

MEDICINE RIVER BRIDGE REHABILITATION

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ABSTRACT

The Medicine River Bridge is located in central Alberta on Highway 11, midway between Red Deer and Rocky Mountain House. The bridge is a three span cast-in-place concrete girder bridge constructed in 1954. It has a clear roadway of 8.5 metres. With traffic volumes in the order of 5000 AADT the bridge is an important link for local commuters to Red Deer and for oilfield traffic to west-central Alberta.

Two deficiencies were identified regarding the functionality of the bridge. First, the bridge's load capacity was in question due to shear cracking of the girders and second, the narrow bridge clear roadway and out-of-date bridgerails were identified as being a safety concern. An assessment of the bridge and development of rehabilitation alternatives showed that bridge rehabilitation was more economical than bridge widening or bridge replacement.

A load evaluation of the bridge was carried out in accordance with the Canadian Highway Bridge Design Code (CHBDC). This load evaluation showed that the girders were deficient in negative moment capacity but not in shear capacity. The girders were strengthened by attaching carbon fibre reinforced plates to the bottom of the bridge deck.

The existing bridgerails were upgraded by mounting a new steel HSS rail in front of the existing bridgerail posts and in line with the existing curb roadway face. To protect traffic from the ends of the bridgerail the approach rail guardrail was securely fastened to the ends of the new HSS rails and locally stiffened to meet crash test requirements.

Rehabilitation of the bridge was carried out in the fall of 2002.

This paper describes the deficiencies identified in the Medicine River Bridge, the assessments carried out to select an appropriate rehabilitation strategy and the design and construction of the selected bridge rehabilitation strategy.

1.0 INTRODUCTION

The Medicine River Bridge is located in Central Alberta on Highway 11, midway between Red Deer and Rocky Mountain House. With traffic volumes in the order of 5000 AADT it is an important link for local commuters to Red Deer and for oilfield traffic traveling to west-central Alberta. It was constructed in 1954 and has spans of 19.5 – 24.4 – 19.5 metres (see Figure 1).

The bridge superstructure consists of four lines of reinforced concrete haunched "I" girders supporting a concrete deck (see Figure 2). The existing bridgerail consists of concrete posts and steel railings. The clear roadway on the bridge is 8.5 metres which is significantly less than the 11.1 metre paved roadway width on Hwy 11 adjacent to the bridge. The bridge substructure consists of concrete piers and abutments supported on piles.

2.0 BRIDGE ASSESSMENT

In 1999, Alberta Transportation retained UMA Engineering to carry out an assessment of the Medicine River Bridge. The primary issues of concern were the structural condition of the bridge girders and safety concerns created by the narrow clear roadway width on the bridge.

1. Structural Condition of Girders

Shear cracking of the concrete “T” girders had been noted and crack widths measured as far back as 1987. Therefore, a key component of the assessment was to document the locations and widths of the girder shear cracks. The most significant shear cracking noted was in the end spans of the bridge, either adjacent to the abutments or at the span quarter points closest to the piers. Three shear cracks appeared to extend through the webs of the girders and have widths greater than 0.25 mm. These three shear cracks are described as follows:

-shear crack 1.2 metres from abutment in west span of bridge (second girder line from north). Crack width varies from 0.23 mm on south face of girder to 0.28 mm on north face of girder. Crack inclination varies from 40° off vertical on south face of girder to 30° off vertical on north face of girder.

-shear crack 1.5 metres from abutment in west span of bridge (north girder line). Crack width varies from 0.30 mm on south face of girder to hairline on north face of girder. Crack inclination varies from 45° off vertical on south face of girder to 50° off vertical on north face of girder.

-shear crack 2.7 metres from abutment in east span of bridge (second girder line from north). Crack width varies from 0.56 mm on south face of girder to 0.25 mm wide on north face of girder. Crack inclination varies from 30° off vertical on south face of girder to 45° off vertical on north face of girder.

