impact 15 degrees
 Exit 0 degrees
 Vehicle Snagging None
 Vehicle Stability Satisfactory
 Maximum Vehicle Rebound Distance Not Applicable
 Bridge Rail Damage None
 Vehicle Damage None
 Vehicle Stopping Distance 21.95 m

FIGURE 5 Summary of test results and sequential photographs, Test LVCS-4.

schematic of the modified breakaway bridge railing is shown in Figure 6.

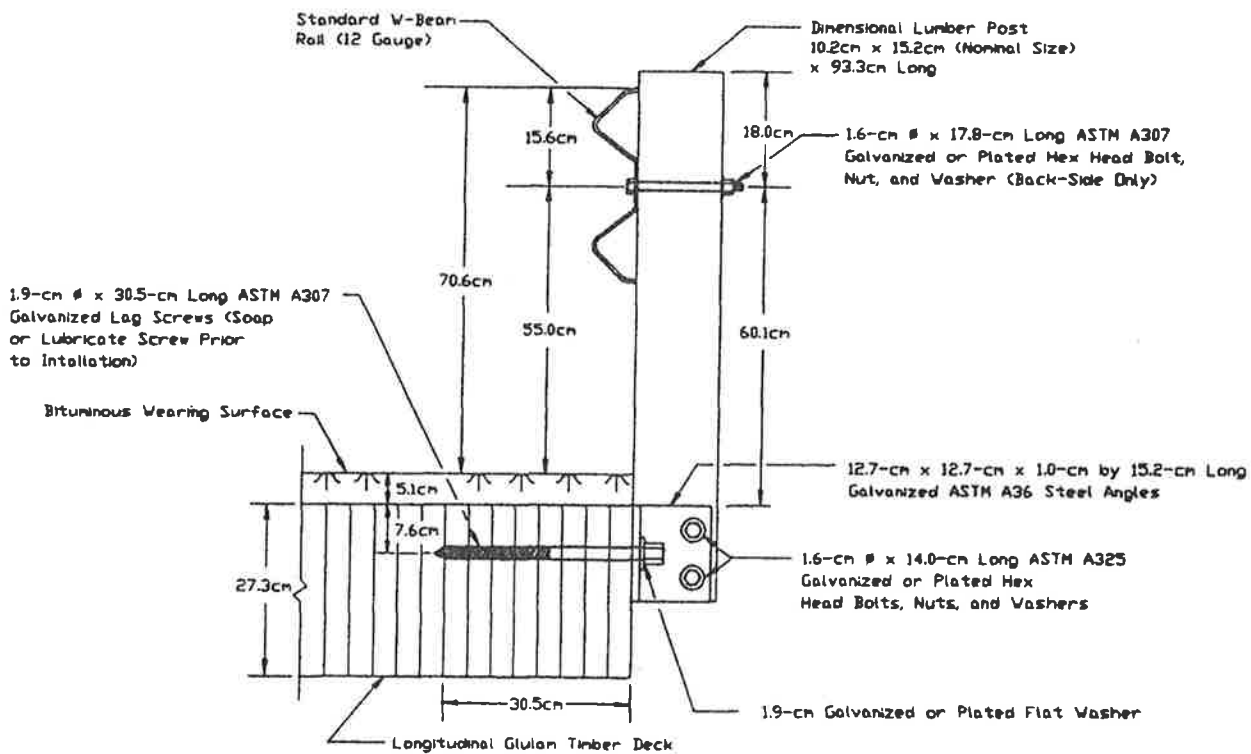
Approach guardrails were placed on each end of the bridge railing. The bridge railing with approach guardrails was 60.96 m (200 ft) long. Each W-beam approach guardrail was 15.24 m (50 ft long) and supported by 15.2-cm (6-in.) by 20.3-cm (8-in.) timber posts spaced on 1.90-m (6-ft 3-in.) centers. Guardrail anchorage was provided at each end by a modified breakaway cable terminal (MBCT) with steel foundation tubes, bearing plates, and channel struts.

The bridge railing was attached to a longitudinal glulam timber deck supported by concrete abutments. The concrete abutments, longitudinal glulam timber deck, and asphalt surface were the same as those used in the development of the curb-type systems.

BARRIER VII Computer Simulation Modeling

Following the preliminary design of the breakaway bridge railing, computer simulation modeling with BARRIER VII was performed to analyze the dynamic performance of the bridge railing before full-scale crash testing (16). Computer simulation was conducted modeling a 1996-kg (4,400-lb) pickup truck driven at 31 mph (500 km/hr) and an angle of 25 degrees according to Test Level 1 of NCHRP Report 350 (2).

The simulation results indicated that the original and modified breakaway bridge railing designs satisfactorily redirected the 1996-kg pickup truck. For the modified design, computer simulation predicted that eight breakaway lumber posts would be broken during impact, and the maximum permanent set and dynamic deflections



Notes:

- (1) Post Spacing 1.90m
- (2) Sawcut in post not shown.
- (3) Many details have been omitted.
- (4) 1in. = 2.54cm

FIGURE 6 Modified breakaway bridge railing.

of the W-beam were predicted to be 56.1 cm (22.1 in.) and 82.5 cm (32.5 in.), respectively. In addition, the predicted peak 0.050-sec average impact force perpendicular to the bridge railing was approximately 27 kN (6 kips).

Full-Scale Crash Testing

Two full-scale crash tests were performed with 3/4-ton pickup trucks on a breakaway bridge railing. The first test, LVBR-1, was conducted on a 61.0-cm (24-in.) high W-beam rail (original design), and the second test, LVBR-2, was conducted on a 70.6-cm (27.78 in.) high W-beam rail with a 2.5-cm (1-in.) saw cut on the tension side of the post (modified design). It was not necessary to conduct a full-scale crash test with a 820-kg minicompact hitting at 50 km/hr (31 mph) and 20 degrees since there was no potential for occupant risk problems arising from wheel snagging caused by the weak timber posts and low impact speed.

Test LVBR-1 (Original Design)

A 1984 Chevrolet C-20 pickup truck weighing 2041 kg (4,499 lb) struck the bridge rail at Post No. 7 at 50.2 km/hr (31.2 mph) and 26.8 degrees. Upon impact, the vehicle's bumper was forced over the top of the W-beam rail. The vehicle's tires then climbed up the face of the W-beam and the vehicle vaulted over the bridge rail. Failure of the bridge rail was attributed to insufficient rail mounting height. Damage to the connection angles and lag screws was also noticed.

Test LVBR-2 (Modified Design)

A 1985 Chevrolet C-20 pickup truck weighing 2043 kg (4,504 lb) struck the bridge rail at Post No. 7 at 49.2 km/hr (30.6 mph) and 24.9 degrees, as shown in Figure 7. A summary of the test results and the sequential photographs are shown in Figure 8. The vehicle became parallel to the bridge railing at 0.652 sec with a velocity of 38.8 km/hr (24.1 mph). Although the vehicle was redirected, it did not exit the bridge railing. The vehicle came to rest 13.4 m (44 ft) downstream from impact with the vehicle's left-side tires and right-side undercarriage resting on the deck surface, as shown in Figure 7. At no time, during impact or at any time thereafter did the vehicle's right-side tires contact the ground.

Vehicle damage was minor. Following the crash test, the vehicle's right-side tires were lifted onto the deck, and the vehicle was driven away. Damage on the right-front quarter panel was caused by vehicle-rail contact, and damage to the right-side undercarriage was caused by contact with the outer top surface of the deck, as

shown in Figure 7. Bridge rail damage was also minimal, as shown in Figure 9. One 1.90-m (6-ft 3-in.) section of W-beam rail was permanently damaged. Eleven posts, Nos. 4 through 14, fractured away from the deck attachment. Five steel angles were deformed downstream of impact because of contact between the angles and the undercarriage of the vehicle.

The modified breakaway bridge rail contained and redirected the test vehicle without allowing it to penetrate or override the barrier. Detached elements, fragments, or other debris from the bridge rail did not penetrate or show potential for penetrating the occupant compartment and would not present any hazard to other traffic or pedestrians. The integrity of the occupant compartment was maintained with no intrusion or deformation. The vehicle remained upright during and after collision, and the vehicle's trajectory did not intrude into adjacent traffic lanes. Thus, the modified breakaway bridge railing successfully met all the evaluation criteria for Test Level 1 of NCHRP Report 350 (2).

CONCLUSIONS

Curb-Type Bridge Railing

The square-shaped bridge rail with a 30.5-cm (12-in.) mounting height successfully redirected the pickup truck after an impact at a speed of 23.2 km/hr (14.4 mph) and an angle of 15 degrees. This result is consistent with the results from Phase II of the developmental testing program. Full-scale crash tests were not performed on the trapezoidal and rectangular shapes with 35.6-cm (14-in.) and 30.5-cm (12-in.) mounting heights, respectively. However, based on findings from the developmental testing program, it was reasoned that these shapes would behave similarly to the square-shaped curb rail and did not require additional full-scale crash testing.

Thus, three curb-type bridge railings were developed for longitudinal timber decks located on low-volume roads, as shown in Figure 10. The top-mounted timber curb railings provide economic and aesthetically pleasing bridge railing alternatives. Material costs for the three curb-type bridge railing systems are reasonably low. The rectangular-shaped railing system has the lowest material costs at \$39.60/m (\$12.07/ft), and the trapezoidal-shaped railing system has the highest material costs at \$47.08/m (\$14.35/ft). In addition, the curb-type railing system is easy to install and should have low construction labor costs. These railing systems could easily be adapted to other types of longitudinal timber decks. Finally, no bridge deck or railing damage was observed during testing on a longitudinal glulam

deck system. Thus, maintenance and repair costs associated with the new curb designs should be very low.

Modified Breakaway Bridge Railing

A flexible railing with a breakaway post system was developed and successfully met the Test Level 1 crash

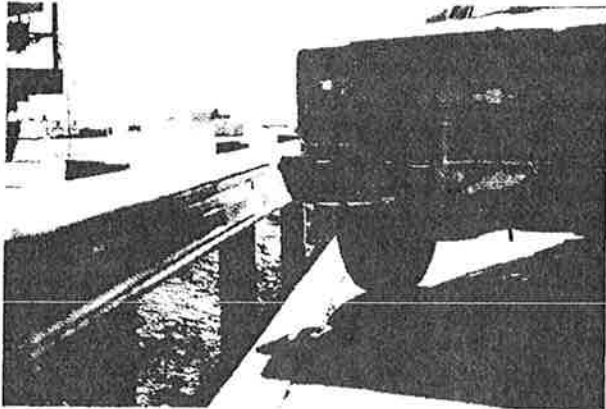


FIGURE 7 Impact location, vehicle trajectory, and vehicle damage, Test LVBR-2.

test conditions of NCHRP Report 350 (2). The 70.6-cm (27.78-in.) high W-beam bridge rail successfully redirected a 3/4-ton pickup truck after an impact at 49.2 km/hr (30.6 mph) and an angle of 25 degrees. The side-mounted railing provides an economic railing with readily available materials. Material costs for the bridge railing system are reasonably low at approximately \$25.85/m (\$7.88/ft). In addition, the breakaway railing system is easy to install and should have low construction labor costs. This railing system should also be adaptable to other types of longitudinal timber decks. In addition, no bridge deck damage was observed after testing; therefore, repair costs should also be kept to an absolute minimum.

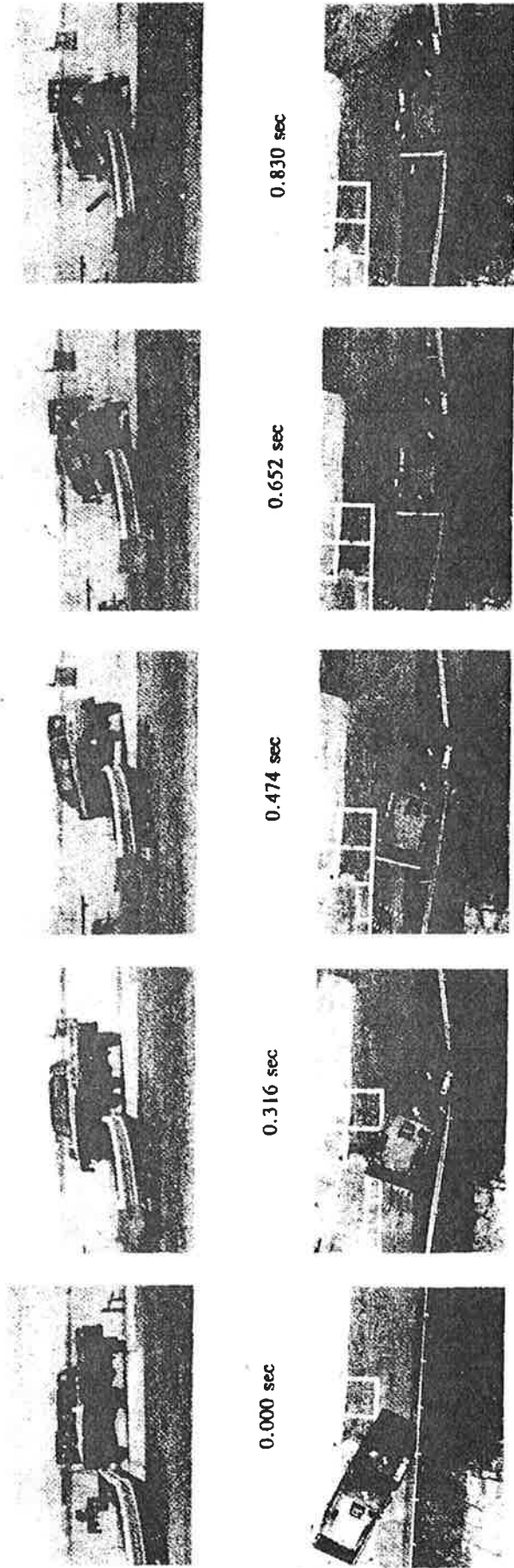
DISCUSSION AND RECOMMENDATIONS

The curb and breakaway bridge railings described herein were developed for low-impact condition requirements. The developmental testing program indicated that the redirective capacity of the curb railings could be increased by modifying the curb height and size, the rail-to-deck attachment, and the capacity of the rail splice connection. Curb railings should be able to meet the performance requirements of Test Levels 1 and 2 of NCHRP Report 350 (2). These higher-performance timber curb railings could be adapted for use in many different barrier applications. As bridge railings, the curbs would provide an aesthetic and economic alternative to conventional steel and concrete railings on many low-volume streets and highways with increased driver visibility. For flexible railings with breakaway posts, the full-scale crash testing program indicates that acceptable impact performance is possible although large dynamic rail deflections can be expected. Therefore, flexible railings with a modified post-to-deck attachment and stronger posts may be able to meet the performance requirements of Test Level 2 from NCHRP Report 350 (2).

Thus, it is recommended that the research described herein be extended to develop higher-performance timber curb railings and barriers and flexible railings for timber bridge decks.

ACKNOWLEDGMENTS

The authors would like to thank the following organizations for their contributions to the success of this research project: the American Institute of Timber Construction (AITC), Vancouver Washington, for donating the glulam materials for the deck construction and the Office of Sponsored Programs and the Center for Infra-



Test Number	LVBR-2
Date	3/18/94
Bridge Rail Installation	Low-Volume Breakaway Bridge Rail
Length	30.48 m
Steel W-Beam Rail	
Size	12 Gauge
Top Mounting Height	70.6 cm
Posts (No. 1 through 15)	
Size	8.9 cm x 14.0 cm x 93.3 cm
Material	Dimension Lumber (CCA)
Grade	No. 2 or Better
Bridge Deck Installation	Longitudinal Glulam Timber Bridge
Panel Size	Deck Panels
Material	27.3 cm x 1.22 m x 5.72 m
Vehicle Model	Glulam Timber Deck Comb. No. 2
Test Inertial Mass	1985 Chevrolet C-20 Pickup
Gross Static Mass	2,043 kg
Vehicle Speed	2,043 kg
Impact	49.2 km/h
Exit	NA

Vehicle Angle	Impact	24.9 degrees
	Exit	NA
Vehicle Sagging		None
Vehicle Stability		Satisfactory
Effective Coefficient of Friction (μ)		0.28 (Fair)
Occupant Impact Velocity - normalized		
	Longitudinal	2.2 m/s (9 m/s) (2)
	Lateral	1.9 m/s (9 m/s) (2)
Occupant Ridedown Deceleration - 0.010-msec average		
	Longitudinal	4.3 g's (15 g's) (2)
	Lateral	3.8 g's (15 g's) (2)
Vehicle Damage		Minor
TAD		1-RFQ-1
VDI		01RFEW1
Maximum Vehicle Rebound Distance		Not Applicable
Bridge Rail Damage		Minor Rail Deformation and Eleven Fractured Posts
Maximum Dynamic Deflection		131.8 cm
Maximum Permanent Set Deflection		115.3 cm

FIGURE 8 Summary of test results and sequential photographs, Test LVBR-2.

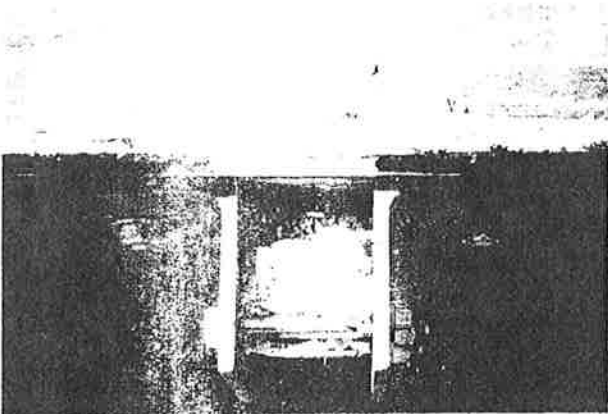
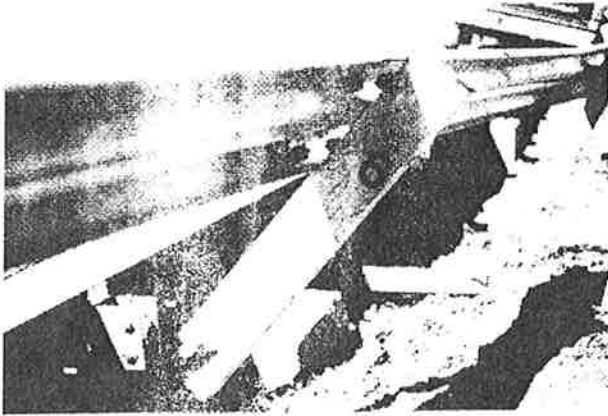
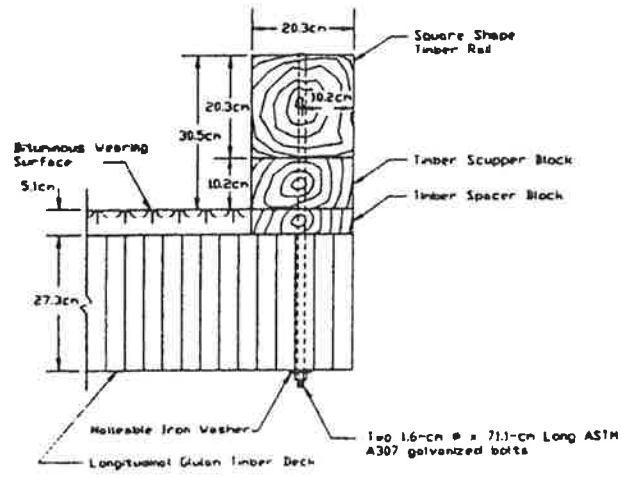
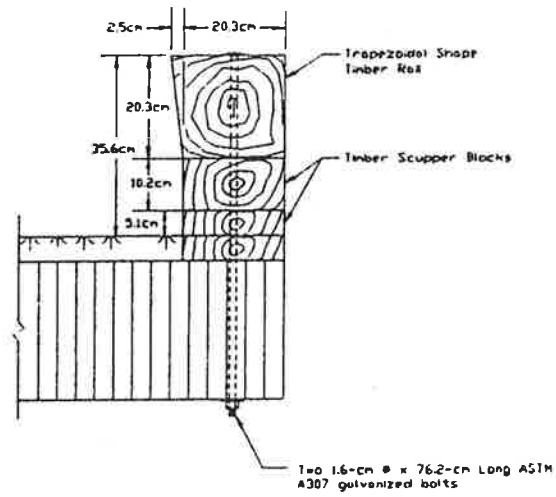


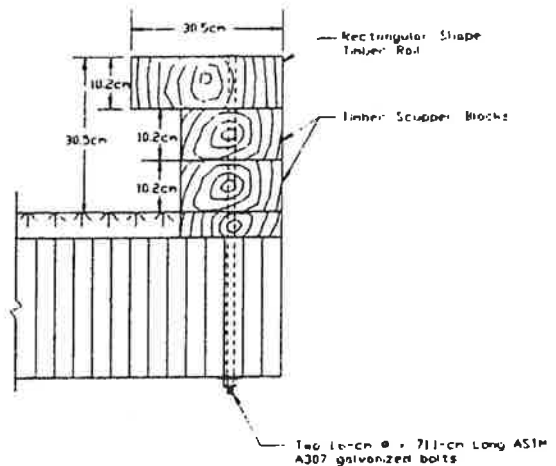
FIGURE 9 Bridge rail damage, Test LVBR-2, three views.



(a)



(b)



(c)

FIGURE 10 (a) Square-shaped curb, (b) trapezoidal-shaped curb, (c) rectangular-shaped curb.

structure Research, University of Nebraska-Lincoln, Lincoln, for matching support.

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Crash-Tested Bridge Railings for Timber Bridges

Michael A. Ritter, *Forest Service, U.S. Department of Agriculture*
Ronald K. Faller, *University of Nebraska-Lincoln*
Sheila R. Duwadi, *Federal Highway Administration*

Bridge railing systems in the United States historically have been designed on the basis of static load criteria given in the AASHTO *Standard Specifications for Highway Bridges*. In the past decade, full-scale vehicle crash testing has been recognized as a more appropriate and reliable method of evaluating bridge railing acceptability. In 1989 AASHTO published *Guide Specifications for Bridge Railings*, which gives the recommendations and procedures to evaluate bridge railings by full-scale vehicle crash testing. In 1993 NCHRP published *Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features*, which provides criteria for evaluating longitudinal barriers. From these specifications, a cooperative research program was initiated to develop and crash test several bridge railings for longitudinal wood decks. The research resulted in the successful development and testing of five bridge railing systems for longitudinally laminated wood bridge decks in accordance with the AASHTO Performance Level 1 and Performance Level 2 requirements and the Test Level 4 requirements of *NCHRP Report 350*.

The primary purpose of a bridge railing is to safely contain vehicles that cross the bridge. To meet this objective, railings must be designed to withstand the force of vehicle impact.