The 0.56 mm wide shear crack was measured as being 0.51 mm wide in 1987 and 0.63 mm wide in 1997. A subsequent girder inspection in 2002 recorded a crack width of 0.56 mm. The crack width as measured by different inspectors can be expected to vary and depends on where the crack is measured and how hard the thickness gauge is forced into the crack. Based on the shear crack widths noted above it was concluded that the 0.56 mm wide shear crack had remained stable between 1987 and 2002.

2. Bridge Clear Roadway

The 8.5 metre clear roadway on the Medicine River Bridge is 2.6 metres narrower than the 11.1 metre paved roadway width of Hwy 11 as it approaches the bridge. This narrow clear roadway width increases the probability of a vehicle colliding with the bridgerail or of shying away from the bridgerail and colliding with an oncoming vehicle.

An accident analysis was carried out for traffic accidents occurring within 500 metres of the bridge for the years 1991 to 1998. Of the eleven accidents recorded three originated on the bridge deck. The analysis determined that at least one, and possibly two, of the collisions could be partially attributed to the reduced clear roadway width on the bridge. Therefore, it was assumed that the narrow bridge clear roadway width caused one additional traffic accident to occur in the years between 1991 and 1998. This corresponds to an increased collision rate, due to the narrow bridge clear roadway width of approximately 0.00038 collisions per day.

The Turner model, as presented in the Geometric Design Guide for Canadian Roads [3], was used to predict the increase in collision rate caused by different bridge clear roadway widths. This model predicted that increasing the bridge clear roadway width from 8.5 metres to 11.6 metres or more

(minimum 2.1 metre wide shoulders) would decrease the increased collision rate due to bridge clear roadway width to 0.00010 collisions/day (75% reduction). Therefore, widening of the bridge clear roadway width to 13.4 metres would give an annual collision cost savings of \$3600 based on an average collision cost of \$34,800. However, it should be noted that the average collision cost of \$34,800 was based on the bridgerails, including approach rails and approach rail transitions meeting current bridgerail safety requirements.

A life cycle cost-benefit analysis was carried out to determine whether it was more economical to widen the bridge now (Alternative 1) or to strengthen the bridge girders and upgrade the bridgerail now and widen the bridge in 10 years when it was anticipated that Highway 11 would be widened to 13.4 metres (Alternative 2). The cost of widening the Medicine River Bridge clear roadway from 8.5 metres to 13.4 metres was estimated to be \$1,500,000 while the cost of strengthening the bridge girders and upgrading the bridgerail was estimated to be \$235,000. The results of the life cycle cost-benefit analysis are shown in Table 2. This analysis showed that the most economical alternative was to widen the bridge in the future when Highway 11 was widened (Alternative 2).

Table 2 – Life Cycle Cost-Benefit Analysis Results

Item	Alternative 1	Alternative 2
Present Value of Increased Collision Costs Over Next 10 Years	\$0	\$29,000
Present Value of Bridge Girder Strengthening and Bridgerail Upgrading	\$0	\$235,000
Present Value of Bridge Widening	\$1,500,000	\$1,013,000
TOTAL Present Value Cost	\$1,500,000	\$1,277,000

Note: Life cycle cost-benefit analysis based on discount rate of 4%.

3.0 DETERMINATION OF REHABILITATION REQUIREMENTS

In 2002, Alberta Transportation decided to proceed with strengthening of the bridge girders and upgrading of the bridgerail of the Medicine River Bridge. UMA Engineering was retained to carry out a load capacity evaluation of the girders in accordance with the Canadian Highway Bridge Design Code (CHBDC) [1] and to provide engineering services for the strengthening of the bridge girders and upgrading of the bridgerails.

1. Load Capacity of Girders

The load evaluation of the concrete “T” girders was carried out in accordance with Section 14 of the CHBDC. The truck models used in the load evaluation were Alberta Transportation’s CS1, CS2 and CS3 load evaluation trucks as shown in Figure 3.