In designing railing systems for highway bridges, engineers traditionally have assumed that vehicle impact forces can be approximated by equivalent static loads that are applied to railing elements. Although railing loads are actually dynamic, the equivalent-static-load method has been used for many years as a simplified approach to standardized railing design. Currently, the *AASHTO Standard Specifications for Highway Bridges (1)* requires that rail posts be designed to resist an outward transverse static load of 44.5 kN (10,000 lb). A portion of this load is also applied to posts in the inward transverse, longitudinal, and vertical directions and to the rail elements. These requirements are identical for all bridges regardless of bridge geometry or traffic conditions. Thus, a railing for a single-lane bridge on a low-volume road must meet the same loading requirements as a railing for a bridge on a major highway.

Despite the widespread use of design requirements based primarily on static load criteria, the need for more appropriate criteria for full-scale vehicle crash tests has long been recognized. The first U.S. guidelines for full-scale vehicle crash testing were published in 1962 (2) in a one-page document that provided basic guidelines for the test vehicle mass, approach speed, and impact angle and provided a degree of uniformity to the traffic barrier research in progress at the time.

Through subsequent use of this document, the need for additional comprehensive guidelines became apparent, and several reports were published during the 1970s through NCHRP. In 1981 NCHRP released *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances* (3). This comprehensive report has been the primary source of crash testing criteria for more than a decade and continues to serve as the basis for current bridge railing testing requirements.

Although crash test criteria have been available for many years, the requirement to implement crash testing as a means of evaluating bridge railings in the United States depended on the jurisdiction. Some states implemented extensive bridge railing crash testing programs, whereas others continued to use static load design exclusively. The first recognition of full-scale crash testing in a national bridge specification came in 1989, when AASHTO published the *Guide Specifications for Bridge Railings* (or AASHTO Guide Specifications) (4). This work presents recommendations for the development, testing, and use of crash-tested bridge railings and refers extensively to NCHRP 230 for crash testing procedures and requirements.

A primary concept of the AASHTO Guide Specifications is that bridge railing performance needs differ greatly from site to site and that railing designs and costs should match site needs. Thus, recommended requirements for railing testing are based on three performance levels: Performance Level 1 (PL-1), PL-2, and PL-3. The PL-1 requirements represent the weakest system, and the PL-3 the strongest system. The relationship between the railing performance level and requirements for a specific bridge depend on a number of factors, such as the type of roadway, design speed, average daily traffic, and percentage of trucks in the traffic mix. The recently published *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features* (5) provides for six test levels to evaluate longitudinal barriers: Test Level 1 (TL-1) through TL-6. Although this document does not include objective criteria for relating a test level to a specific roadway type, the lower test levels generally are intended for use on roadways with lower service levels and certain types of work zones, whereas the higher test levels are intended for use on higher-service-level roadways. Most highways on which wood bridges are installed will require railings that meet either the AASHTO PL-1 or PL-2 requirements or the NCHRP 350 TL-1 through TL-4 requirements. A railing that meets either PL-3, TL-5, or TL-6 requirements currently has a very limited application for wood bridges because of the high traffic volume and speeds associated with these levels.

The AASHTO Guide Specifications are optional, and the use of static load design criteria is permitted. How-

ever, emphasis on the use of crash-tested railings for new federally funded projects has increased significantly the role of full-scale crash testing as a means of evaluating railing performance. It is anticipated that AASHTO will adopt the guide specifications in the future, making crash-tested railings mandatory for most bridges. FHWA has officially adopted NCHRP *Report 350* as a replacement for *NCHRP Report 230*. At this time, it is unclear if AASHTO will adopt the Report 350 criteria into its guide specifications or retain the current criteria based on Report 230.

As of August 1990, 25 bridge railings had been successfully crash tested in accordance with the requirements of the AASHTO Guide Specifications and approved for use on federal-aid projects by FHWA (6). Of these railings, 24 are for concrete bridge decks and 1 is for a wood deck. For wood bridges to compete with other bridges in the future, a range of crash-tested bridge railings for different wood bridge types will be required. Because of this need, national emphasis was placed on developing a limited number of crash-tested railings for wood bridges.

OBJECTIVE AND SCOPE

To meet the need for crashworthy railings for wood bridges, the Forest Products Laboratory, USDA Forest Service, in cooperation with the Midwest Roadside Safety Facility of the University of Nebraska at Lincoln, FHWA, and the wood products industry, initiated a program to develop crash-tested bridge railings for longitudinal wood decks. The program objectives were to develop five crashworthy railings: three to meet AASHTO PL-1, one to meet AASHTO PL-2, and one to meet NCHRP *Report 350* TL-4. The scope of the project was limited to railings for longitudinal wood decks, 252 mm (10 in.) or greater in thickness, and constructed of glued-laminated (glulam) timber, spike-laminated lumber, or stress-laminated lumber. In each system, the lumber laminations are placed edgewise and oriented with the lumber length parallel to the direction of traffic. A brief description of each longitudinal deck bridge type is provided in *Timber Bridges: Design, Construction, Inspection, and Maintenance* (7).

Longitudinal glulam timber decks are constructed of panels that consist of individual lumber laminations glued together with waterproof structural adhesives. The panels are 1.07 to 1.38 m (3.5 to 4.5 ft) wide and effectively function as a large, solid block of wood. To form the bridge deck, panels are placed side by side and interconnected by transverse distributor beams bolted to the deck underside at intervals of 2.4 m (8 ft) or less. These distributor beams are designed to transfer vertical

loads between adjacent panels. They are not designed to resist lateral loads.

Spike-laminated decks are constructed of sawn lumber laminations 102 mm (4 in.) in nominal thickness. The individual laminations are interconnected with spikes that are typically 8 or 9.5 mm (5/16 or 3/8 in.) in diameter and 356 to 406 mm (14 to 16 in.) long. The decks are commonly manufactured in panels that are 1.5 to 2.1 m (5 to 7 ft) wide and interconnected with transverse distributor beams in a manner similar to longitudinal glulam timber decks.

Stress-laminated decks are constructed of sawn lumber laminations that are typically 51 to 102 mm (2 to 4 in.) in nominal thickness. The laminations are stressed together with high-strength steel bars that are placed in holes drilled through the center of the wide faces of the laminations. When tensioned, the bars create compression between the laminations, and the entire deck effectively acts as a solid, orthotropic wood plate.

TEST REQUIREMENTS AND EVALUATION CRITERIA

Test requirements and evaluation criteria for this project followed procedures defined in the AASHTO Guide Specifications (including applicable references to *NCHRP Report 230*) and the *NCHRP Report 350* criteria. These procedures establish a uniform methodology for testing and evaluating railings so that the safety performance of different railing designs, tested and evaluated by different agencies, can be compared. It is impractical and impossible to test all railings for all possible vehicle and impact conditions. Therefore, the procedures specify a limited number of tests using severe vehicle impact conditions and a set of criteria against which test results may be evaluated.

Test Requirements

Vehicle impact requirements for railing crash testing depend on the railing performance or test level and are specified as requirements for vehicle type and weight, impact speed, and impact angle relative to the longitudinal railing axis. Testing for PL-1 requires two vehicle impact tests, and testing for PL-2 and TL-4 requires three vehicle impact tests. A summary of the requirements for PL-1, PL-2, and TL-4 is given in Table 1. In some cases, all tests for a given level may not be required if a railing with similar geometry and strength was tested previously and found to be satisfactory.

In addition to vehicle impact requirements, the AASHTO Guide Specifications and the *NCHRP Report 350* criteria also specify requirements for data acquisition and construction of the bridge railings. Require-

ments for data acquisition are referenced to Reports 230 and 350 and include specific data collection parameters and techniques that must be completed before, during, and after the crash test. Construction requirements specify that the bridge railing be designed, constructed, erected, and tested in a manner representative of actual installations. To assess properly the performance of most bridge railings, they must also be evaluated as a system in combination with the bridge superstructure for which it is intended. This is very important for railings for wood bridges, because the attachment of the railing to the bridge deck and the ability of the wood superstructure to resist applied railing loads may often be the controlling parameters.

Evaluation Criteria

Evaluation criteria for full-scale crash testing are based on three appraisal areas: structural adequacy, occupant risk, and vehicle trajectory after the collision. Criteria for structural adequacy are intended to evaluate the ability of the railing to contain, redirect, or permit controlled vehicle penetration in a predictable manner. Occupant risk evaluates the degree of hazard to occupants of the impacting vehicle. Vehicle trajectory after the collision is concerned with the path and final position of the impacting vehicle and the probable involvement of the impacting vehicle with other traffic. Note that these criteria address only the safety and dynamic performance of the railing and do not include service criteria such as aesthetics, economics, bridge damage, or post-impact maintenance requirements. The following evaluation criteria are summarized from the AASHTO Guide Specifications for PL-1 and PL-2 testing (similar evaluation criteria are provided in *NCHRP Report 350*):

1. The railing shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the railing is acceptable.
2. Detached elements, fragments, or other debris from the railing shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
3. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.
4. The vehicle shall remain upright during and after collision.
5. The railing shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle does not yaw more than 5 degrees away from the railing

from time of impact until the vehicle separates from the railing.

6. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction μ , where $\mu = 0.0-0.25$ is good, $\mu = 0.26-0.35$ is fair, and $\mu \geq 0.36$ is marginal. Requirements for computing μ are given in the AASHTO Guide Specifications.

7. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 610-mm (2-ft) longitudinal and 305-mm (1-ft) lateral displacements, shall be less than 9.15 m/sec (30 ft/sec) in the longitudinal direction and 7.63 m/sec (25 ft/sec) in the lateral direction. In addition, the highest 10-msec average vehicle accelerations subsequent to the instant of hypothetical passenger impact should be less than 147 m/sec² (483 ft/sec²) in the longitudinal and lateral directions.

8. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 30.5 m (100 ft) plus the length of the *test* vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 6.1 m (20 ft) from the line of the traffic face of the railing.

DEVELOPMENT PHASE

Using a fundamental understanding of the performance characteristics of each deck type, development work was initiated to formulate a methodology for the railing tests. Because of economics and time, it was considered impractical to develop and test different railing systems for each longitudinal deck type. Instead, a more feasible approach was undertaken to develop several railing systems that could be adapted to each of the three longi-

tudinal deck types, without modifications that would result in reduced performance. To accomplish this, it was determined that railing development and testing should use the weakest deck type. This decision was based on the premise that if successful tests could be completed on the weakest deck, the railing could be adapted to stronger decks without hurting performance.

In assessing the potential resistance of each deck type to transverse railing impact forces, the strength of the wood and mechanical reinforcement was considered. Of primary concern was loading that could introduce tension perpendicular to grain stress in the wood deck.

Of the three deck types, the stress-laminated deck was considered the strongest for transverse railing loads, because the high-strength steel bars are continuous across the deck width. Loads developed at vehicle impact can be effectively distributed across the deck by the bars, making the entire deck width effective in resisting the applied loads.

The spike-laminated deck was considered to be of intermediate strength. If railing loads are applied transverse to the panel length, the loads are resisted by the spikes in withdrawal. Because of this, tension perpendicular to grain in the lumber laminations is not a concern; however, the spikes could be pulled from the deck, resulting in longitudinal separations between the laminations, and additional reinforcement would be required.

The glulam timber deck was considered to be the weakest in resisting railing loads, because the glulam timber panels act as solid pieces of wood, and loads applied transverse to the panel length are most likely to introduce tension perpendicular to grain and failure in the upper panel section. Mechanical reinforcement was considered necessary for longitudinal glulam timber

TABLE 1 Vehicle Impact Requirements for PL-1, PL-2, and TL-4 Bridge Railings

AASHTO Performance Level (4)	Impact Conditions		
	Small Car (816 kg)	Pickup Truck (2,449 kg)	Medium Single-Unit Truck (8,165 kg)
1	80.5 km/h 20 deg	72.4 km/h 20 deg	
2	96.6 km/h 20 deg	96.6 km/h 20 deg	80.5 km/h 15 deg
NCHRP 350 Test Level (5)	Impact Conditions		
	Small Car (820 kg)	Pickup Truck (2,000 kg)	Single-Unit Van Truck (8,000 kg)
4	100 km/h 20 deg	100 km/h 25 deg	80 km/h 15 deg

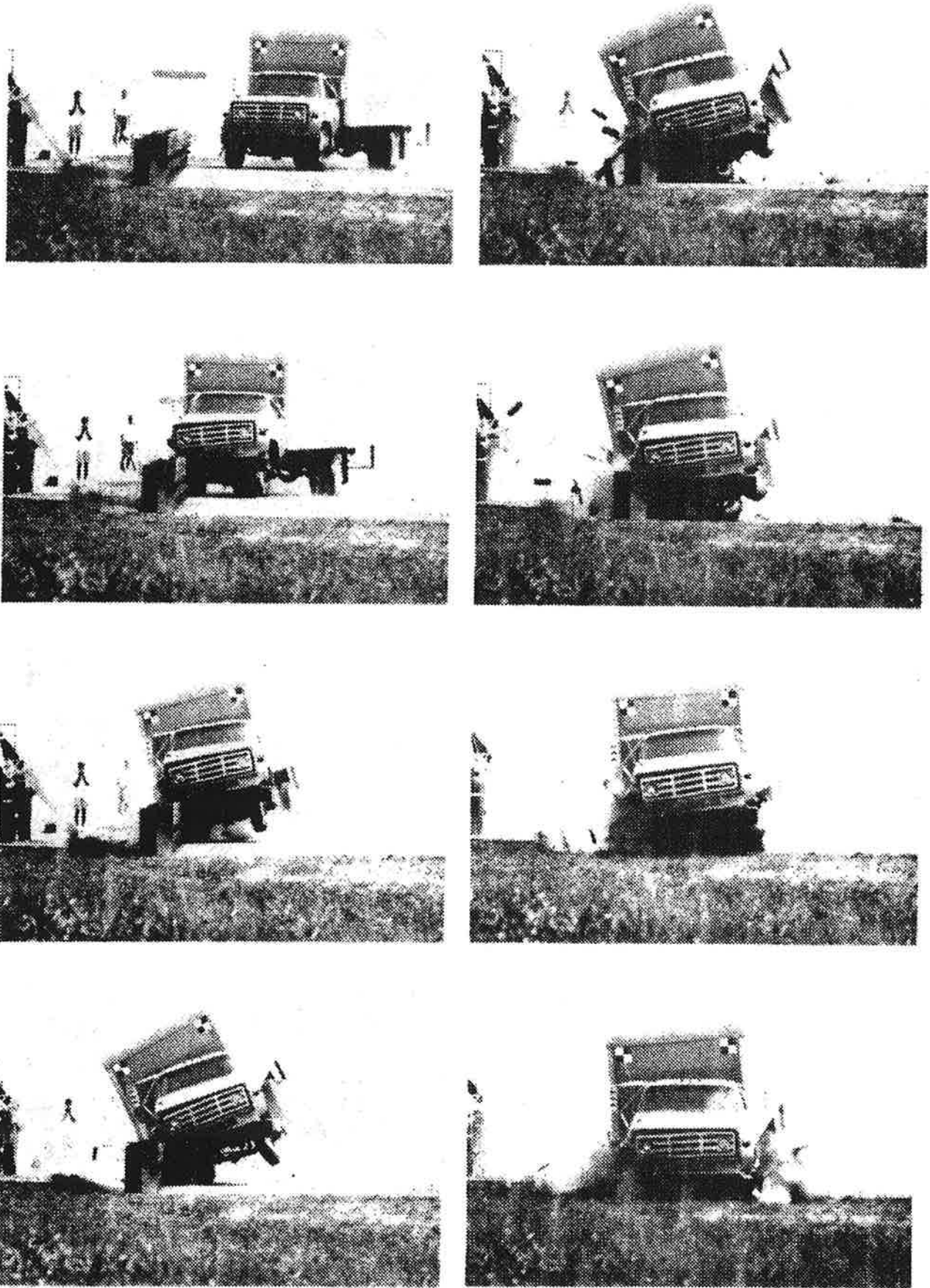


FIGURE 1 Crash-test sequence for 8172-kg (18,000-lb) truck traveling at 80.5 km/hr (50 mph), hitting railing at 15-degree angle to longitudinal railing axis.

decks to resist railing loads without damage. Thus, the glulam timber deck was considered the weakest deck for transverse railing loads and was selected for full-scale crash testing. If bridge railings performed acceptably on the glulam timber system, it was rationalized that the railings could be adapted to the other longitudinal wood bridge decks with no reduction in railing performance.

The primary emphasis of the railing design process was to develop railings that would meet the requirements for the AASHTO Guide Specifications and *NCHRP Report 350*. In addition, it was determined that consideration be given to (a) the extent of probable damage to the structure after vehicle impact and the difficulty and cost of required repairs; (b) adaptability of the railing to different wood deck types; (c) cost of the railing system to the user, including material, fabrication, and construction; (d) ease of railing construction and maintenance; and (e) aesthetics.

The conclusion of the development phase involved the design of several railing systems and preparation of plans and specifications for testing. The selection and design of these final systems were based on a review of other railings that had been crash tested successfully, as well as those that are used on wood bridges but had not been crash tested. To the extent possible, feasible designs were evaluated using computer simulation models. Although several proven computer models were used, it was difficult to adapt the programs for wood components because the behavior and properties of the wood systems at ultimate loading were unknown. Data collected during the crash testing were used to refine input parameters and more accurately predict railing performance in subsequent tests.

TEST METHODOLOGY

Testing of all bridge railings was completed at the Midwest Roadside Safety Facility in Lincoln, Nebraska. The site is located at an airport and was formerly a taxiway and parking area for military aircraft. It includes approximately 11 ha (27 acres) of concrete pavement and 1.6 ha (4 acres) of soil surface. To complete railing testing, a test bridge was constructed that measured approximately 2.4 m (8 ft) wide and 28.6 m (93.75 ft) long, in five simply supported spans measuring 5.72 m (18.75 ft) each. The deck was constructed of glulam timber panels 273 mm (10.75 in.) thick and 1.2 m (4 ft) wide. The glulam timber for the deck was Combination 2 Douglas fir given in the *AASHTO Standard Specifications for Highway Bridges (1)* and was treated with pentachlorophenol in heavy oil in accordance with American Wood Preservers' Association (AWPA) Standard C14 (8). Two glulam timber panels were placed

side by side to achieve the 2.4-m (8-ft) width, and transverse distributor beams were attached to the deck underside per AASHTO requirements (1). The test bridge was supported by concrete footings that were placed in excavations so that the top of the test bridge was level with the concrete surface at the site.

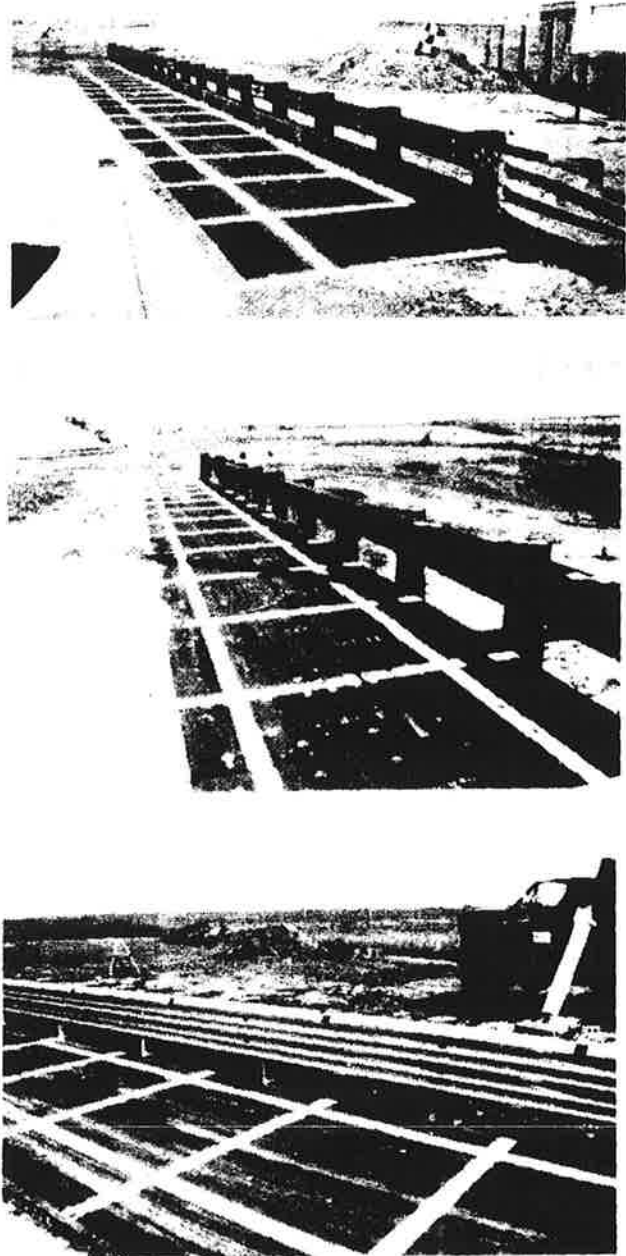


FIGURE 2 Bridge railings successfully crash tested to AASHTO PL-1 (photographs taken before testing).