Based on historical Alberta Transportation practices it was assumed that the concrete design strength was 20.7 MPa and the rebar yield strength was 276 MPa. However, in 1987, 18 concrete cores were taken from the girder diaphragms. The compressive strengths of these cores had a mean value of 45.3 MPa and a standard deviation of 4.5 MPa. It was assumed that the cores were dried seven days in air before testing. This information combined with Section A14.1.2 of the CHBDC gave an equivalent concrete design strength of 36.5 MPa. This equivalent concrete design strength is 76% greater than the original design strength of 20.7 MPa.

The distribution of truck load between girders was determined using the computer program SECAN [2]. This program is based on the semi-continuum method and models the bridge deck as a semi-continuous plate and the girders as line members. The percentages of the truck load carried by the interior and exterior girders in moment and shear are shown in Table 1. Also shown in Table 1 are the percentages of truck load carried by the girders as determined from the CHBDC. The percentages of truck load per girder as determined by SECAN are approximately 15% less than those determined by the CHBDC for the interior girders and 36 to 52% less for the exterior girders.

Table 1 – Percentage of Truck Load Carried by Interior and Exterior Girders

Girder	SECAN		CHBDC	
	Moment	Shear	Moment	Shear
Interior Girders	0.66	0.68	0.76	0.79
Exterior Girders	0.52	0.52	0.71	0.79

The shear capacities of the girders were determined based on the General Method (Modified Compression Field Theory) specified in Section 8.9 of the CHBDC and the girder shear stirrups meeting minimum area requirements. The actual shear stirrup areas provided were never less than 98% of those required by the CHBDC. Girder shear stirrup spacings were up to 10% greater than those allowed by the CHBDC. A resistance adjustment factor (U) of 0.94 was used for determining girder shear capacities except at locations where CHBDC minimum stirrup area requirements or maximum stirrup spacings were not met. At these locations a resistance factor adjustment of 0.82 was used.

The girder load evaluation results for the CS3 truck were as follows:

- a girder load capacity of 64.6 tonnes in shear which was slightly higher than the required load capacity of 63.5 tonnes;
- a girder load capacity of 87.0 tonnes in positive moment which was significantly higher than the required load capacity of 63.5 tonnes; and
- a girder load capacity of 33.6 tonnes in negative moment which was significantly lower than the required load capacity of 63.5 tonnes. The deficient negative moment capacity locations were located at the negative moment rebar cut-off locations and not at the piers.

Based on the above it was concluded that the concrete “I” girders required strengthening in negative moment only. Negative moment strengthening was recommended not only to increase the negative moment capacities of the girders but to also limit the widths of any negative moment cracks occurring at the end span quarter points closest to the piers. It was feared that negative moment cracking at these locations might lead to increased shear cracking and a reduction in the shear capacity of the girders.

2. Bridgerail Upgrading

The existing bridgerails on the Medicine River Bridge consist of concrete posts and upper and lower steel railings (see Figure 4). Vertical steel bars fill the gap between the upper and lower railings. The bridgerails are mounted on safety curbs and terminate in concrete parapets at the abutments. Flex-

beam-guardrails are located on the approaches at each corner of the bridge but are not attached to the bridgerails. Safety concerns identified with the existing bridgerails were:

- inadequate strength to resist the barrier loads specified in the CHBDC;
- concrete posts not being setback from the steel railings resulting in the potential for excessive vehicle snagging on the posts;
- safety curbs that had the potential to vault vehicles over the bridgerails or cause vehicle rollover; and
- approach guardrails that did not adequately protect the ends of the bridgerails against vehicle collisions.

Due to the above bridgerail deficiencies and the 1999 recommendation to delay bridge widening for 10 years it was recommended that the bridgerails be upgraded to meet Performance Level 1 (PL-1) requirements as specified by the CHBDC.

4.0 BRIDGE GIRDER STRENGTHENING

1. Girder Strengthening Alternatives

The bridge load evaluation showed that negative moment strengthening of the girders was required from 0.5L to 0.8L in the end spans and from 0.2L to 0.4L and from 0.6L to 0.8L in the centre span. Two alternatives were considered for negative moment strengthening. The first alternative was to attach steel plates to the bottom of the concrete deck using adhesive anchors (see Figure 5). The second alternative was to attach carbon fibre reinforced plates (CFRP) to the bottom of the deck using epoxy adhesive (see Figure 6).