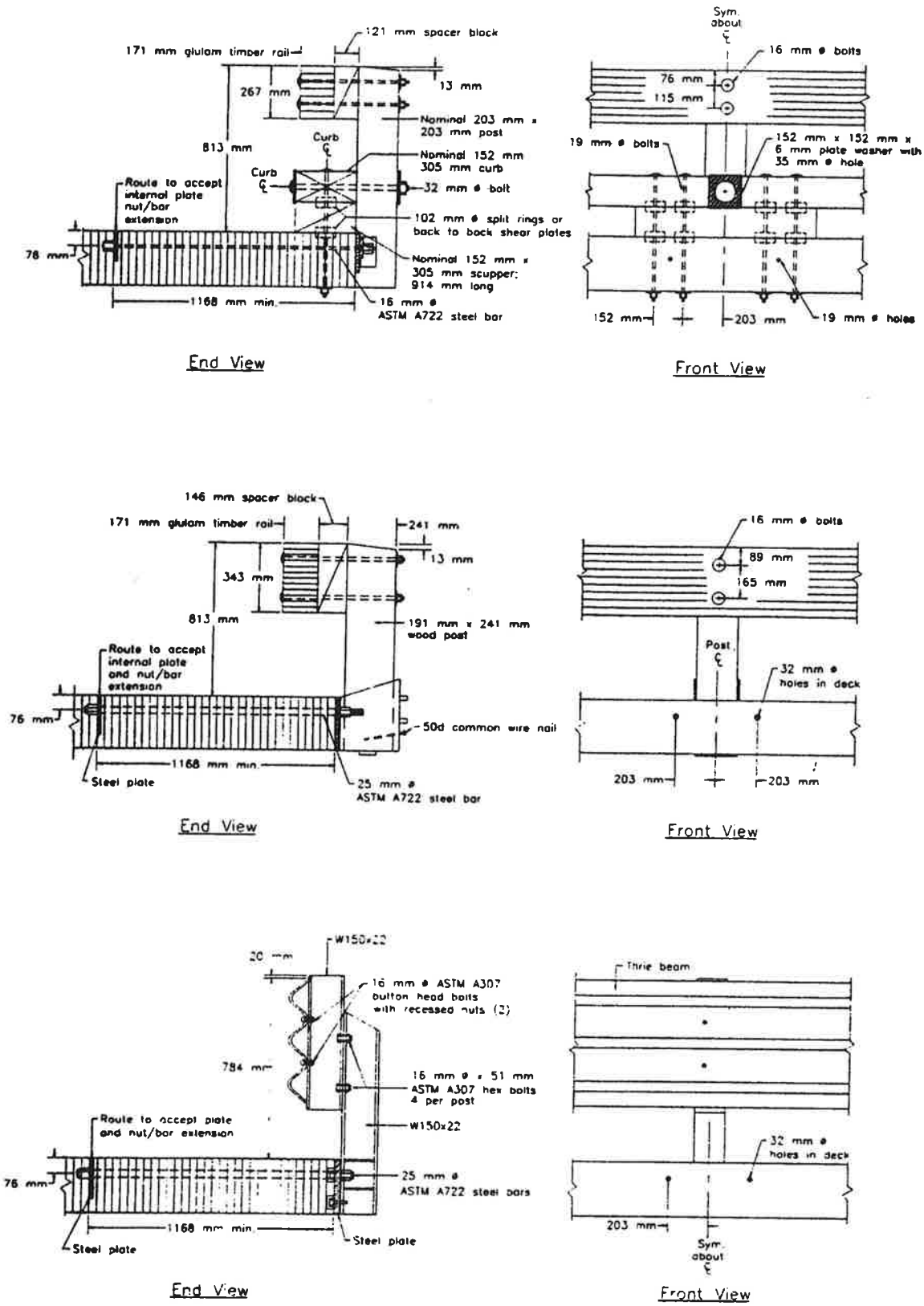


FIGURE 3 Drawings of bridge railings successfully crash tested to AASHTO PL-1: *top*, glulam timber rail with curb; *middle*, glulam timber rail without curb; *bottom*, steel rail.

Vehicle propulsion and guidance were provided by steel cable configurations. For propulsion, a reverse cable tow with a 1:2 mechanical advantage was used. A cable was attached to the front of the vehicle, routed through a series of pulleys, and connected to a tow vehicle that traveled in a direction opposite to the test vehicle. The unoccupied test vehicle was then pulled by the tow vehicle and released from the tow cable approximately 9.2 m (30 ft) before impact. A vehicle guidance system developed by Hinch was used to steer the test vehicle (9). Using this system, the left front wheel hub is attached to a tensioned steel cable that maintains the vehicle's direction along a designated straight path. Approximately 9.2 m (30 ft) from impact, the guidance connection is sheared off and the vehicle separates from the guidance cable. A crash-test sequence for an 8172-kg (18,000-lb) vehicle is shown in Figure 1.

Data acquisition parameters and techniques for the crash testing program were based on requirements of the AASHTO Guide Specifications and *NCHRP Report 350* and followed three testing phases: pretest, test, and posttest. In the pretest phase, the as-built bridge railing and vehicle were documented using photography and drawings that indicated the applicable configuration, dimensions, and vehicle weight. During the test phase, data on the vehicle impact speed, impact angle, trajectory, and accelerations were collected primarily through the use of high-speed motion picture photography and accelerometers mounted on the vehicle. In the posttest phase, the condition of the railing, bridge superstructure, and vehicle were documented using photography and standardized damage assessment methods, including the traffic accident data scale (10) and vehicle damage index (11). Additional instrumentation was placed

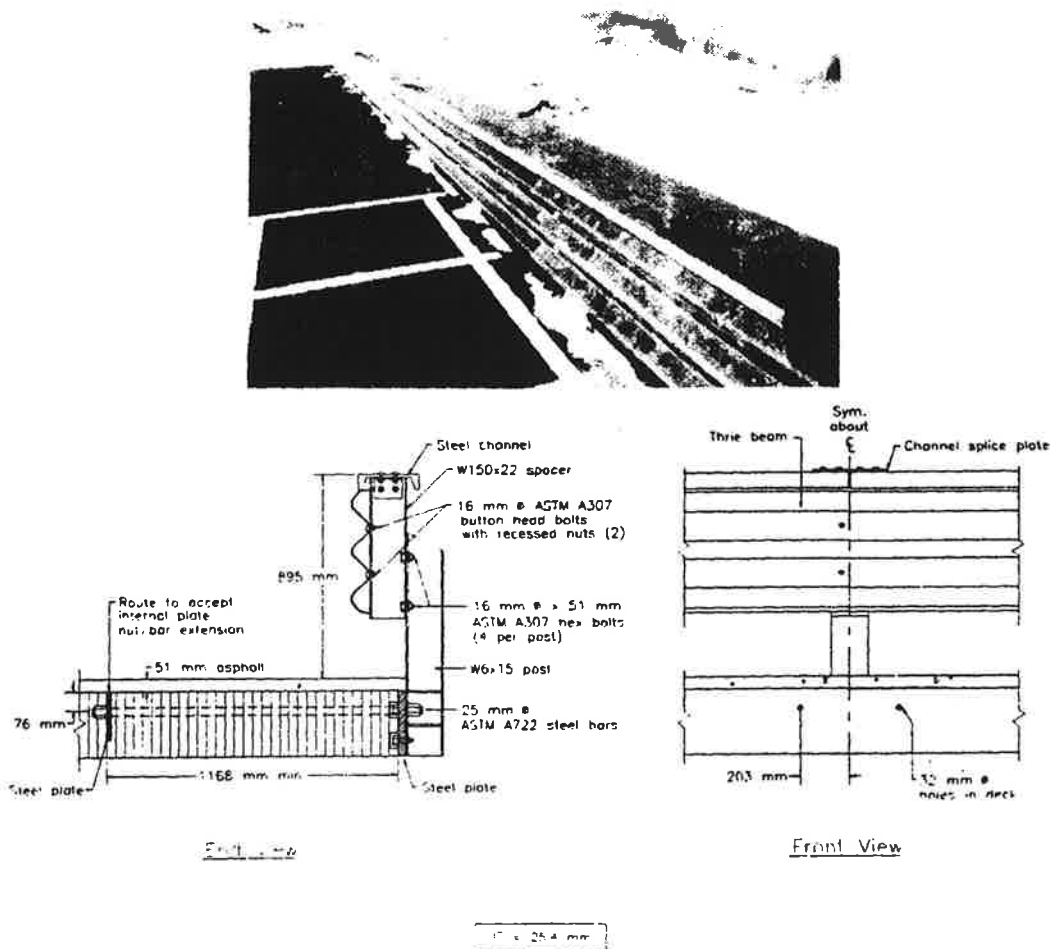


FIGURE 4 Steel thrie beam bridge railing successfully crash tested to AASHTO PL-2 (photograph taken before testing).

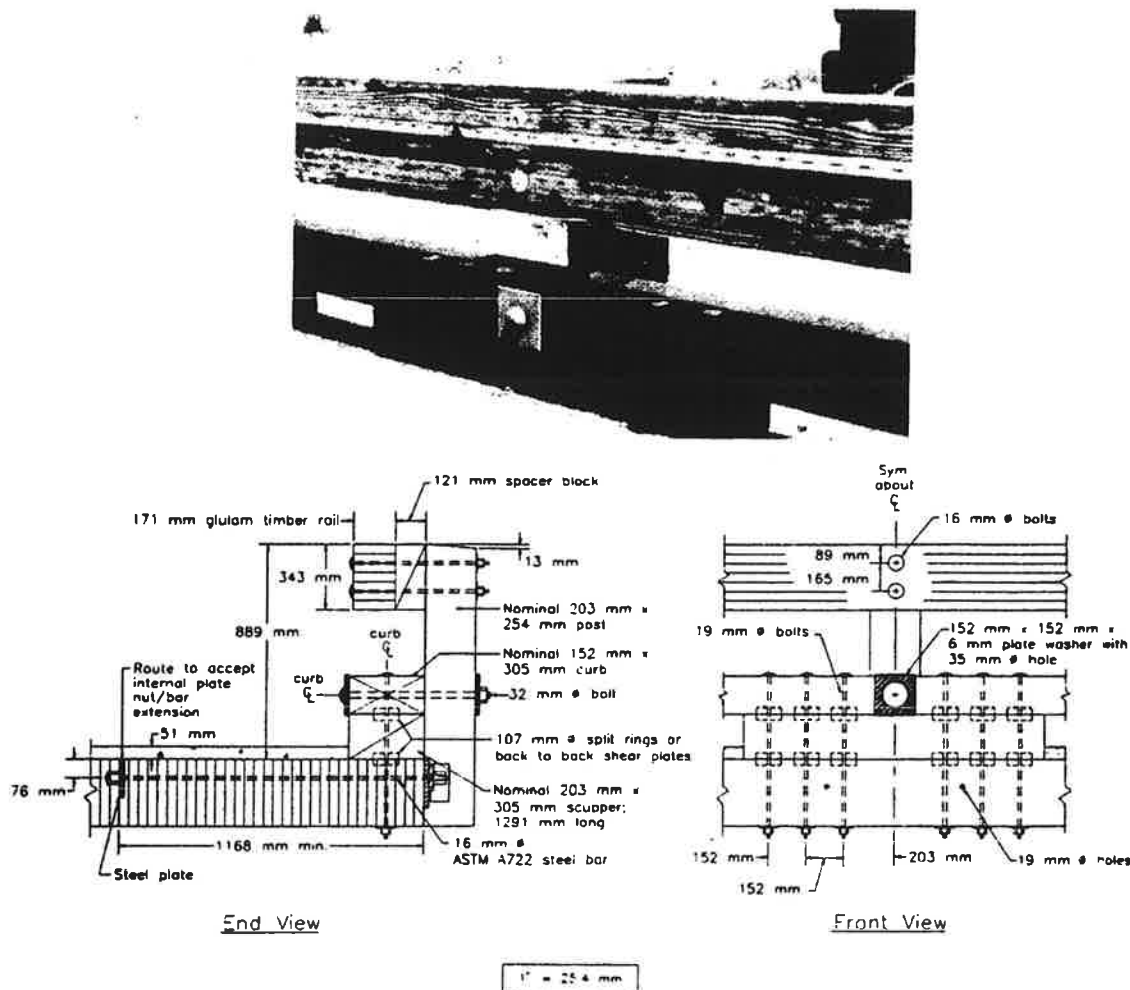


FIGURE 5 Glulam timber bridge railing successfully crash tested to NCHRP Report 350 TL-4 (photograph taken before testing).

on some railings to assess vehicle impact forces transmitted to the bridge railing and superstructure.

RESULTS AND DISCUSSION

As a result of the development and testing program, five bridge railings were successfully developed and tested for longitudinal wood decks. Three of these railings were tested at PL-1, one was tested at PL-2, and one was tested at TL-4. Each railing was tested on the glulam timber deck and is adaptable to the spike-laminated and stress-laminated decks. All designs used posts spaced 1.9 m (6.25 ft) on center and high-strength steel bars through a portion of the bridge deck to act as reinforcement in distributing railing loads without dam-

age to the bridge. Glulam timber for the rail members was Combination 2 Douglas fir as given in the AASHTO *Standard Specifications for Highway Bridges* (1), treated with pentachlorophenol in heavy oil to AWPAC14 requirements (8). Sawn lumber for posts, curbs, scuppers, and spacer blocks was No. 1 Douglas fir (1), treated with creosote to AWPAC14 requirements (8).

A detailed discussion of the testing and results for each railing system is beyond the scope of this paper but is presented in detail in previous publications (12,13). Overall, no damage to the test bridge was evident from any of the vehicle impact tests. For the railing systems with glulam timber rails, damage to the railing was primarily gouging and scraping resulting from the vehicle impact. All glulam timber railing remained

intact and serviceable after the tests, and replacement of the railing was not considered necessary. For the steel thrie beam railings, there was permanent deformation in the rail and post in the vicinity of the impact location. This would necessitate replacement of specific railing and post members, but damage was relatively minor considering the severity of the impact. A brief description of each railing design follows.

PL-1 Railings

The three tested PL-1 railings included a glulam timber railing with curb, a glulam timber railing without curb, and a steel thrie beam railing. Photographs and drawings of the PL-1 railings are shown in Figures 2 and 3, respectively.

The glulam timber railing with curb consisted of a single glulam timber railing mounted on a sawn lumber post. The post was connected with a single bolt to a lumber curb that was supported by scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts and timber connectors.

The glulam timber railing without curb consisted of a single glulam timber railing mounted on a sawn lumber post. The lower portion of the post was placed in a steel box that was attached to the bridge deck with high strength steel bars.

The steel railing consisted of a 10-gauge steel thrie beam railing mounted to a steel, wide flange post. The lower end of the post was bolted to a steel plate that was connected to the bridge deck with high-strength steel bars.

PL-2 Railing

The one PL-2 railing included a steel thrie beam railing, as shown in Figure 4. The steel railing was a modified version of that tested at PL-1. Minor changes in the railing geometry and the addition of a steel channel section above the rail element were necessary to resist the increased loads at PL-2.

TL-4 Railing

The one TL-4 railing included a glulam timber railing with curb, as shown in Figure 5. The railing consisted of a single glulam timber railing mounted on a sawn lumber post and was a modification of the curb system tested at PL-1. Because of the greater loads at TL-4, railing and post sizes were increased, as were bolts and timber connectors attaching the curb and scupper to the bridge deck.

CONCLUDING REMARKS

This program clearly demonstrates that crashworthy railing systems are feasible for longitudinal wood decks. Even at high-impact conditions required by AASHTO PL-2 and *NCHRP Report 350 TL-4*, the railing systems performed well with no significant damage to the bridge superstructure. The development of crashworthy railing systems has overcome a significant barrier to the use of longitudinal deck wood bridges.

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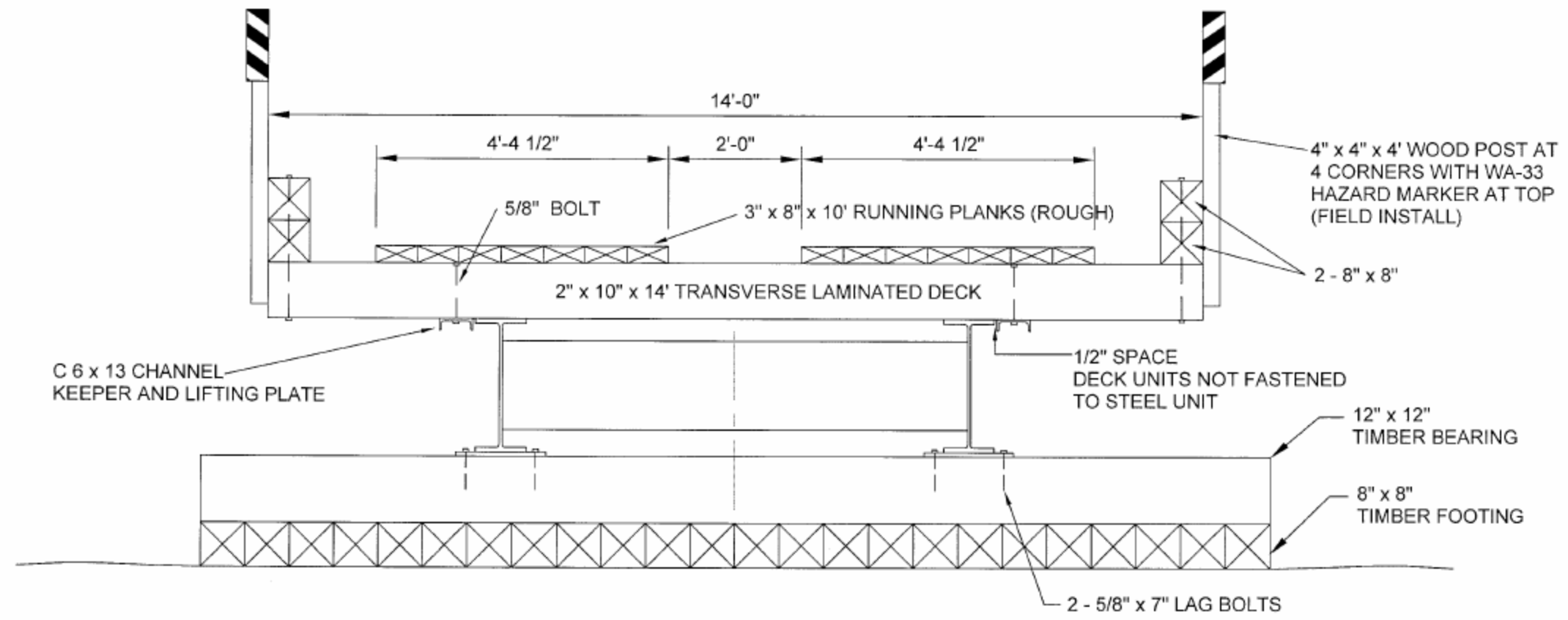
F Appendix F - Sample Drawings of Crash-Tested Barriers for Low Volume Road Bridges



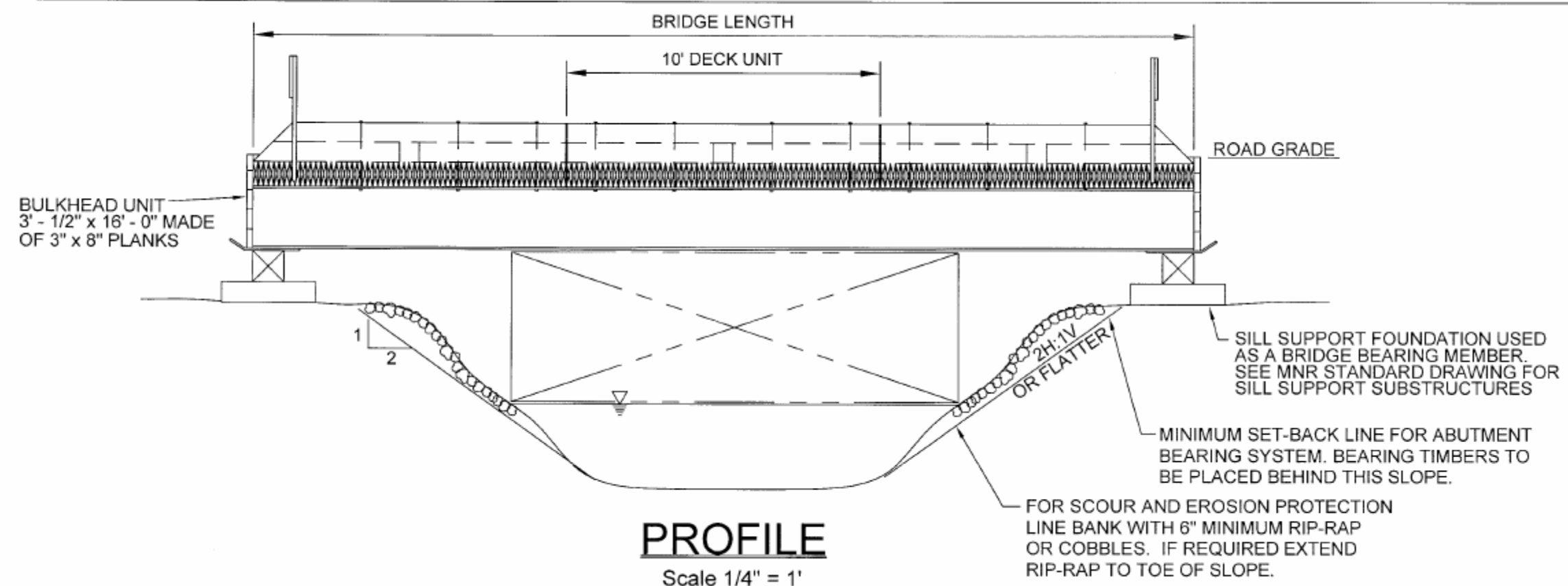
NOTES:
 SEE DWG. 1 OF 5 FOR GENERAL NOTES.
GUIDELINES FOR UNFACTORED SOIL CAPACITY

- CLAY SOILS**
1. FOR VERY SOFT, SOFT AND FIRM CLAYS (CAN BE PENETRATED AN INCH BY THE THUMB WITH MODERATE EFFORT: CONSULT A PROFESSIONAL ENGINEER FOR DESIGN.
 2. FOR STIFF CLAY (READILY INDENTED BY THE THUMB BUT PENETRATED ONLY WITH GREAT EFFORT) - 40 FT. MAX. BRIDGE SPAN
 3. FOR VERY STIFF AND HARD CLAYS (THOSE INDENTED BY THUMB NAIL) - 40 FT. MAX. BRIDGE SPAN.

- GRANULAR SOILS**
1. FOR LOOSE AND COMPACT SANDS (EASILY PENETRATED WITH A 3/8"Ø REINFORCING ROD DRIVEN WITH A 5-LB HAMMER: CONSULT A PROFESSIONAL ENGINEER FOR DESIGN.
 2. FOR DENSE SAND, COMPACT AND DENSE WELL GRADED - SAND MIXTURES, GLACIAL TILL, ROCK FILL AND BEDROCK - 40 FT. MAX. BRIDGE SPAN.



CROSS SECTION
 Scale 1/2" = 1'



PROFILE
 Scale 1/4" = 1'

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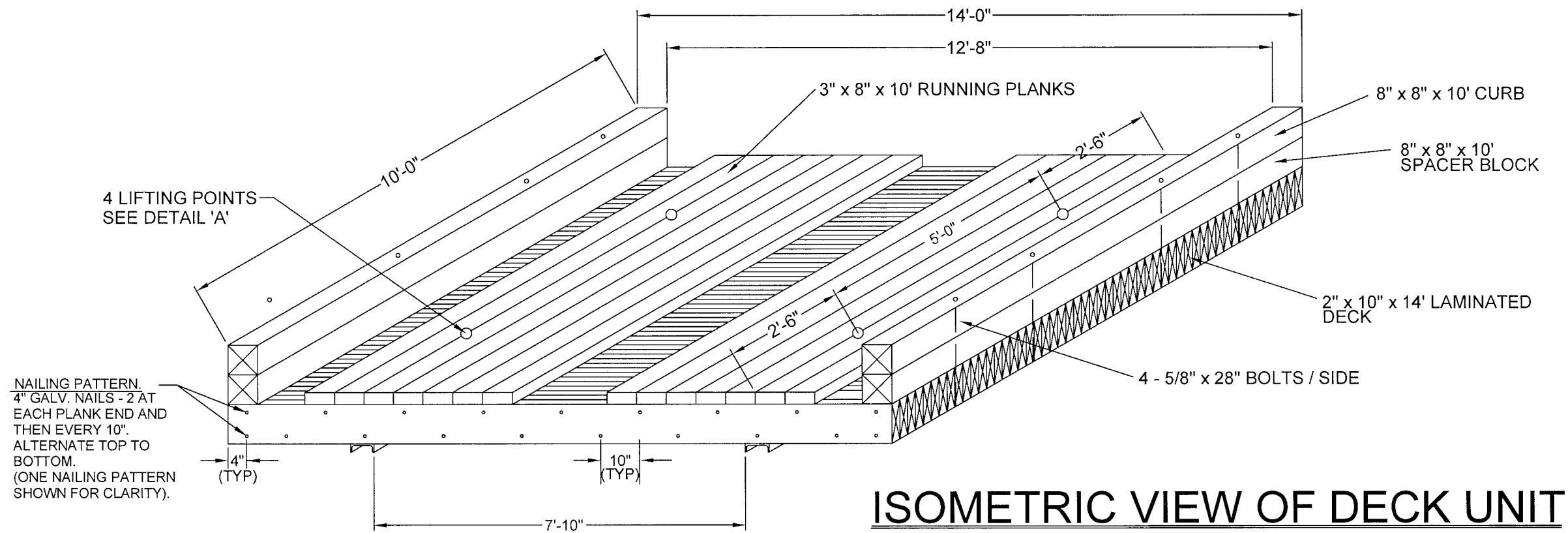
Ministry of Natural Resources



PROJECT PORTABLE 2 STRINGER BRIDGE (IMPERIAL)

DRAWING TITLE CROSS SECTION & PROFILE

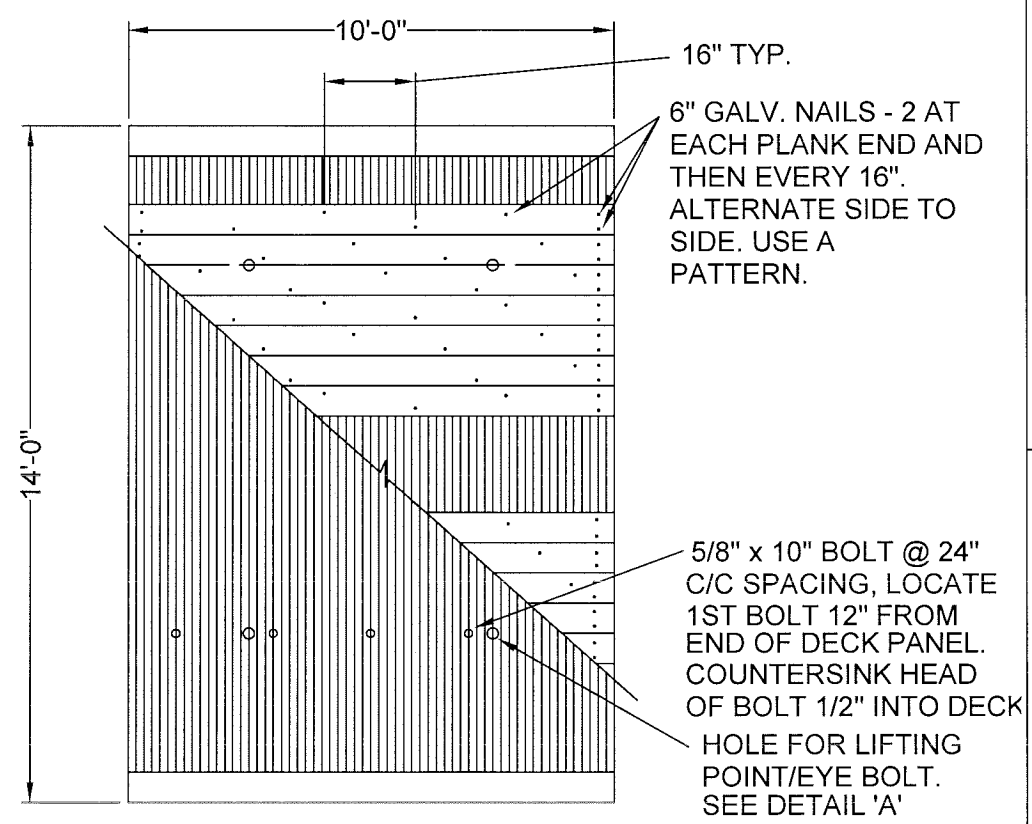
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DRAWING 2 of 5	REV 0



ISOMETRIC VIEW OF DECK UNIT

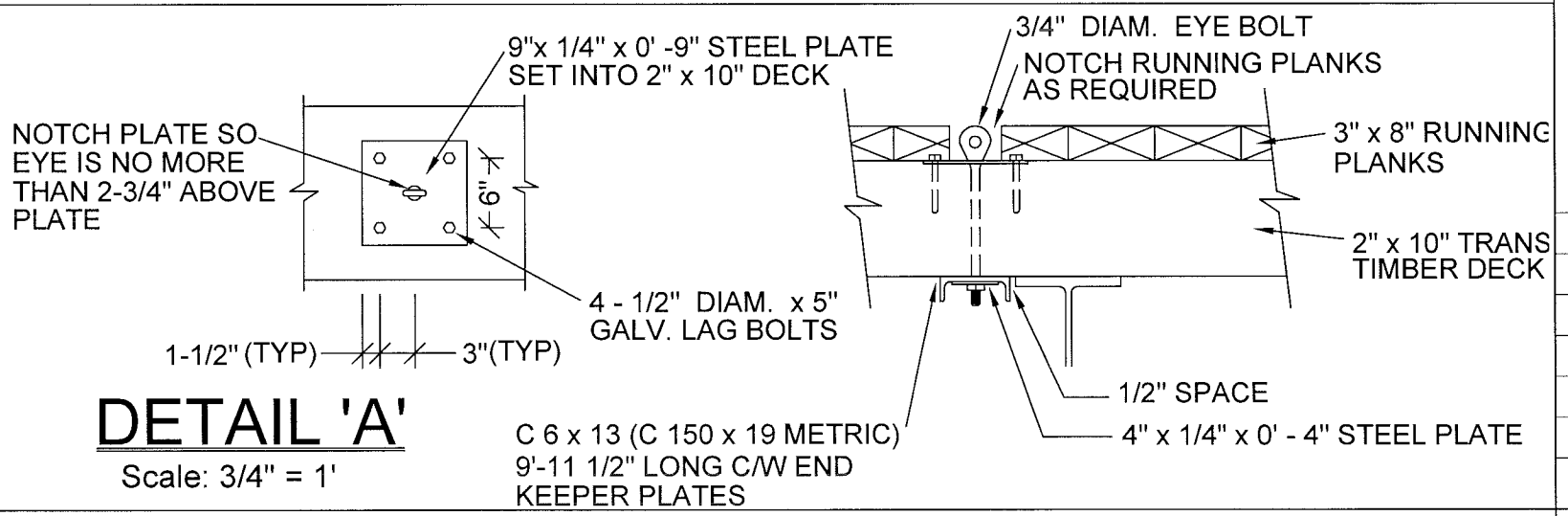
Scale: 3/8" = 1'

- NOTES:**
- SEE DWG. 1 OF 5 FOR GENERAL NOTES.
 - 1) DECK UNIT WEIGHT IS 6,600 LBS.
 - 2) BULKHEAD UNIT WEIGHT IS 600 LBS.



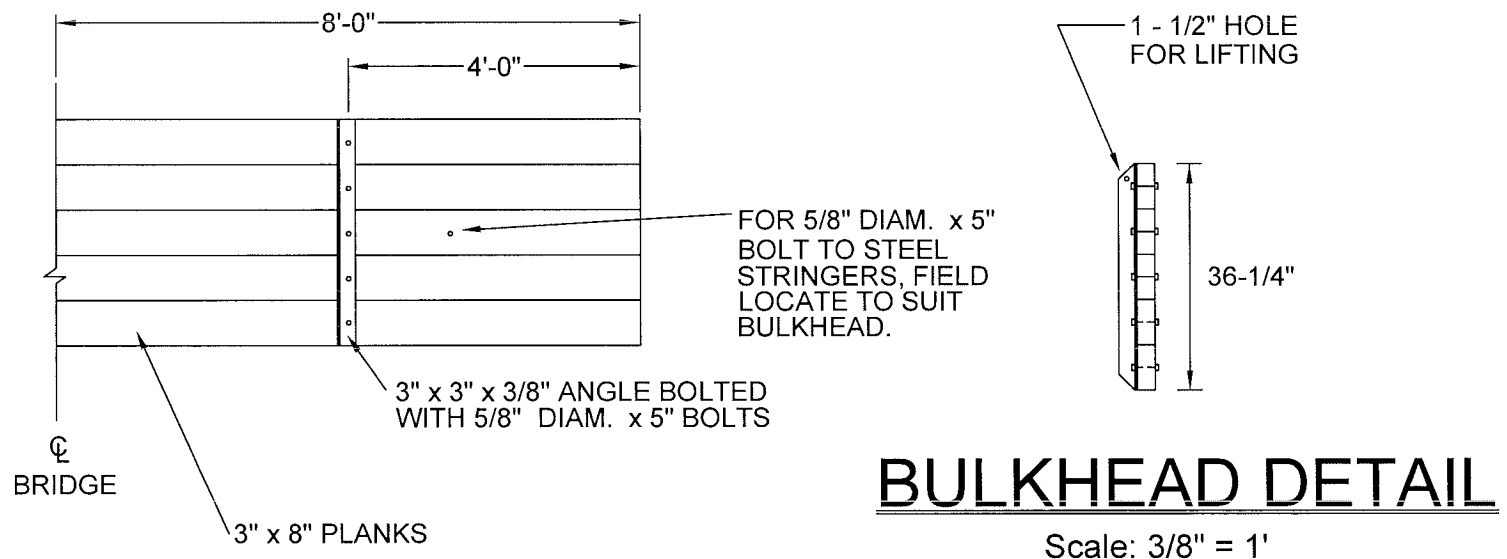
BOLT AND NAIL DETAIL

Scale: 1/4" = 1'



DETAIL 'A'

Scale: 3/4" = 1'



BULKHEAD DETAIL

Scale: 3/8" = 1'

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Ministry of Natural Resources

PROJECT: PORTABLE 2 STRINGER BRIDGE (IMPERIAL)

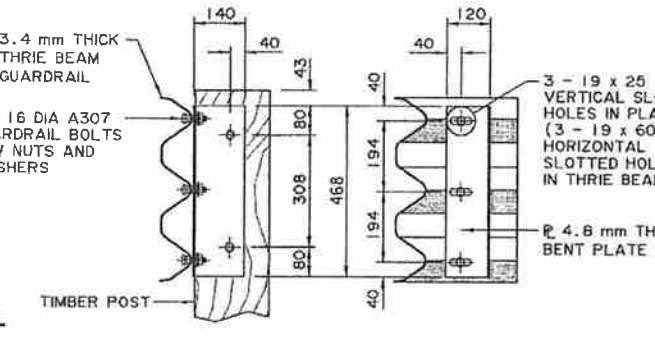
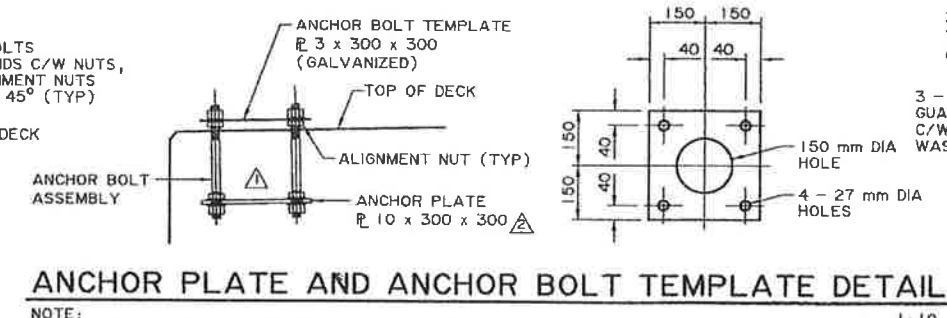
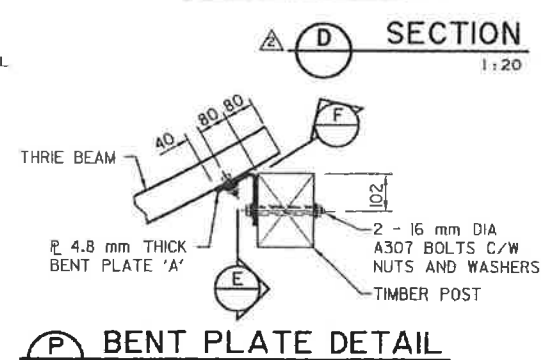
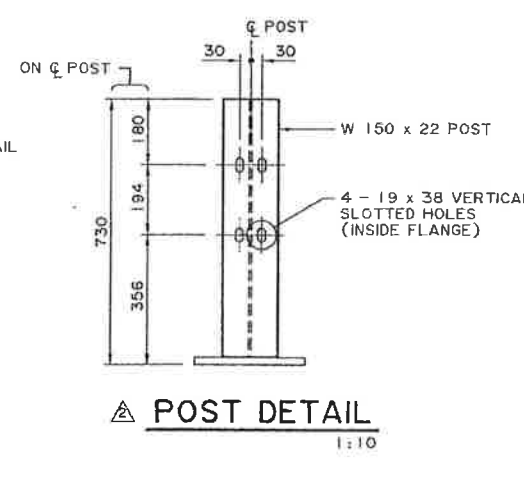
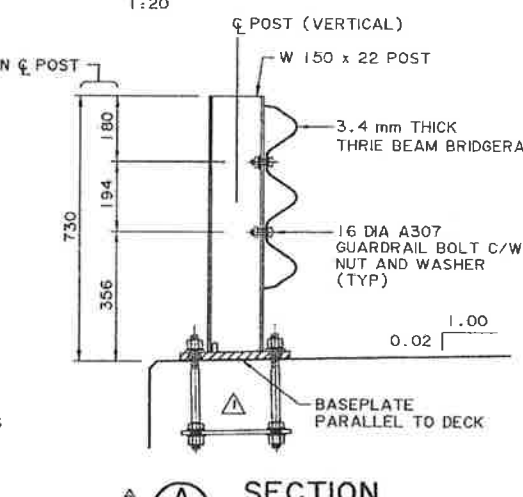
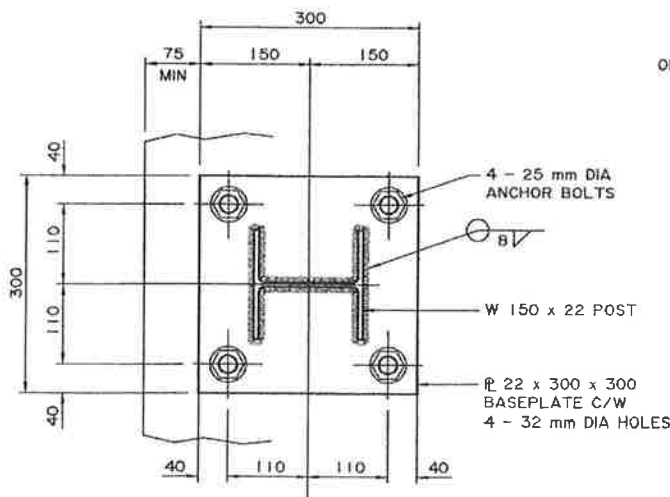
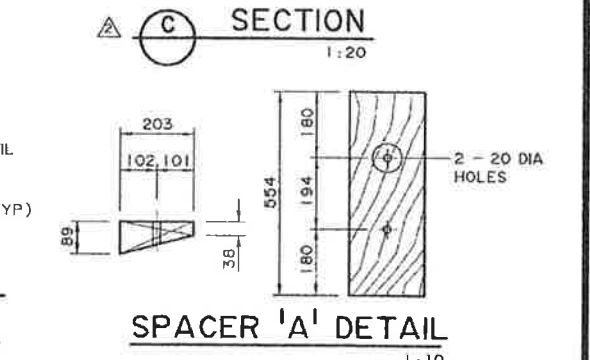
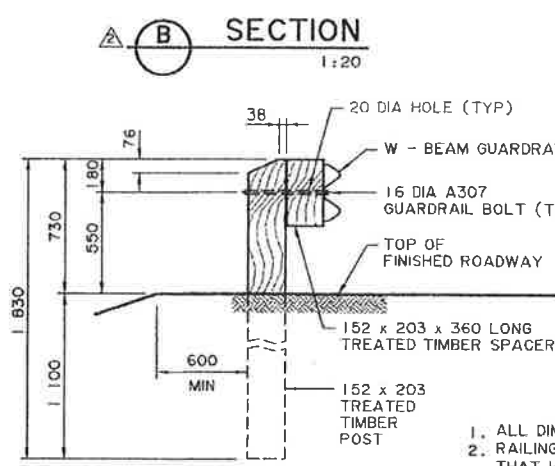
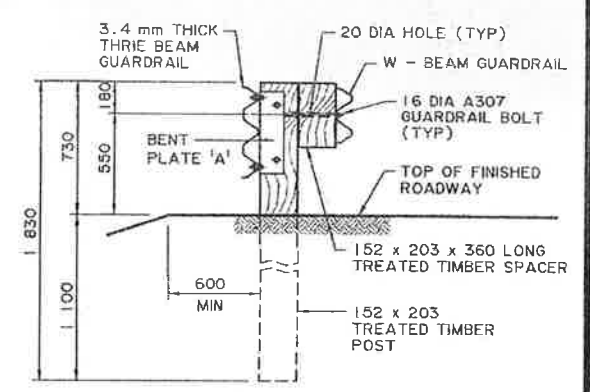
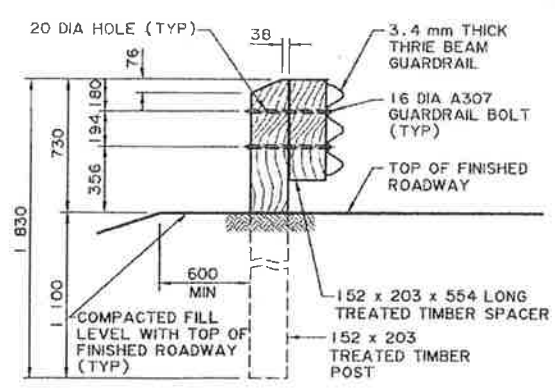
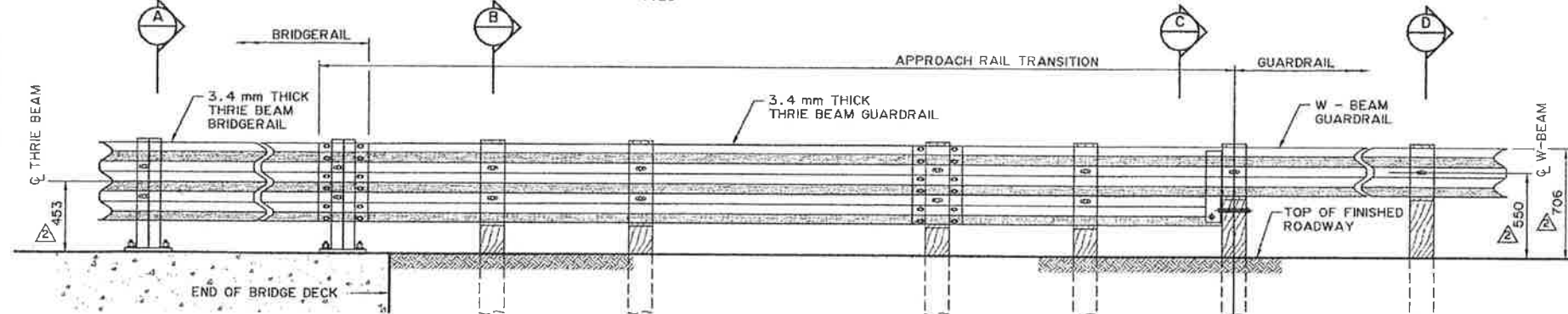
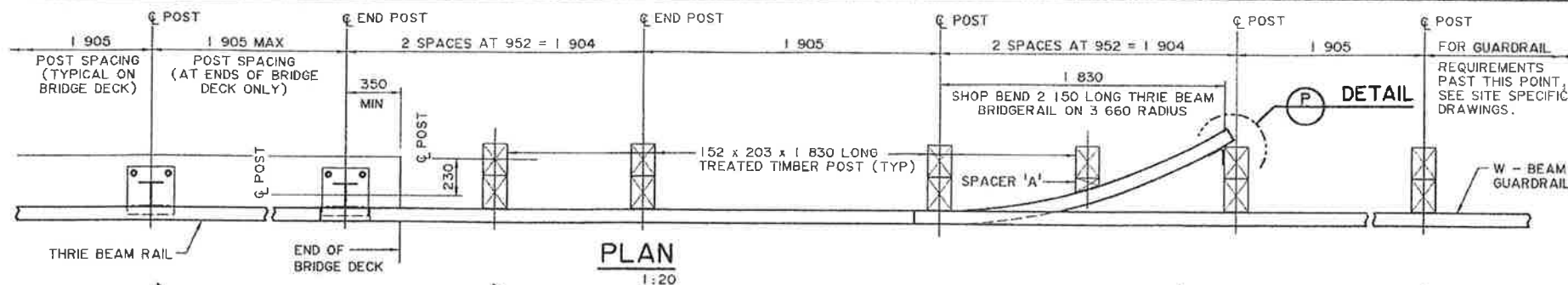
DRAWING TITLE: MODULAR DECK UNITS

SCALE: AS NOTED

DATE: February 6, 2008

DRAWING: 3 of 5

REV: 0



GENERAL NOTES

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
2. RAILING CONFIGURATION IS BASED ON A RAILING CONFIGURATION THAT HAS BEEN CRASH TESTED AND MEETS THE REQUIREMENTS OF PERFORMANCE LEVEL 1 OF THE AASHTO GUIDE SPECIFICATIONS FOR BRIDGE RAILING, 1989.
3. RAILING SHALL NOT BE USED WITH CURBS.
4. RAILING SHALL BE USED FOR CLEAR ROADWAYS LESS THAN 9.0 m ONLY.
5. DESIGN OF DECK REBAR SHALL BE CARRIED OUT ON A SITE SPECIFIC BASIS TO DEVELOP THE CAPACITY OF THE BRIDGERAIL POSTS, BASED ON $F_y = 350 \text{ MPa}$.