The construction cost of strengthening with steel plates was estimated to be 7% less than the construction cost of strengthening with CFRP plates. However, the CFRP strengthening alternative was selected for detailed design and construction. This decision was made because it provided an opportunity for Alberta Transportation to tender and construct a project using CFRP technology in a low risk setting, i.e. no increase in bridge loading after strengthening.

2. CFRP Strengthening Design

The CFRP girder strengthening was designed in accordance with the requirements of Section 16 of the CHBDC and ISIS Canada Design Manual No. 4 (Strengthening Reinforced Concrete Structures with Externally-Bonded Fibre Reinforced Polymers) [4]. The CFRP plates specified were Sika Carbodur S1012 CFRP plates or approved equivalent. The design properties of the CFRP plates were taken to be:

- tensile strength = 2800 MPa
- elongation at break = 1.7%
- elastic modulus = 165,000 MPa
- cross-sectional area = 120 mm²

A resistance factor (ϕ) of 0.75 was used for the design of the CFRP plates which is less than the resistance factor (ϕ) of 0.85 allowed by the CHBDC. To control cracking the maximum allowable strain in the CFRP plates was limited to 0.7% at the ultimate limit state.

The extent of the CFRP negative moment strengthening is shown in Figure 7. The ends of the CFRP plates were encased in carbon fibre fabric placed perpendicular to the plates. This was done to improve the anchorage of the CFRP plates in the high negative moment regions adjacent to the piers.

5.0 BRIDGERAIL UPGRADING

To meet PL-1 bridgerail requirements an HSS152x152 bridgerail was placed in front of the existing bridgerail (see Figure 8). The roadway face of the HSS rail was placed in line with the roadway face of the curb to effectively eliminate the safety curb. The top of the HSS rail was placed 665 mm above the roadway surface. This is only 15 mm less than the 680 mm minimum height requirement for PL-1 bridgerails in the CHBDC. The upgraded bridgerail had a 259 mm vertical opening between the bottom of the HSS rail and the top of the curb and had a rail contact width to bridgerail height ratio of 0.61 (see Figure 8). This bridgerail geometry met the geometric requirements of Appendix A13.1 of the 1998 AASHTO LRFD Bridge Design Specifications [5] for reducing vehicle snagging on the bridgerail to an acceptable level. The geometric requirements specified in AASHTO for the bridgerail were a maximum vertical opening width of 300 mm and a minimum rail contact width to bridgerail height of 0.25.

The HSS152x152 bridgerail was designed to distribute vehicle collision loads between the existing concrete posts to the extent necessary to reduce the loads to a level less than the existing post capacities. This design was carried out in accordance with the provisions of Appendix A13.3.2 of the 1998 AASHTO LRFD Bridge Design Specifications. The governing failure mode occurred over three bridgerail spans (see Figure 9).

The approach rail transitions consist of flex-beam guardrails attached to the ends of the HSS bridgerails and channel rubrails attached to the ends of the concrete curbs (see Figure 8). This type of approach rail transition has been successfully crash tested to TL-2 crash test requirements while attached to the end of a curb mounted flexbeam bridgerail. TL-2 crash test requirements are specified in NCHRP Report 350 [6] and are similar to but somewhat less severe than PL-1 crash test requirements.

6.0 CONSTRUCTION

Rehabilitation of the Medicine River Bridge was carried out in September and October, 2002.

1. CFRP Strengthening

The CFRP strengthening work was carried out from a working platform that was suspended from the girders and was capable of containing any deleterious spills occurring on the platform. In-stream work was not allowed at any time including during erection and dismantling of the working platform.

The bonding surfaces between the CFRP plates and the bottom of the concrete deck were prepared by filling in small voids in the deck with epoxy, grinding down concrete ridges left by the deck formwork and roughening the deck surface by sandblasting. Delaminated deck concrete on the bonding surfaces was repaired and working cracks crossing the bonding surfaces were epoxy

injected. Holes were cored and chipped through the intermediate concrete girder diaphragms as required to allow the CFRP plates to pass through.