FABRICATION

1. BRIDGERAIL, INCLUDING APPROACH RAIL TRANSITION, SHALL CONFORM TO THE CURRENT REQUIREMENTS OF THE SPECIFICATIONS FOR BRIDGE CONSTRUCTION SECTION 12 - BRIDGERAIL AND SECTION 14 - GUARDRAIL.
2. ALL PLATE STEEL AND STRUCTURAL SHAPES SHALL CONFORM TO CSA G40.21 GRADE 350W OR ASTM A36.
3. ALL ANCHOR BOLTS SHALL CONFORM TO THE REQUIREMENTS OF ASTM A193 GRADE B7 ($F_y = 725 \text{ MPa}$, $F_u = 860 \text{ MPa}$). ALL NUTS AND WASHERS SHALL CONFORM TO A325. GALVANIZING SHALL STRICTLY FOLLOW THE FOLLOWING PROCEDURE WITH THE PRESENCE OF THE CONSULTANT:
 - BRUSH BLAST ANCHOR BOLTS TO REMOVE MILL SCALE AND OIL AFTER THREADING ENDS.
 - FLASH PICKLING NOT TO EXCEED 5 MINUTES.
 - QUICK DRY PRIOR TO HOT-DIP GALVANIZING (DO NOT STORE IN FLUX OR ACID RINSE).
4. ALL W-BEAM AND THRIE BEAM GUARDRAIL (INCLUDING W-THRIE BEAM TRANSITION SECTION) SHALL HAVE A MIN. YIELD STRENGTH OF 345 MPa.
5. TIMBER POSTS AND SPACERS SHALL BE COAST DOUGLAS FIR, PACIFIC COAST HEMLOCK OR LODGEPOLE PINE CONFORMING TO THE STRESS GRADE "SELECT STRUCTURAL POSTS AND TIMBERS" (NLGA PARAGRAPH 131 a).
6. ALL WELDING SHALL CONFORM TO CURRENT AWS SPECIFICATION D1.5.
7. ALL STEEL MATERIALS SHALL BE HOT-DIP GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH CSA G164 UNLESS NOTED OTHERWISE.

ERECTION

1. BRIDGERAIL ANCHOR BOLTS SHALL BE TIGHTENED AN ADDITIONAL $\frac{1}{2}$ TURN OF THE NUT PAST THE "SNUG TIGHT" CONDITION.
2. ALL POSTS SHALL BE PERPENDICULAR TO GRADE.
3. ALL DIMENSIONS ARE MEASURED PARALLEL TO TOP OF BRIDGE DECK AND ALONG THE CENTRELINE OF ANCHOR BOLT ASSEMBLIES.
4. LINE AND ELEVATION OF RAIL SHALL HAVE A TOLERANCE OF 6 mm.
5. ALL NON-STANDARD GUARDRAIL LENGTHS SHALL BE SAW CUT TO SUIT AND ALL NON-STANDARD GUARDRAIL HOLES SHALL BE DRILLED. FLAME CUTTING OF GUARDRAIL SHALL NOT BE ALLOWED. APPLY TWO COATS OF ZINC RICH PAINT ON AREAS DAMAGED BY SAW CUTTING OR DRILLING.

ANCHOR BOLT ASSEMBLY DETAIL

NOTE:
ANCHOR BOLT ASSEMBLY SHALL BE MINIMUM 25 mm CLEAR FROM BOTTOM OF DECK.

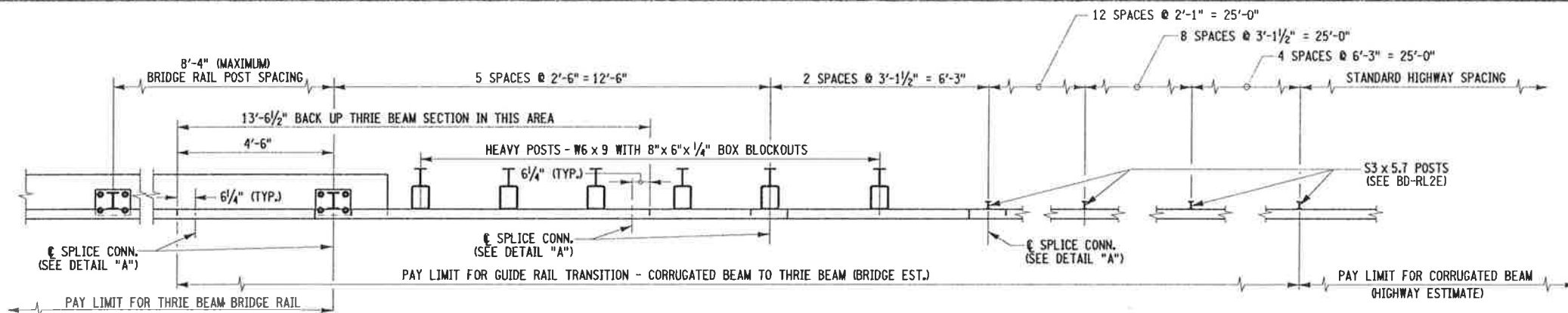
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				<p>APPROVED EXECUTIVE DIRECTOR TECHNICAL STANDARDS BRANCH</p>	<p>ORIGINAL SIGNED BY TIM HAWNT</p>
<p>DATE: 2007-02-14 APPROACH RAIL HEIGHT</p>				<p>DATE: NOV 22, 2000</p>	<p>DATE: 2000-03-27</p>
<p>DATE: 2004-03-01 GENERAL NOTES</p>				<p>DATE: NOV 22, 2000</p>	<p>DATE: NOV 22, 2000</p>
<p>REV DATE REVISIONS BY</p>				<p>DATE: NOV 22, 2000</p>	<p>DATE: NOV 22, 2000</p>

Alberta INFRASTRUCTURE

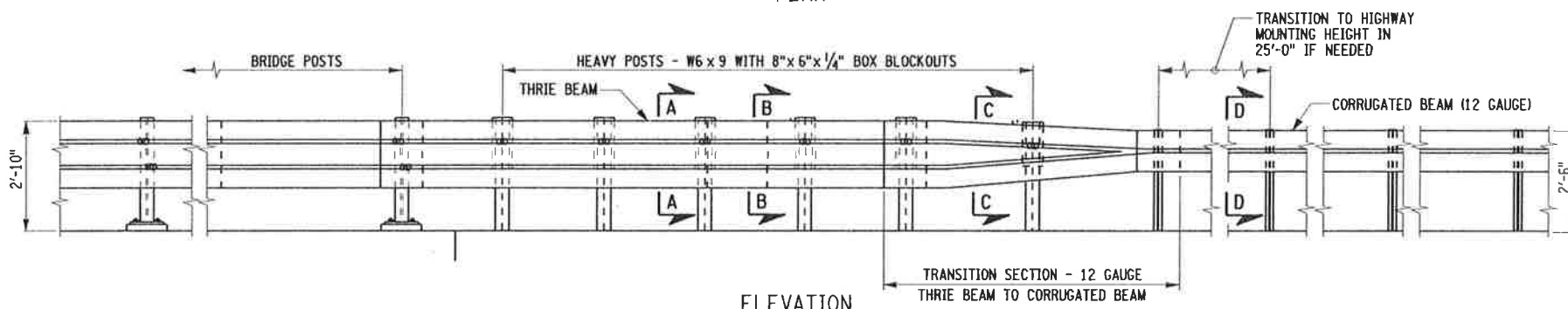
PL-1 LOW HEIGHT THRIE BEAM BRIDGERAIL

DATE: 2000-03-27 SHEET: S-1653-00 DRAWING: 831471

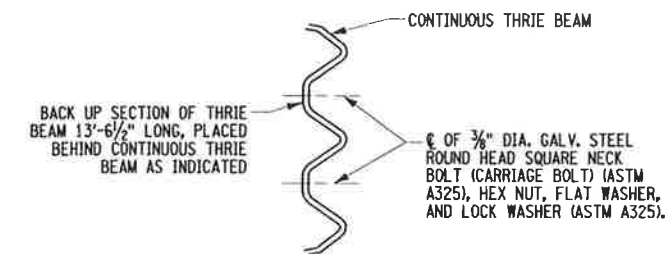
NOTE TO THE DESIGNER:
 THE RAIL SYSTEM SHOWN IN THESE DETAILS IS INTENDED FOR USE ONLY ON SHORT, LOW LEVEL BRIDGES THAT CARRY LOW VOLUME (LESS THAN 500 AADT), LOCAL RURAL HIGHWAYS WHERE A SMALL NUMBER OF HEAVY VEHICLES ARE EXPECTED AND SPEEDS ARE EITHER POSTED OR REDUCED. FOR SELECTION GUIDANCE, REFER TO THE PERFORMANCE LEVEL ONE (PL-1) CRITERIA AS DEFINED IN THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS OR THE 1989 AASHTO GUIDE SPECIFICATIONS FOR BRIDGE RAILING. THIS RAILING SYSTEM IS NOT RECOMMENDED FOR USE ON BRIDGES WHOSE STRUCTURE LENGTH BETWEEN ABUTMENTS EXCEEDS 50 FEET.



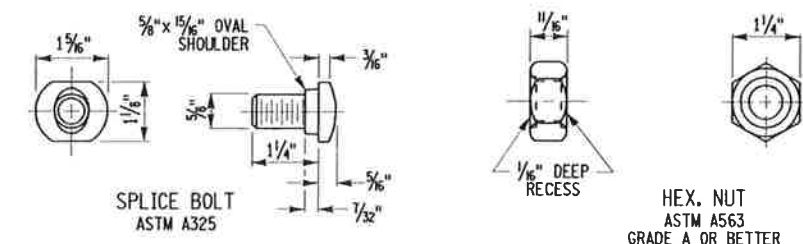
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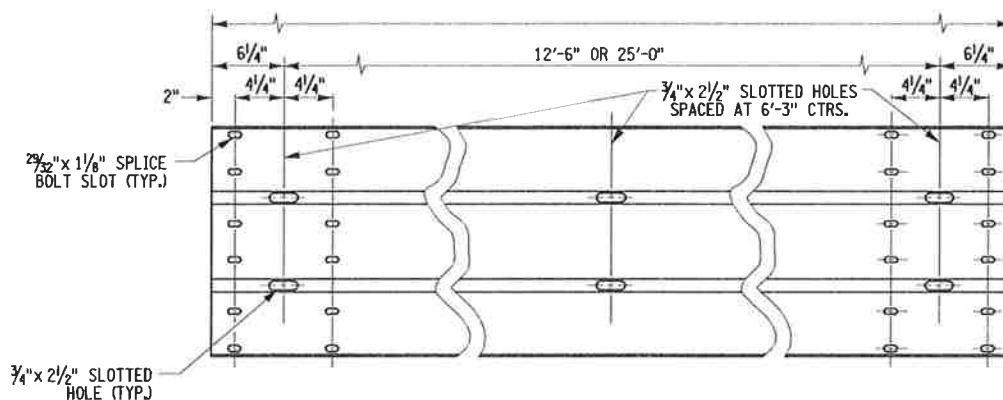
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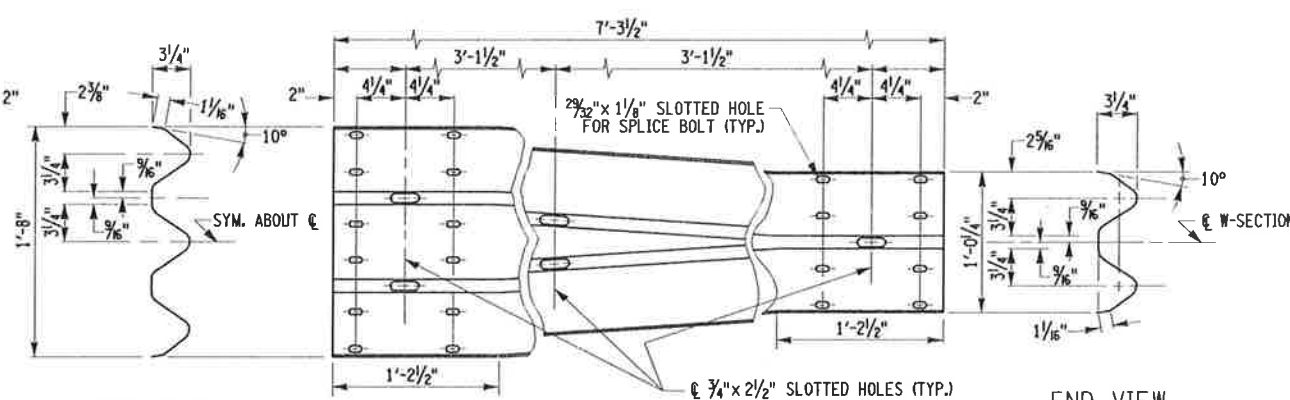
SECTION A-A
BACK UP THRIE BEAM DETAIL



DETAIL "A"
BEAM SPLICE HARDWARE
NOT TO SCALE



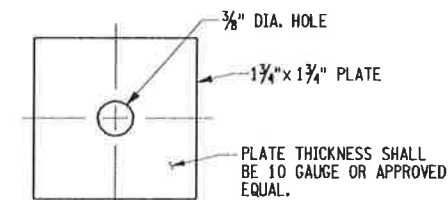
TYPICAL THRIE BEAM SECTION
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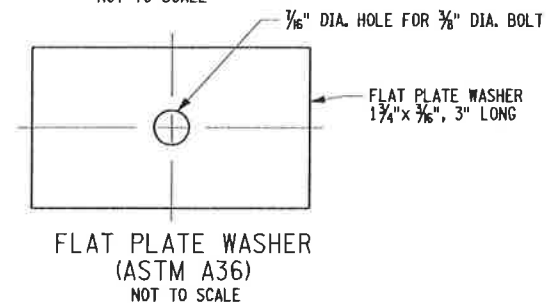
END VIEW
OF TRANSITION

TRANSITION SECTION
NOT TO SCALE

END VIEW
OF TRANSITION

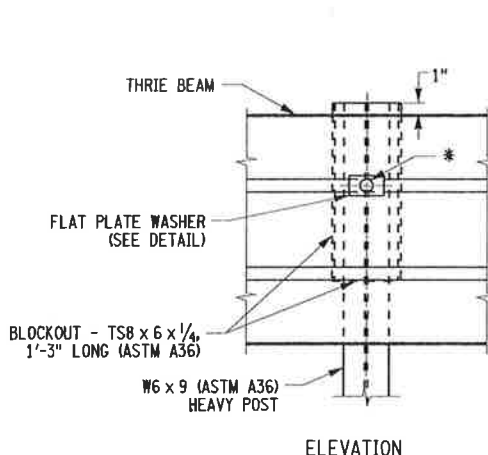


SQUARE WASHER
(ASTM A36)
NOT TO SCALE



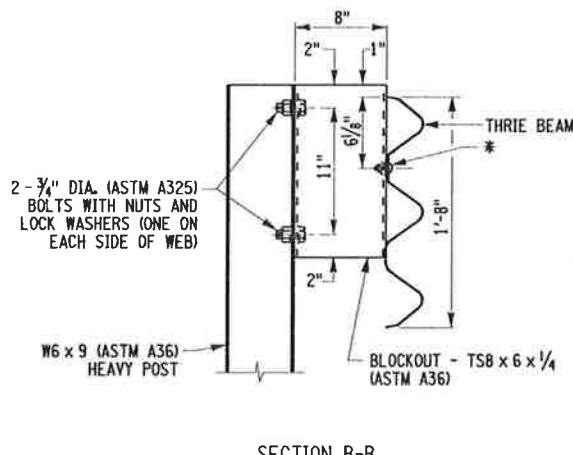
FLAT PLATE WASHER
(ASTM A36)
NOT TO SCALE

NOTE: DETAILS ON THE DRAWINGS LABELED AS "NOT TO SCALE" ARE INTENTIONALLY DRAWN NOT TO SCALE FOR VISUAL CLARITY. ALL OTHER DETAILS, FOR WHICH NO SCALE IS SHOWN, ARE DRAWN PROPORTIONAL AND ARE FULLY DIMENSIONED.



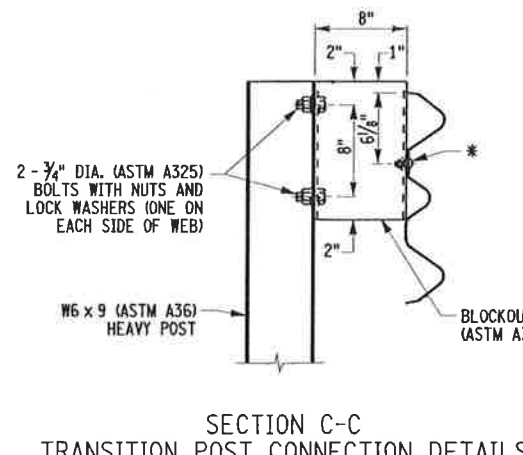
ELEVATION

TYPICAL HEAVY POST CONNECTION DETAILS
NOT TO SCALE

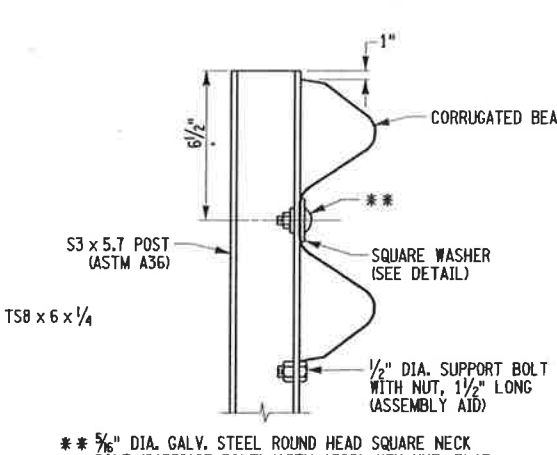


SECTION B-B

* 3/8" DIA. GALV. STEEL ROUND HEAD SQUARE NECK BOLT (CARRIAGE BOLT) (ASTM A325), HEX NUT, FLAT WASHER, AND LOCK WASHER (ASTM A325).



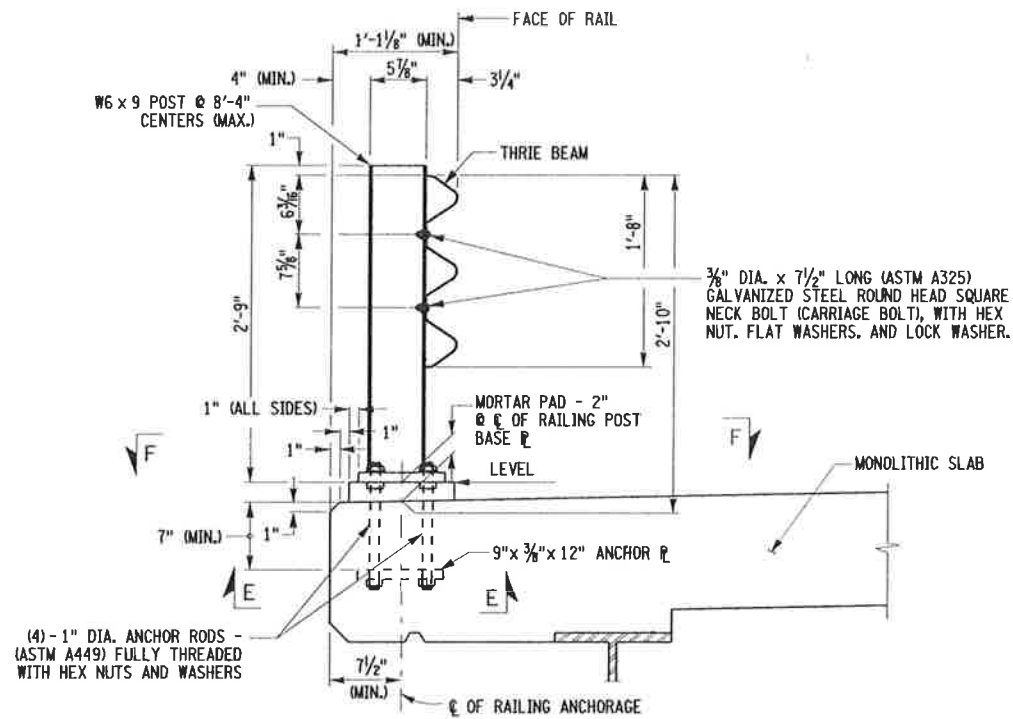
SECTION C-C
TRANSITION POST CONNECTION DETAILS
NOT TO SCALE



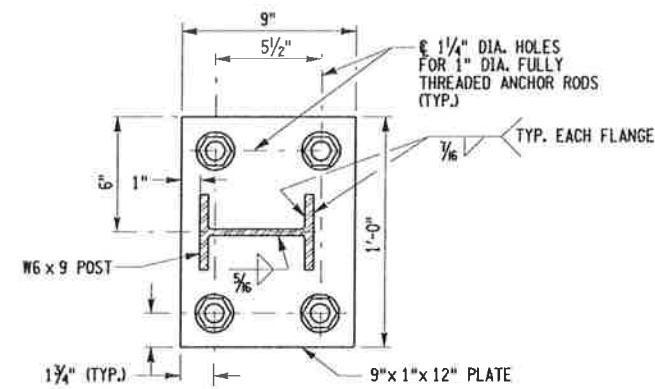
SECTION D-D
TYPICAL S3X5.7 POST CONNECTION DETAILS
NOT TO SCALE

** 5/8" DIA. GALV. STEEL ROUND HEAD SQUARE NECK BOLT (CARRIAGE BOLT) (ASTM A325), HEX NUT, FLAT WASHER, AND LOCK WASHER (ASTM A325).

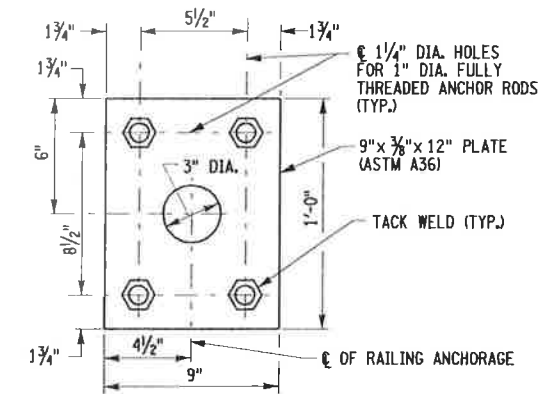
ISSUED 5/01/08	STATE OF NEW YORK DEPARTMENT OF TRANSPORTATION OFFICE OF STRUCTURES
REVISED	
THRIE BEAM BRIDGE RAIL FOR LOW VOLUME (NON-NHS) BRIDGES (1 OF 2)	
APPROVED: 1/18/08 ORIGINAL SIGNED BY GEORGE A. CHRISTIAN, P.E. DEPUTY CHIEF ENGINEER (STRUCTURES)	ISSUED UNDER EB 08-002 EFFECTIVE WITH THE LETTING OF 1/08/09



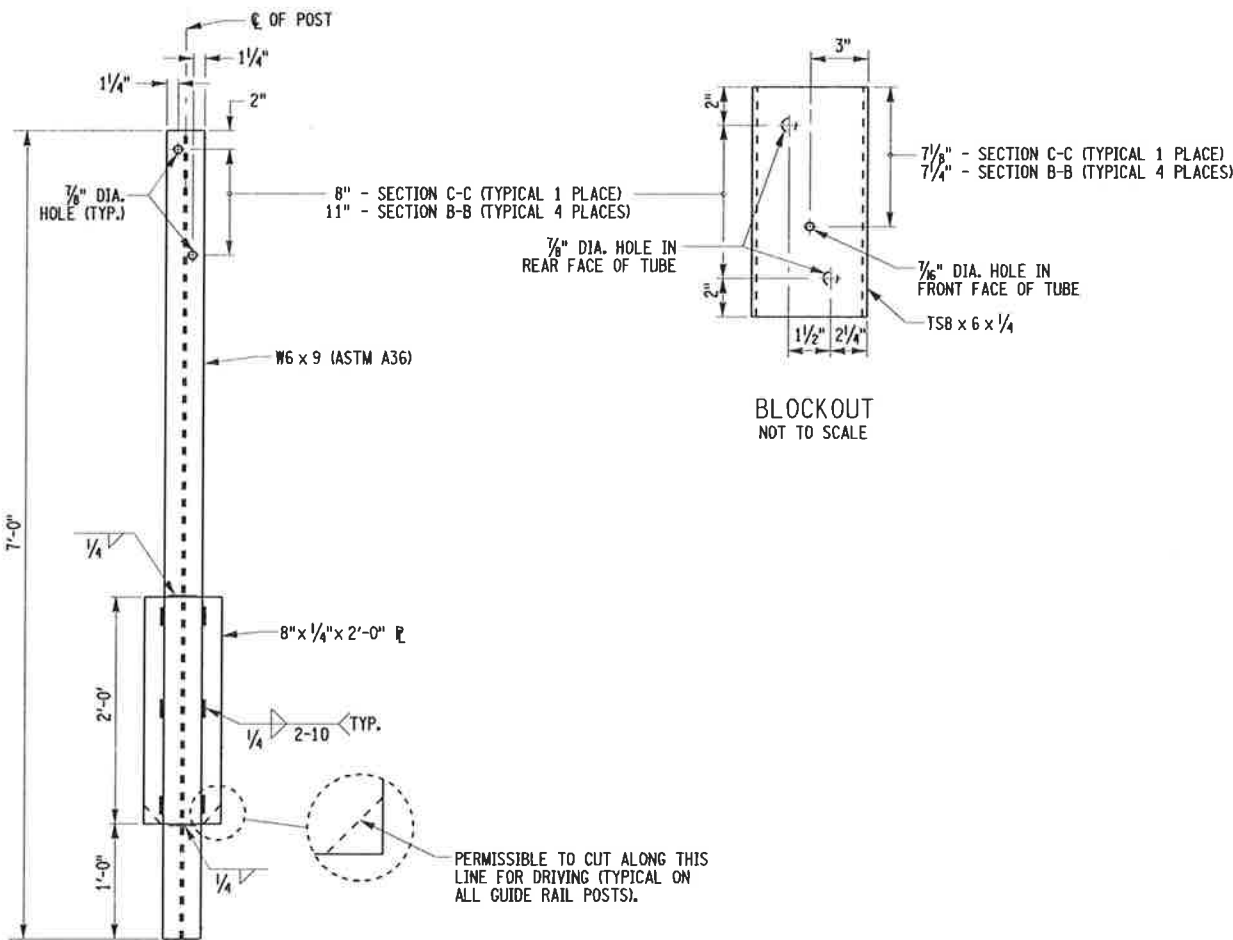
SECTION ON BRIDGE



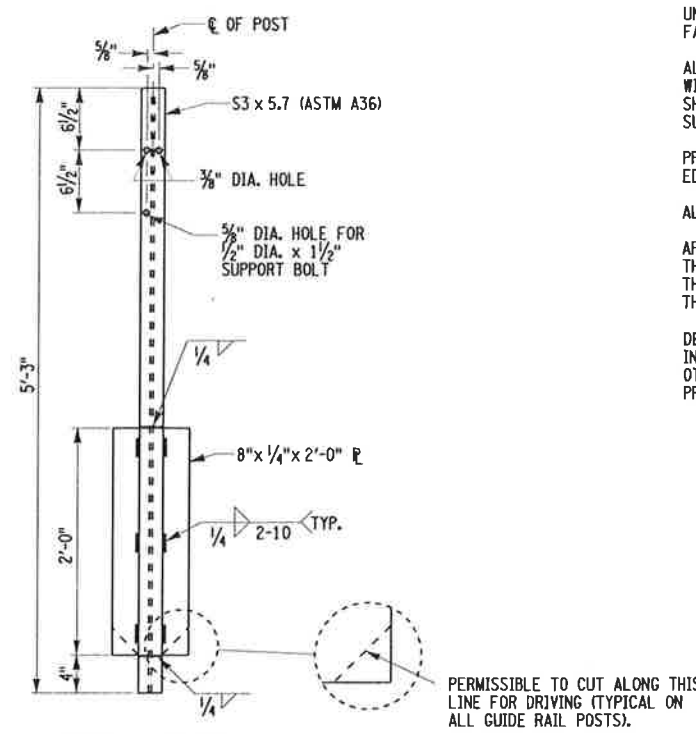
SECTION F-F



SECTION E-E
(ANCHOR PLATE)



FRONT ELEVATION
(TRAFFIC SIDE)
HEAVY POST DETAIL

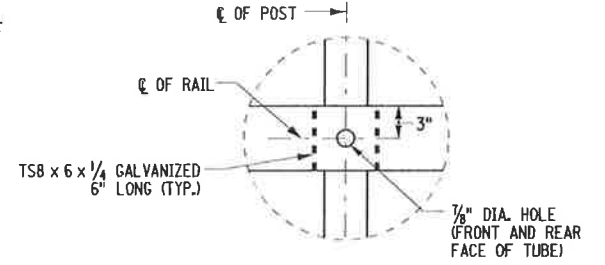
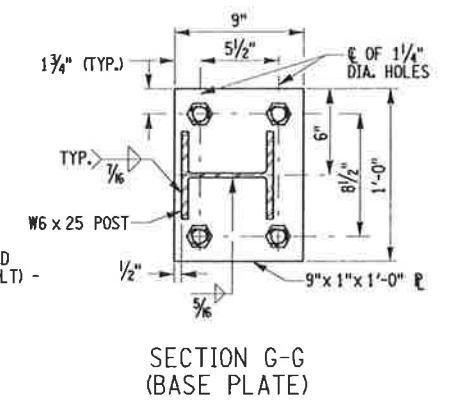
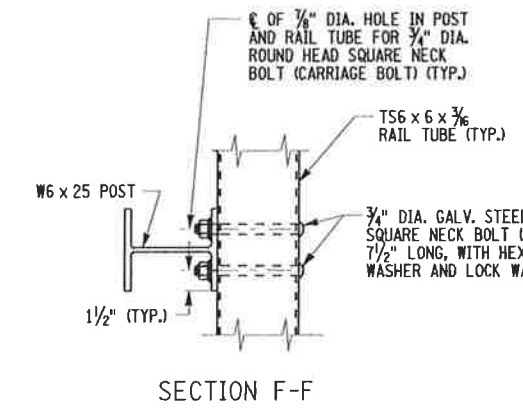
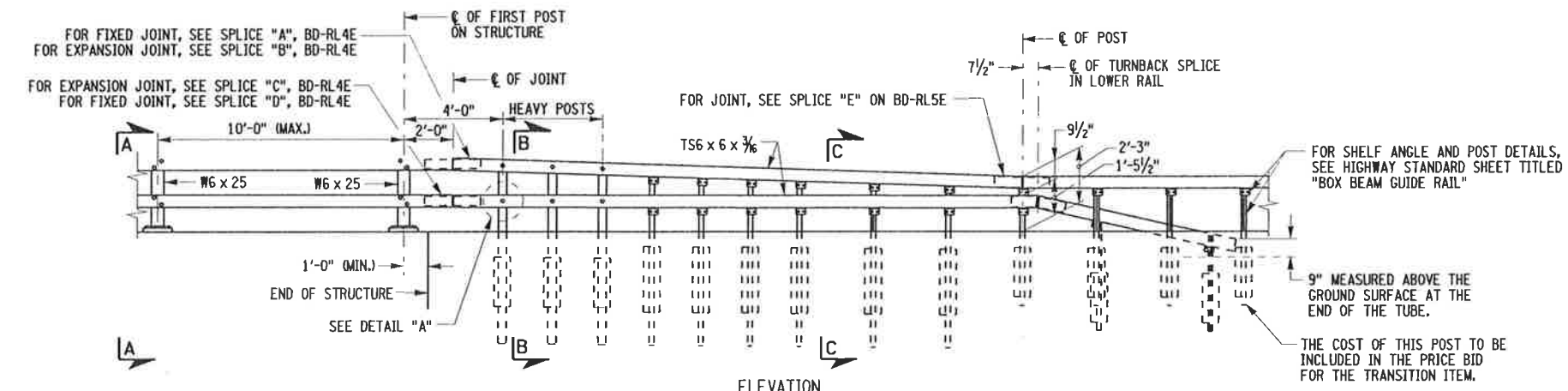
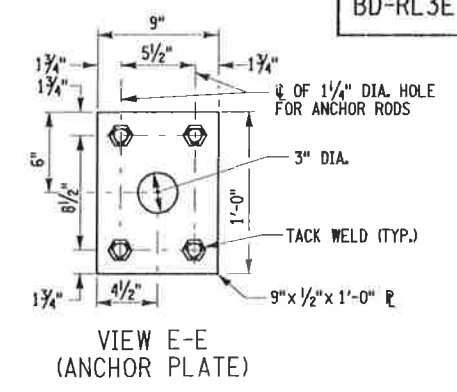
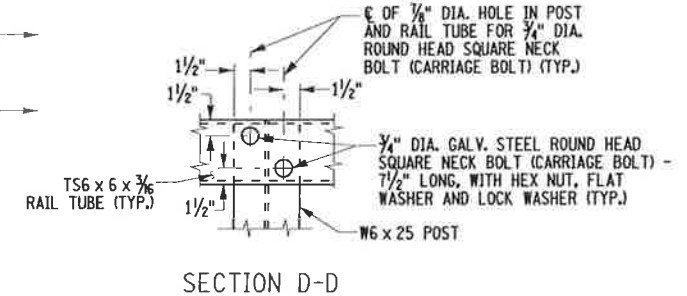
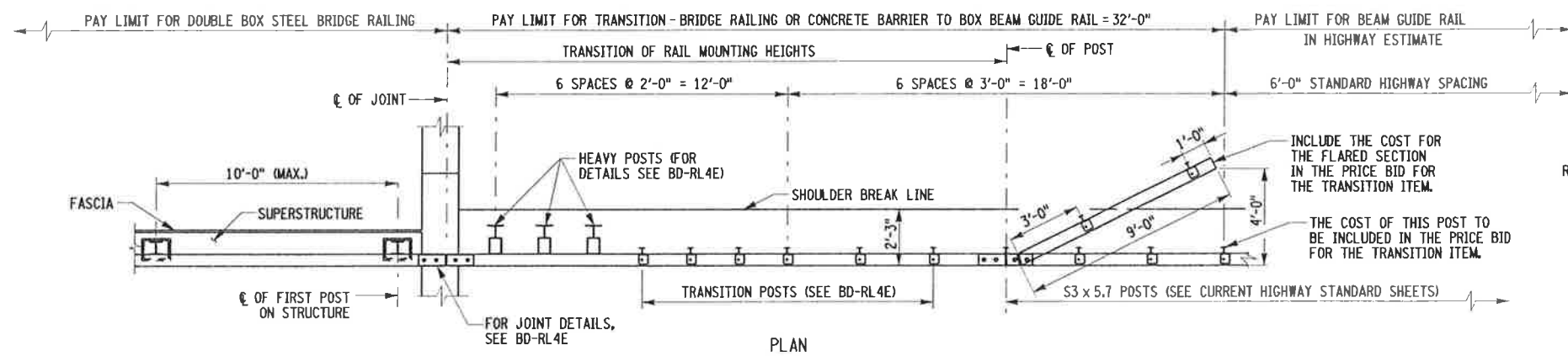


FRONT ELEVATION
(TRAFFIC SIDE)
TYPICAL S3 x 5.7 POST

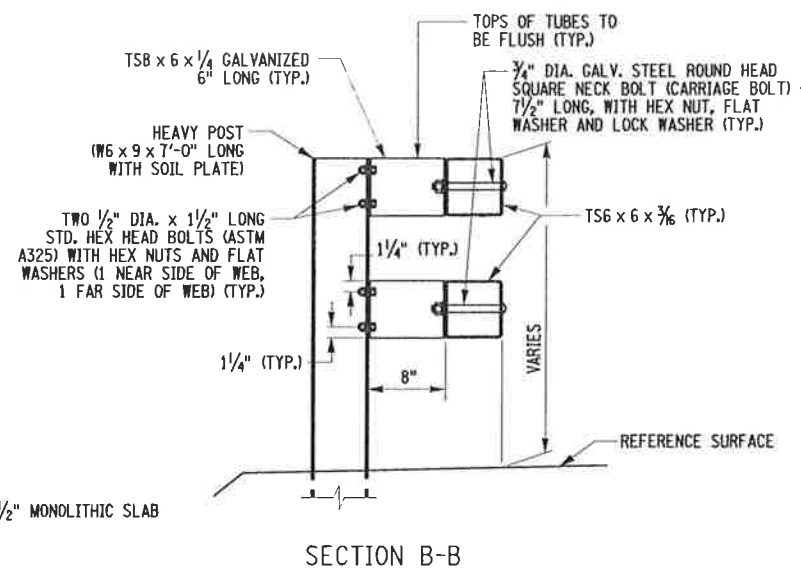
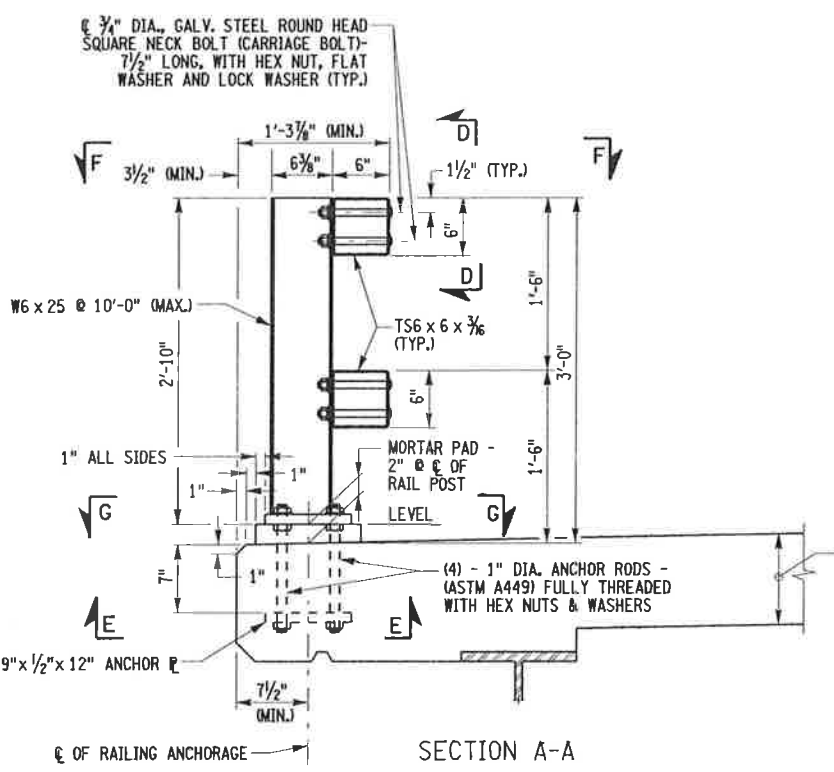
NOTES:

- ALL THRIE BEAM SECTIONS SHALL BE 10 GAUGE.
- THE TRANSITION SECTION FROM CORRUGATED BEAM TO THRIE BEAM SHALL BE 12 GAUGE.
- THE MATERIAL FROM WHICH THE THRIE BEAM IS FABRICATED SHALL CONFORM TO MATERIAL SPECIFICATION 710-20.
- RAILING TO BE FABRICATED AND ERECTED SO THAT THE RAIL SHALL BE PARALLEL TO GRADE AND THE RAILING POSTS ARE TRULY VERTICAL.
- THE BASE PLATES SHALL BE PERPENDICULAR TO THE POSTS UNLESS OTHERWISE NOTED.
- THRIE BEAMS, SQUARE WASHERS, FLAT PLATE WASHERS, POST BEAMS, BASE PLATES, CARRIAGE BOLTS, FLAT WASHERS, NUTS, ANCHOR PLATES, AND ANCHOR RODS SHALL BE PAID FOR UNDER THE RAILING ITEM. THE MORTAR PADS SHALL BE PAID FOR UNDER A SEPARATE ITEM.
- UNLESS COVERED BY OTHER SPECIFICATION, ALL DIMENSIONS RELATED TO THE FABRICATION OF THE STEEL RAILING SHALL HAVE A TOLERANCE OF 1/16 INCH.
- ALL COMPONENTS OF THE THRIE BEAM SYSTEM SHALL BE GALVANIZED IN ACCORDANCE WITH MATERIAL SPECIFICATION 719-01 AFTER FABRICATION. ANY FIELD DRILLING SHALL REQUIRE THAT THE GALVANIZING BE REPAIRED IN ACCORDANCE WITH SUBSECTION 719-01 OF THE NEW YORK STATE STANDARD SPECIFICATIONS.
- PRIOR TO GALVANIZING THE ASSEMBLED POST, GRIND ALL EDGES OF PLATES AND CUT EDGES OF THE POST TO A MINIMUM RADIUS OF 1/16 INCH.
- ALL NUTS SHALL BE TORQUED TO THE SATISFACTION OF THE ENGINEER.
- AFTER THE ANCHOR ROD NUTS HAVE BEEN TIGHTENED TO THE SATISFACTION OF THE ENGINEER, THE RODS SHALL BE FLAME CUT ONE (1) INCH ABOVE THE NUT, AND THE FIRST THREE THREADS ABOVE THE NUT SHALL BE DAMAGED AS ORDERED BY THE ENGINEER.
- DETAILS ON THE DRAWINGS LABELED AS "NOT TO SCALE" ARE INTENTIONALLY DRAWN NOT TO SCALE FOR VISUAL CLARITY. ALL OTHER DETAILS, FOR WHICH NO SCALE IS SHOWN, ARE DRAWN PROPORTIONAL AND ARE FULLY DIMENSIONED.

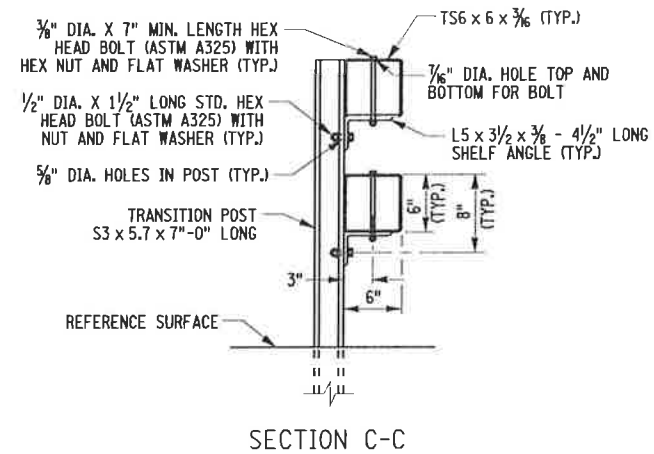
ISSUED 5/01/08	STATE OF NEW YORK DEPARTMENT OF TRANSPORTATION OFFICE OF STRUCTURES
REVISED	
THRIE BEAM BRIDGE RAIL FOR LOW VOLUME (NON-NHS) BRIDGES (2 OF 2)	
APPROVED: 1/18/08 ORIGINAL SIGNED BY GEORGE A. CHRISTIAN, P.E. DEPUTY CHIEF ENGINEER (STRUCTURES)	ISSUED UNDER EB 08-002 EFFECTIVE WITH THE LETTING OF 1/08/09



RAILING TRANSITION TO HIGHWAY BOX BEAM GUIDE RAIL



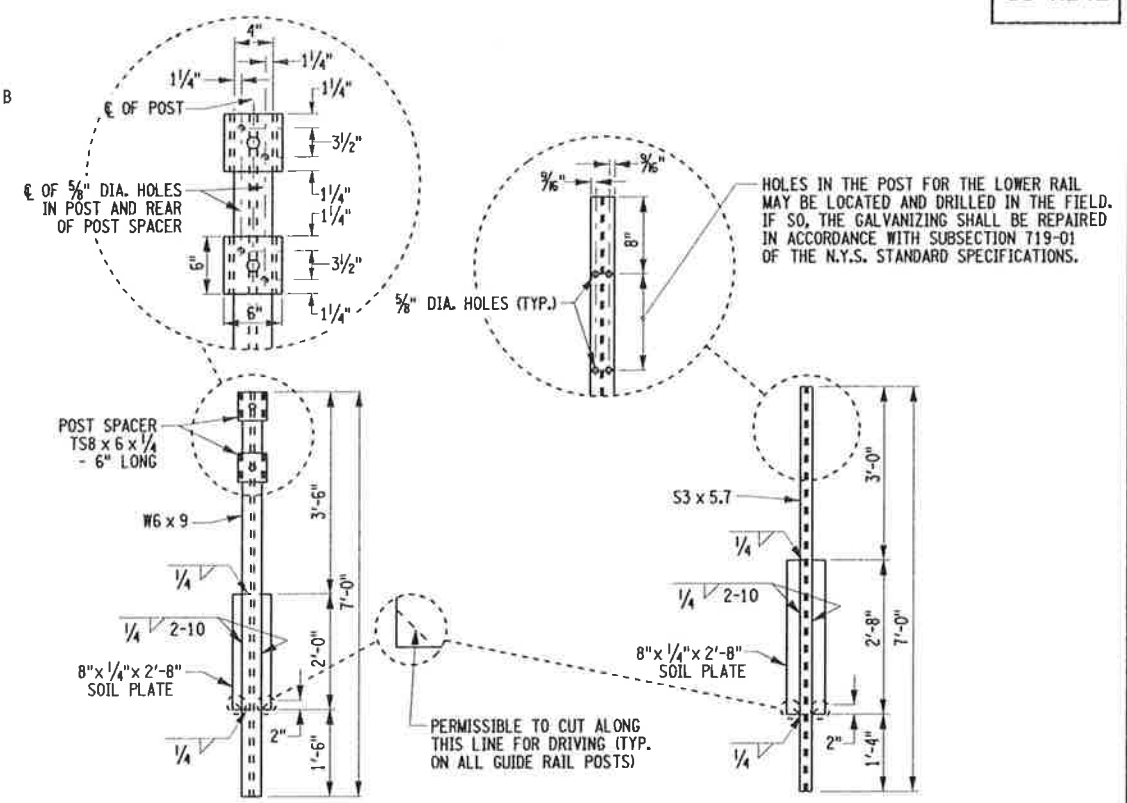
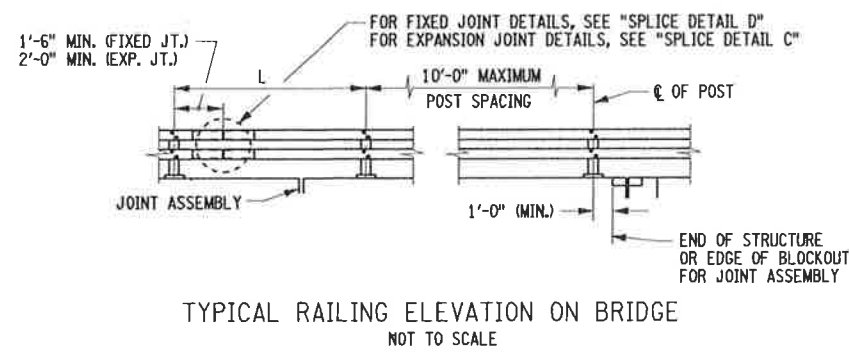
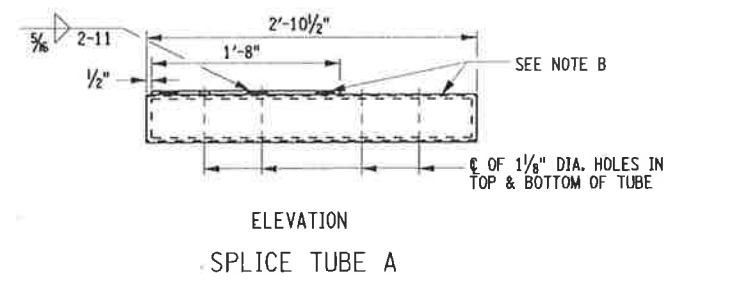
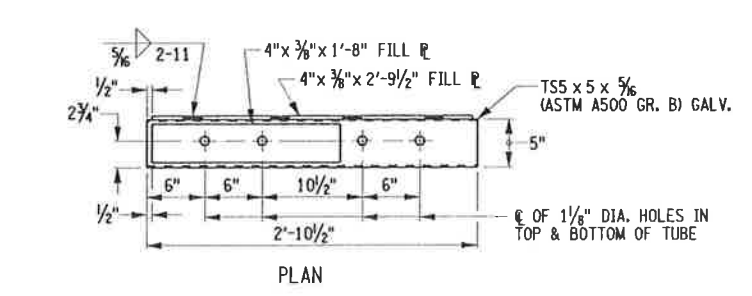
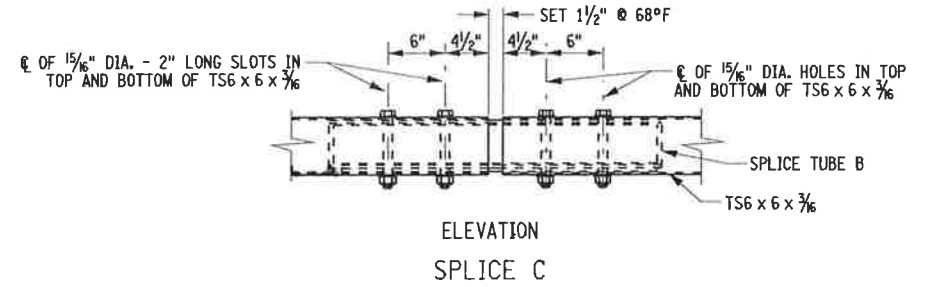
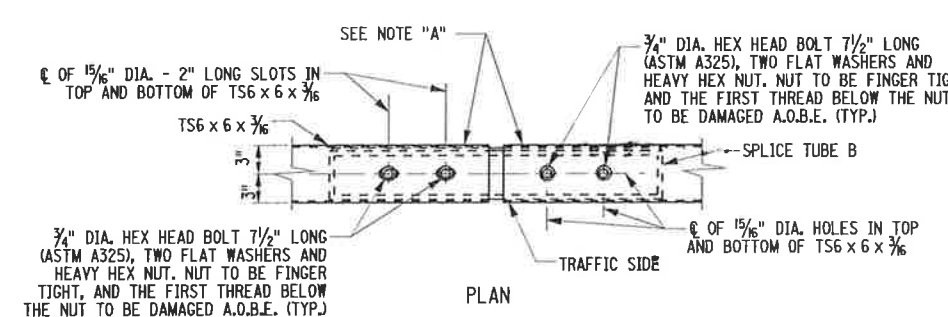
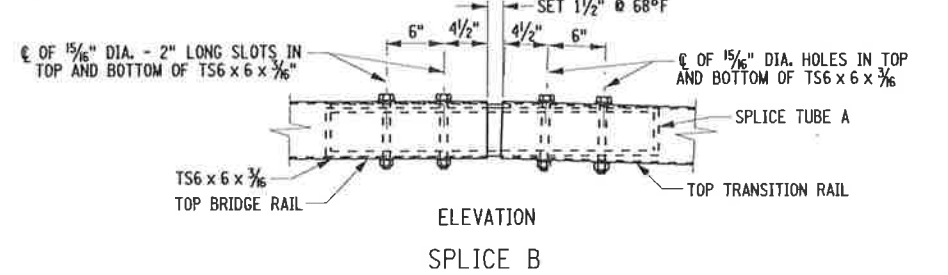
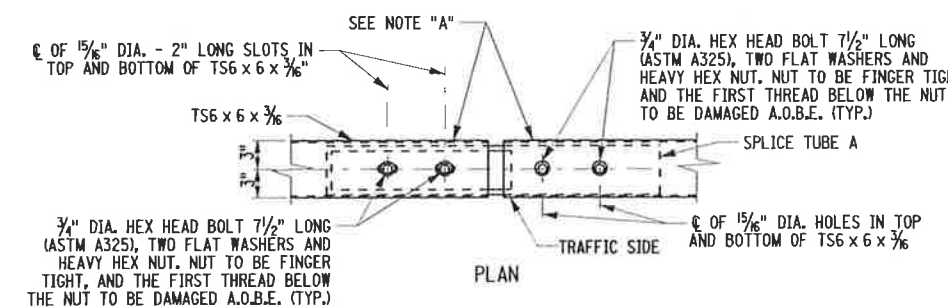
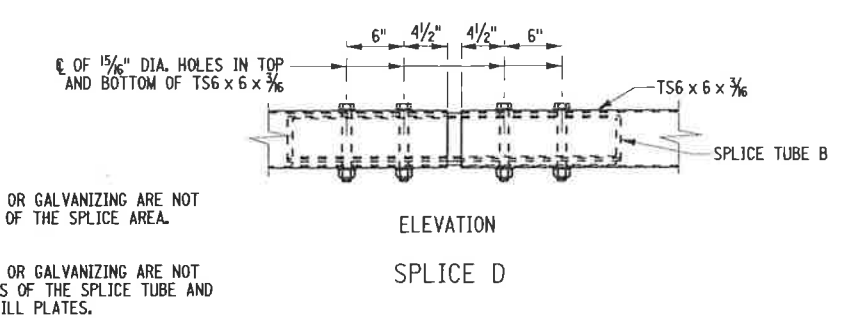
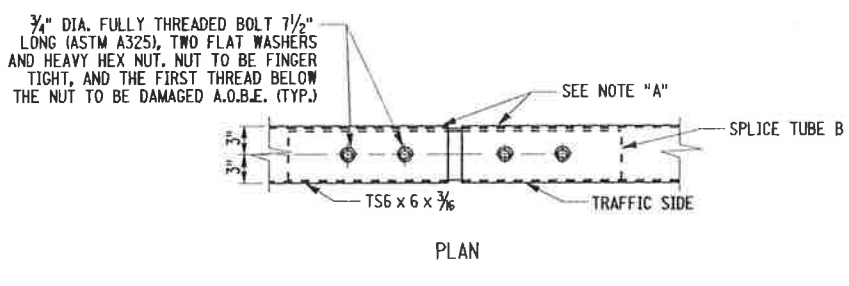
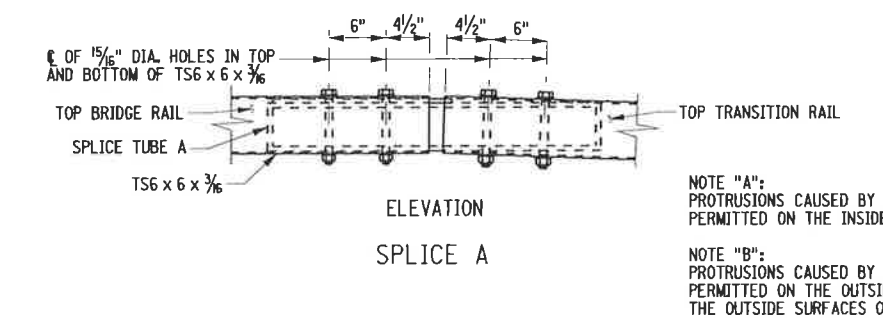
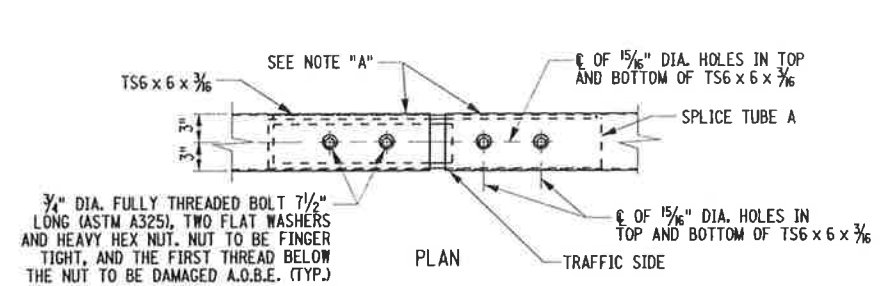
NOTE:
FOR INSTALLATION ON PRESTRESSED UNIT SUPERSTRUCTURE SEE DRAWING 3 OF 3.



NOTE TO THE DESIGNER:
THE RAIL SYSTEM SHOWN IN THESE DETAILS IS INTENDED FOR USE ONLY ON SHORT, LOW LEVEL BRIDGES THAT CARRY LOW VOLUME (LESS THAN 1500 AADT), LOCAL RURAL HIGHWAYS WHERE A SMALL NUMBER OF HEAVY VEHICLES ARE EXPECTED AND SPEEDS ARE EITHER POSTED OR REDUCED. FOR SELECTION GUIDANCE, REFER TO THE PERFORMANCE LEVEL ONE (PL-1) CRITERIA AS DEFINED IN THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS OR THE 1989 AASHTO GUIDE SPECIFICATIONS FOR BRIDGE RAILING. THIS RAILING SYSTEM IS NOT RECOMMENDED FOR USE ON BRIDGES WHOSE STRUCTURE LENGTH BETWEEN ABUTMENTS EXCEEDS 100 FEET.

NOTES:
FOR "SPLICE A" THRU "SPLICE D," SEE BD-RL4E.
FOR "SPLICE E," SEE BD-RL5E.
FOR ADDITIONAL NOTES, SEE BD-RL5E.
DETAILS ON THE DRAWINGS LABELED AS "NOT TO SCALE" ARE INTENTIONALLY DRAWN NOT TO SCALE FOR VISUAL CLARITY. ALL OTHER DETAILS, FOR WHICH NO SCALE IS SHOWN, ARE DRAWN PROPORTIONAL AND ARE FULLY DIMENSIONED.

ISSUED 5/01/08	STATE OF NEW YORK DEPARTMENT OF TRANSPORTATION OFFICE OF STRUCTURES
REVISED	
DOUBLE BOX BEAM BRIDGE RAIL CURBLESS FOR LOW VOLUME (NON-NHS) BRIDGE (1 OF 3)	
APPROVED: 1/18/08 ORIGINAL SIGNED BY GEORGE A. CHRISTIAN, P.E. DEPUTY CHIEF ENGINEER (STRUCTURES)	ISSUED UNDER EB 08-002 EFFECTIVE WITH THE LETTING OF 1/08/09



NOTES:

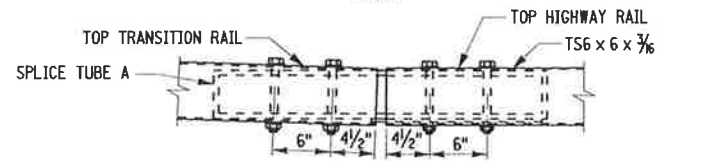
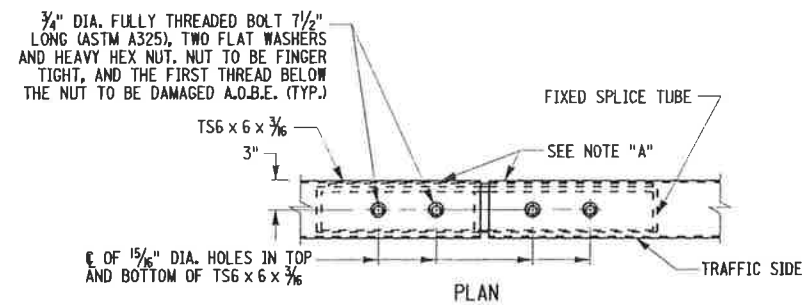
FOR ADDITIONAL NOTES, SEE BD-RL5E.

FOR LOCATION OF "SPLICE A" THRU "SPLICE D", SEE BD-RL3E.

FOR "SPLICE E" AND "SPLICE TUBE B", SEE BD-RL5E.

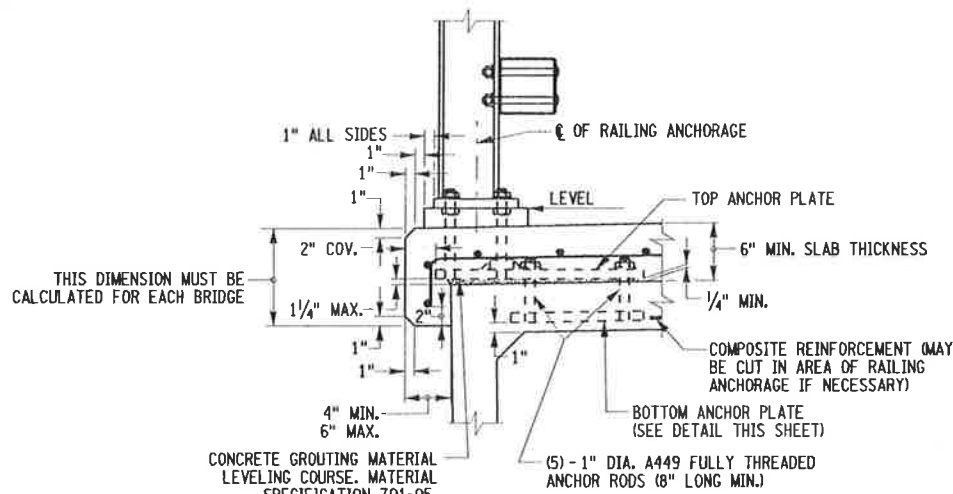
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ISSUED 5/01/08		STATE OF NEW YORK DEPARTMENT OF TRANSPORTATION OFFICE OF STRUCTURES
REVISIONS		<p>DOUBLE BOX BRIDGE RAIL CURBLESS FOR LOW VOLUME (NON-NHS) BRIDGES (2 OF 3)</p> <p>APPROVED: 1/18/08 ORIGINAL SIGNED BY GEORGE A. CHRISTIAN, P.E. DEPUTY CHIEF ENGINEER (STRUCTURES)</p> <p>ISSUED UNDER EB 08-002 EFFECTIVE WITH THE LETTING OF 1/08/09</p>

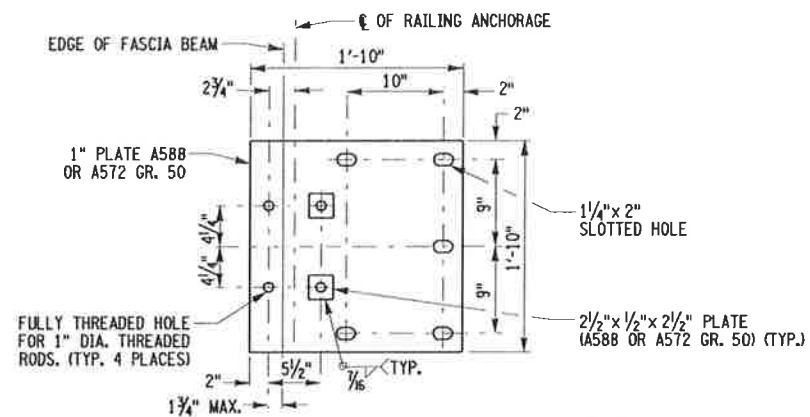


ELEVATION

SPLICE E

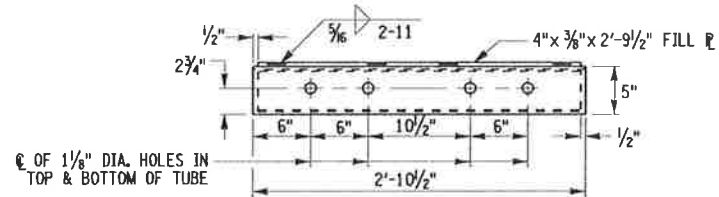


SECTION A-A
PRESTRESSED UNIT SUPERSTRUCTURE
(BOX BEAM SHOWN, SLAB UNIT SIMILAR)

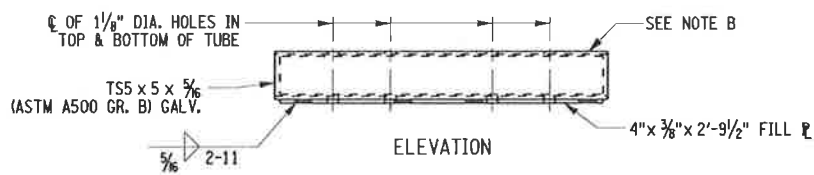


TOP ANCHOR PLATE

NOTES:
THE COST OF THE TOP ANCHOR PLATE AND THE FOUR (4) 1" DIA. RAILING ANCHOR RODS SHALL BE INCLUDED IN THE PRICE BID FOR THE BRIDGE RAILING.
TOP AND BOTTOM ANCHOR PLATES TO BE GALVANIZED IN ACCORDANCE WITH MATERIAL SPECIFICATION 719-01.



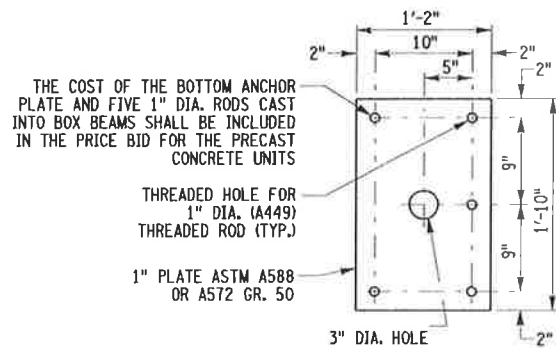
PLAN



ELEVATION

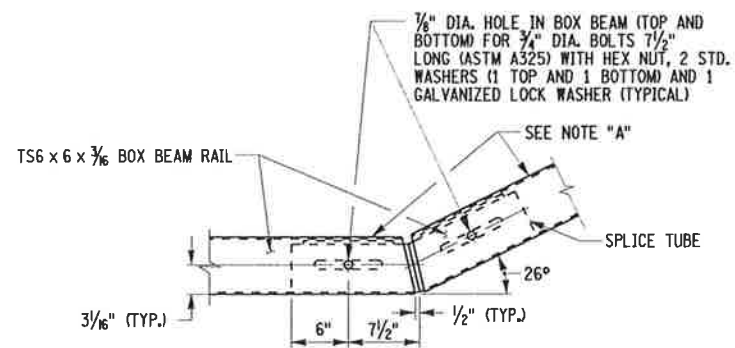
SPLICE TUBE B

NOTE "A":
PROTRUSIONS CAUSED BY WELDING OR GALVANIZING ARE NOT PERMITTED ON THE INSIDE WALLS OF THE SPLICE AREA.
NOTE "B":
PROTRUSIONS CAUSED BY WELDING OR GALVANIZING ARE NOT PERMITTED ON THE OUTSIDE WALLS OF THE SPLICE TUBE AND THE OUTSIDE SURFACES OF THE FILL PLATES.

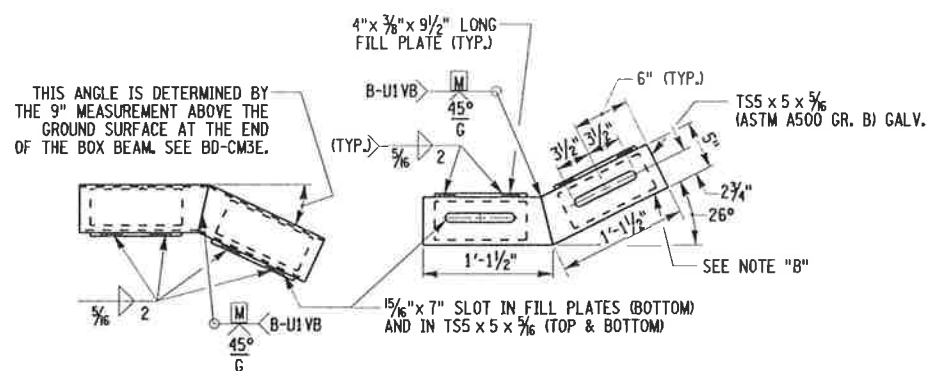


BOTTOM ANCHOR PLATE

NOTES:
RAIL TUBES SHALL BE IN CONFORMANCE WITH MATERIAL SPECIFICATION 710-23.
ALL RAILING IS TO BE FABRICATED AND ERECTED SO THAT THE RAILS ARE PARALLEL TO EACH OTHER AND THE RAILING POSTS ARE TRULY VERTICAL.
THE BASE PLATES SHALL BE PERPENDICULAR TO THE POST UNLESS OTHERWISE NOTED.
TUBULAR STEEL RAILS, RAIL TUBE SPLICE ASSEMBLIES, BOLTS, NUTS, LOCK WASHERS, AND PLAIN FLAT WASHERS SHALL BE PAID FOR UNDER THE RAILING ITEM.
ANCHOR RODS, NUTS, WASHERS, AND ANCHOR PLATES SHALL BE PAID FOR UNDER THE RAILING ITEM.
AFTER THE ANCHOR ROD NUTS HAVE BEEN PLACED AND TIGHTENED TO THE SATISFACTION OF THE ENGINEER, THE RODS SHALL BE CUT OFF BY FLAME CUTTING 1" ABOVE THE NUT, AND THE FIRST THREE THREADS ABOVE THE NUT SHALL BE DAMAGED AS ORDERED BY THE ENGINEER.
BRIDGE RAILS SHALL SPAN A MINIMUM OF THREE POSTS.
UNLESS COVERED BY OTHER SPECIFICATIONS, ALL DIMENSIONS RELATED TO THE FABRICATION OF THE STEEL RAILING SHALL HAVE A TOLERANCE OF 1/16 INCH.
PRIOR TO GALVANIZING THE ASSEMBLED POST, GRIND ALL EDGES OF ALL PLATES TO A MINIMUM RADIUS OF 1/16 INCH.
EXCEPT AS NOTED, ALL BOLTS TO BE TORQUED SNUG TIGHT (APPROXIMATELY 100 FT.-LBS.).
ALL STEEL SHALL BE GALVANIZED AFTER WELDING IN ACCORDANCE WITH THE N.Y.S. STANDARD SPECIFICATION SUBSECTION 719-01.



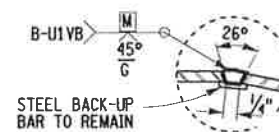
TURN BACK SPLICE



ELEVATION

PLAN

TURN BACK SPLICE TUBE

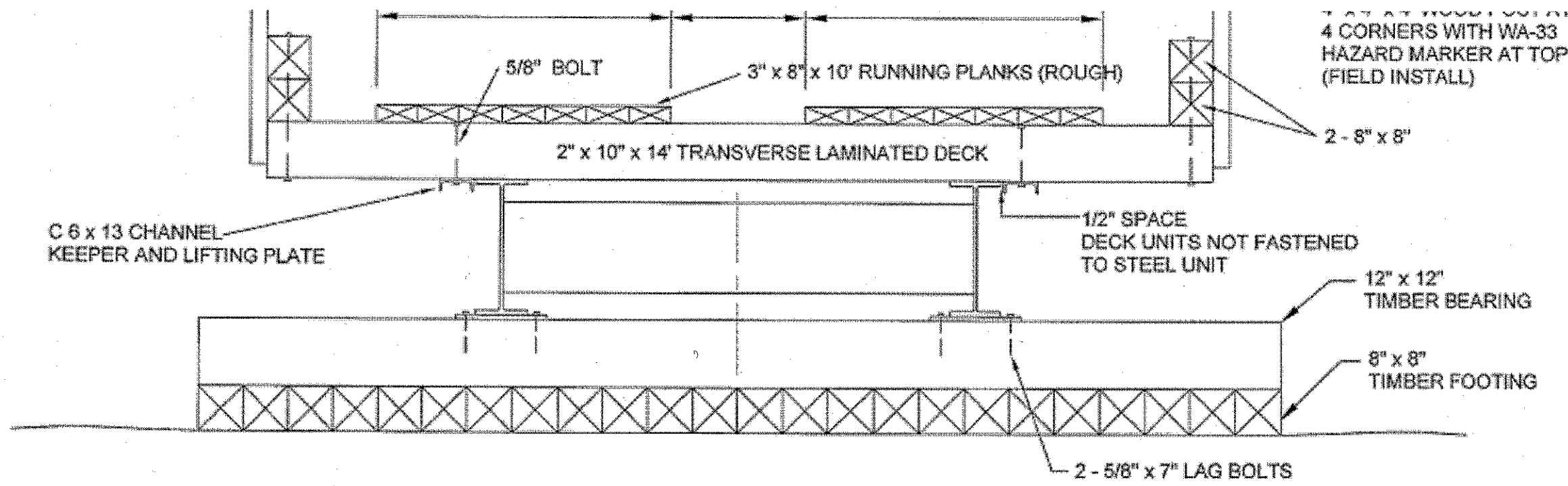


WELD FOR SPLICE TUBE
NOT TO SCALE

NOTES:
FOR LOCATION OF "SPLICE E", SEE BD-RL3E.
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ISSUED 5/01/08	STATE OF NEW YORK DEPARTMENT OF TRANSPORTATION OFFICE OF STRUCTURES
REVISED	
DOUBLE BOX BRIDGE RAIL CURBLESS FOR LOW VOLUME (NON-NHS) BRIDGES (3 OF 3)	
APPROVED: 1/18/08 ORIGINAL SIGNED BY GEORGE A. CHRISTIAN, P.E. DEPUTY CHIEF ENGINEER (STRUCTURES)	ISSUED UNDER EB 08-002 EFFECTIVE WITH THE LETTING OF 1/08/09

G Appendix G - Ontario Ministry of Natural Resources Standard Bridge Barriers

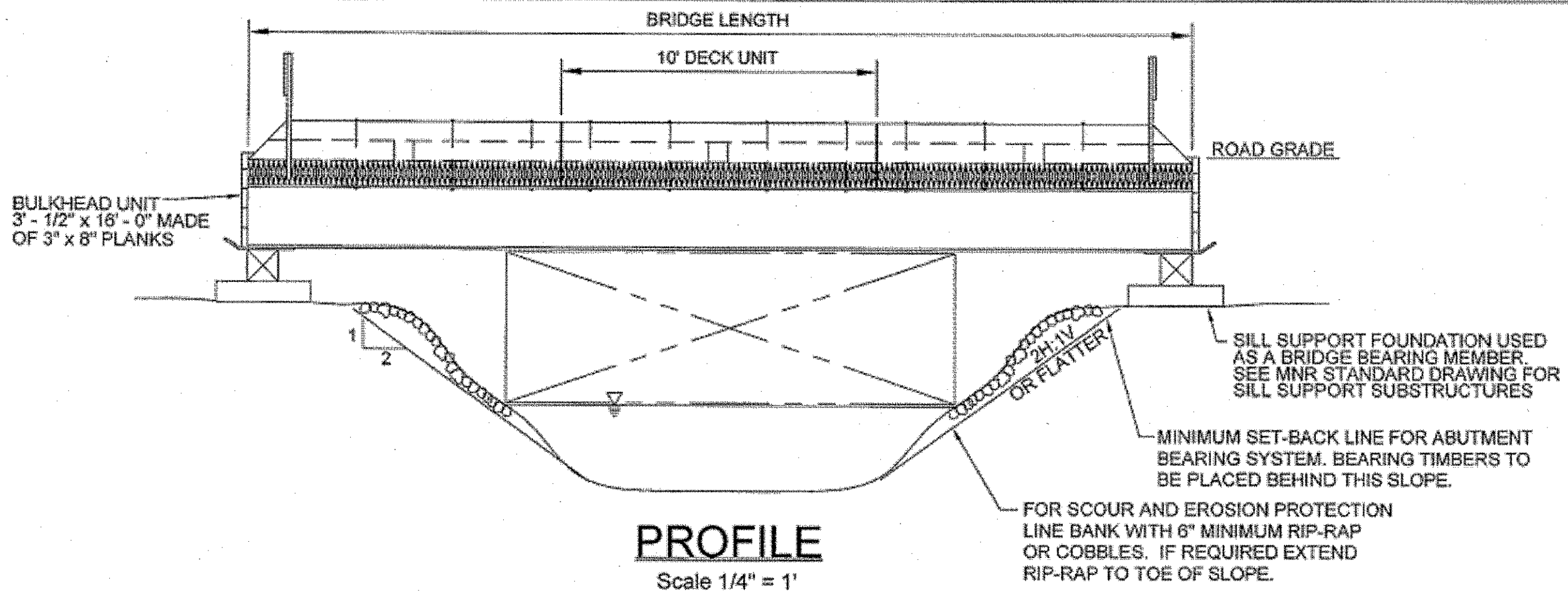


CROSS SECTION
Scale 1/2" = 1'

4 CORNERS WITH WA-33 HAZARD MARKER AT TOP (FIELD INSTALL)

- CLAY SOILS**
1. FOR VERY SOFT, SOFT AND FIRM CLAYS (CAN BE PENETRATED AN INCH BY THE THUMB WITH MODERATE EFFORT: CONSULT A PROFESSIONAL ENGINEER FOR DESIGN.
 2. FOR STIFF CLAY (READILY INDENTED BY THE THUMB BUT PENETRATED ONLY WITH GREAT EFFORT) - 40 FT. MAX. BRIDGE SPAN
 3. FOR VERY STIFF AND HARD CLAYS (THOSE INDENTED BY THUMB NAIL) - 40 FT. MAX. BRIDGE SPAN.

- GRANULAR SOILS**
1. FOR LOOSE AND COMPACT SANDS (EASILY PENETRATED WITH A 3/8"Ø REINFORCING ROD DRIVEN WITH A 5-LB HAMMER: CONSULT A PROFESSIONAL ENGINEER FOR DESIGN.
 2. FOR DENSE SAND, COMPACT AND DENSE WELL GRADED - SAND MIXTURES, GLACIAL TILL, ROCK FILL AND BEDROCK - 40 FT. MAX. BRIDGE SPAN.



PROFILE
Scale 1/4" = 1'

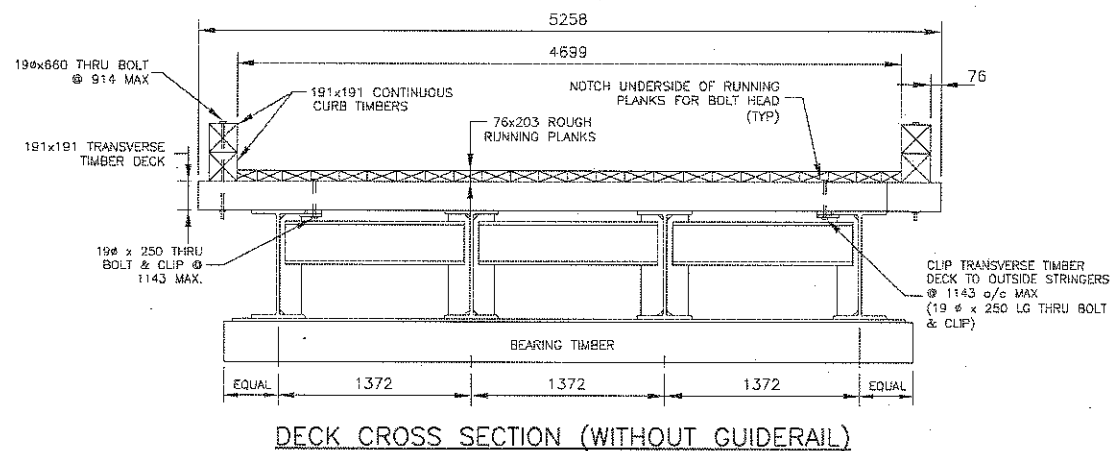
DATE	REV No.	REVISION	BY	APPD

Ministry of Natural Resources
Ontario

PROJECT
PORTABLE 2 STRINGER BRIDGE (IMPERIAL)

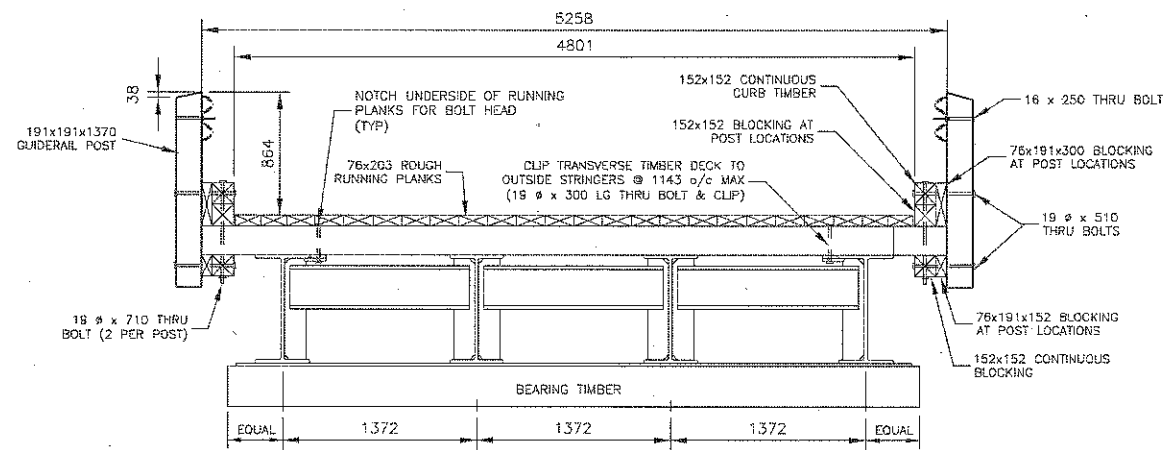
DRAWING TITLE
CROSS SECTION & PROFILE

SCALE	AS NOTED	DATE	February 8, 2008
		DRAWING	REV
		2 of 5	0



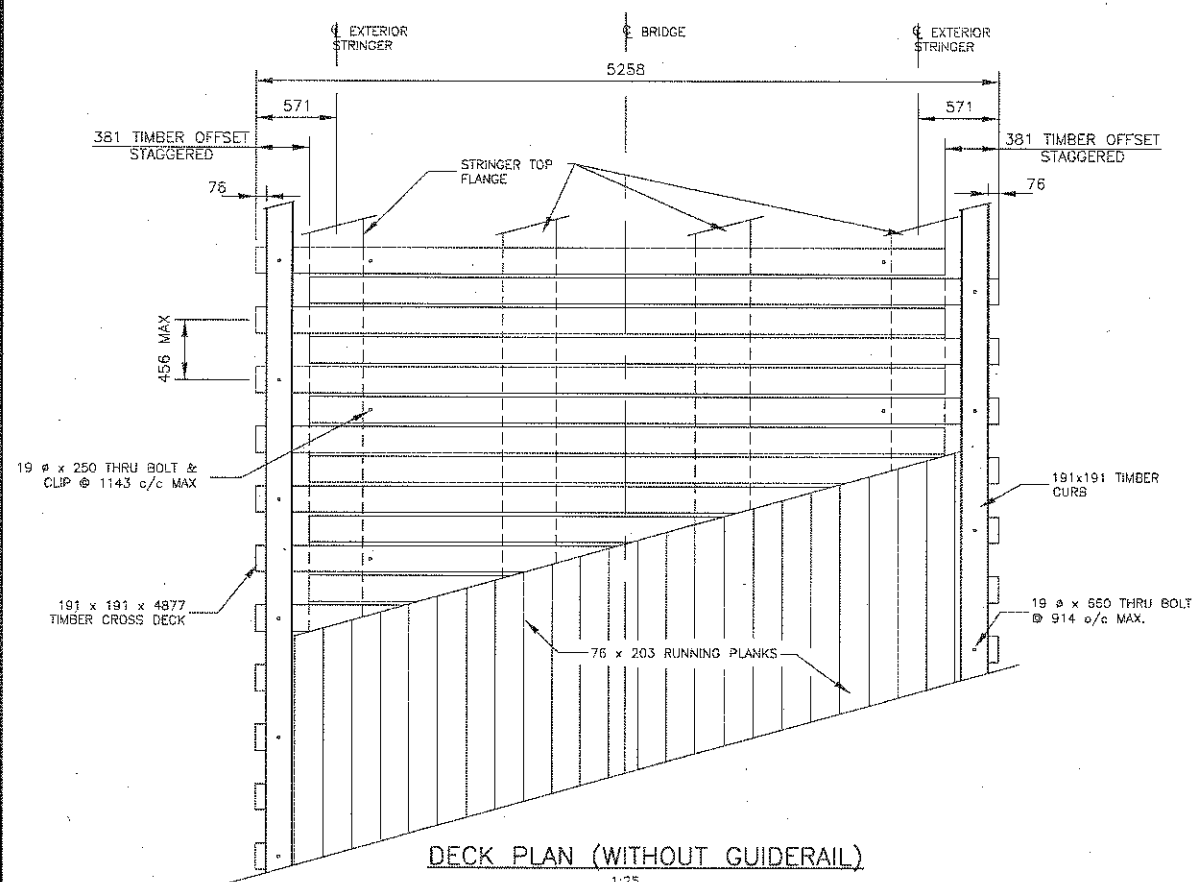
DECK CROSS SECTION (WITHOUT GUIDERAIL)

1:25



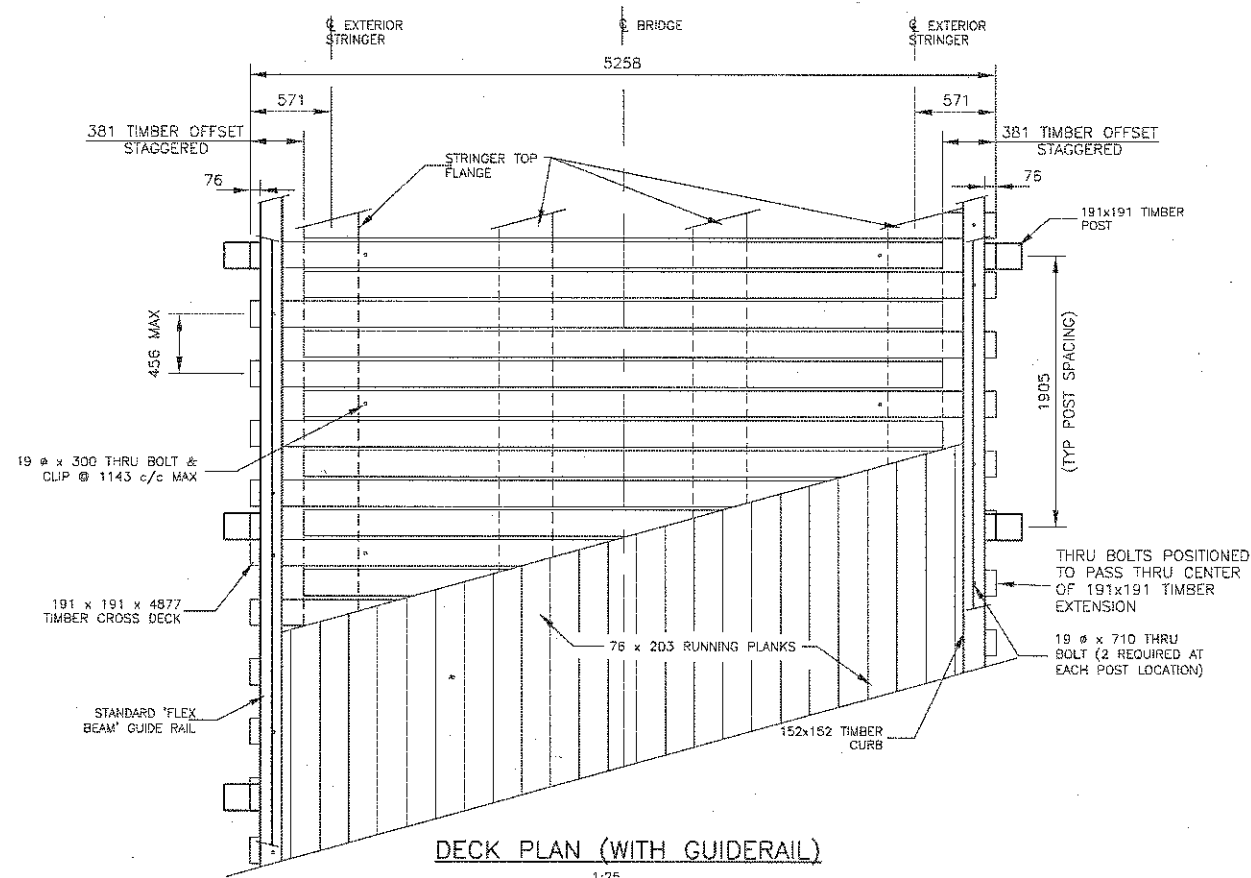
DECK CROSS SECTION (WITH GUIDERAIL)

1:25



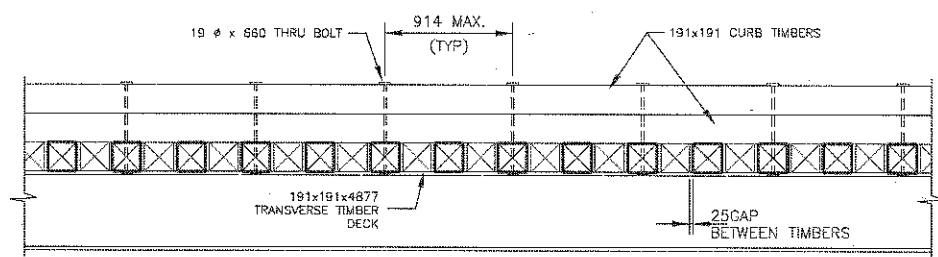
DECK PLAN (WITHOUT GUIDERAIL)

1:25



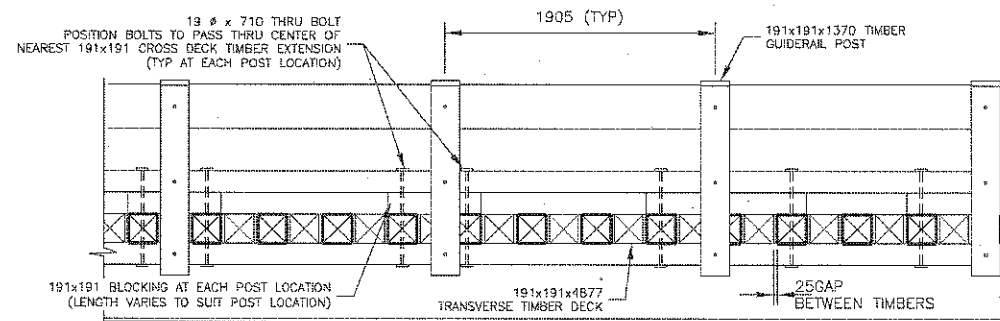
DECK PLAN (WITH GUIDERAIL)

1:25



TRANSVERSE TIMBER DECK ELEVATION (WITHOUT GUIDERAIL)

NOTE: DECK CROSSERS - S.P.F. No. 1 GRADE OR BETTER



TRANSVERSE TIMBER DECK ELEVATION (WITH GUIDERAIL)

NOTE: DECK CROSSERS - S.P.F. No. 1 GRADE OR BETTER

- NOTES:**
- GUIDE RAILS ARE TO BE CONSIDERED ON LONG OR HIGH BRIDGES
 - BRIDGES ARE CONSIDERED LONG IF GREATER THAN 15m IN LENGTH.
 - BRIDGES ARE CONSIDERED HIGH IF THE FALL DISTANCE IS GREATER THAN 6m OR WHERE A SIGNIFICANT HAZARD EXISTS BELOW THE STRUCTURE.
 - 76x203 ROUGH RUNNING PLANKS SHALL BE FASTENED WITH TWO 150mm SPIKES AT EACH END AND ONE 150mm SPIKE @ 457 ON ALTERNATING SIDES.

Date	Rev	Revision	By	Appd

Ministry of Natural Resources



Project
Standard Drawings
Bridges on Crown Land

Drawing Title

Deck Details

Scale	Date	Drawing	Rev
As Noted	Feb. 11, 2004	5 of 6	0