The construction contract specified that pull-off tests, carried out in accordance with ACI 503R, were to be used to verify that the tensile strength of the deck concrete and the bond of the epoxy to the concrete were a minimum of 1.5 MPa. One pull-off test was required in each bridge span (3 tests total). Only three tests were required because the concrete was assumed to have adequate strength based on the strengths of concrete cores taken from the girder diaphragms in 1987. These concrete cores had an average compressive strength of 45.3 MPa.

The strengths recorded from the first three pull-off tests were 2.01 MPa, 1.88 MPa and 0.79 MPa. It was speculated that the deficient strength result might be due to eccentricities applied to the concrete during coring or testing. Therefore two additional pull-off tests, including one at the location of the failed test, were ordered. The strengths recorded from these additional pull-off tests were 2.14 MPa and 0.85 MPa. Unfortunately neither of the pull-off tests was carried out at the location of the initial failed test. It was then decided to carry out an additional 15 pull-off tests including tests at the locations of the two failed tests. These tests recorded tensile strengths ranging between 3.43 MPa and 1.50 MPa which satisfied project requirements.

The construction contract required that the design properties of the Sika Carbodur S1012 CFRP plates be verified in accordance with the requirements of Section 16.5.2 of the CHBDC. Five tests were carried out on the plates. These tests gave an effective tensile strength of 2899 MPa, a minimum elongation at failure of 1.7% and a minimum modulus of elasticity of 166,000 MPa. These results met or exceeded the material properties that were assumed in the design of the CFRP plates.

Installation of the CFRP plates began on October 1 and was completed on October 12, 2002. Because of the cool weather conditions heating and hording was required to adequately cure the epoxy adhesive bonding the CFRP plates to the concrete deck. Heating and hording was provided from October 1 until October 17, five days after the last of the CFRP plates were installed.

The total construction cost of the CFRP strengthening was approximately \$243,000 (\$400 per metre of CFRP plate placed). This cost included the cost of mobilization, traffic accommodation, working platform erection and dismantling, deck surface preparation, materials testing and CFRP plate supply and installation.

2. Bridgerail Upgrading

The new HSS bridgerails were attached to the existing concrete posts with steel clips and the existing concrete abutment parapets by bolting through the parapets. The ends of the HSS rails were attached to the flex-beam guardrail approach rail transitions with steel connection plates. The existing concrete curbs were extended to the ends of the abutment parapets and made continuous with the channel rubrails of the approach rail transition (see Figure 8). The existing flex-beam guardrail approach rails were realigned to connect with the new approach rail transitions. Traffic on the bridge was reduced to one lane during the work.

The total construction cost of the bridgerail/approach rail transition upgrading was approximately \$91,000 (\$520 per metre of bridgerail/approach rail transition placed). This cost included the cost of

mobilization, traffic accommodation, existing bridgerail post repair, extending of the concrete curbs and supply and installation of the new bridgerails and approach rail transitions.

7.0 REFERENCES

- [1] CSA. 2000. *Canadian Highway Bridge Design Code*. CAN/CSA-S6-00. Canadian Standards Association, Rexdale, Ontario.
- [2] Jaeger, L.G. and Bakht, B. 1989. *Bridge Analysis by Microcomputer*. McGraw-Hill Book Company, New York.
- [3] TAC. 1999. *Geometric Design Guide for Canadian Roads*. Transportation Association of Canada, Ottawa, Ontario.
- [4] ISIS Canada. 2001. *Strengthening Reinforced Concrete Structures with Externally-Bonded Fibre Reinforced Polymers*. Design Manual No. 4. Intelligent Sensing for Innovative Structures, A Canadian Network of Centres for Excellence, Winnipeg, Manitoba.
- [5] AASHTO. 1998. *LRFD Bridge Design Specifications*. American Association of State Highway and Transportation Officials, Washington, D.C.
- [6] NCHRP. 1993. *Recommended Procedures for the Safety Performance Evaluation of Highway Features*. Report No. 350. National Co-operative Highway Research Program, Washington, D.C.

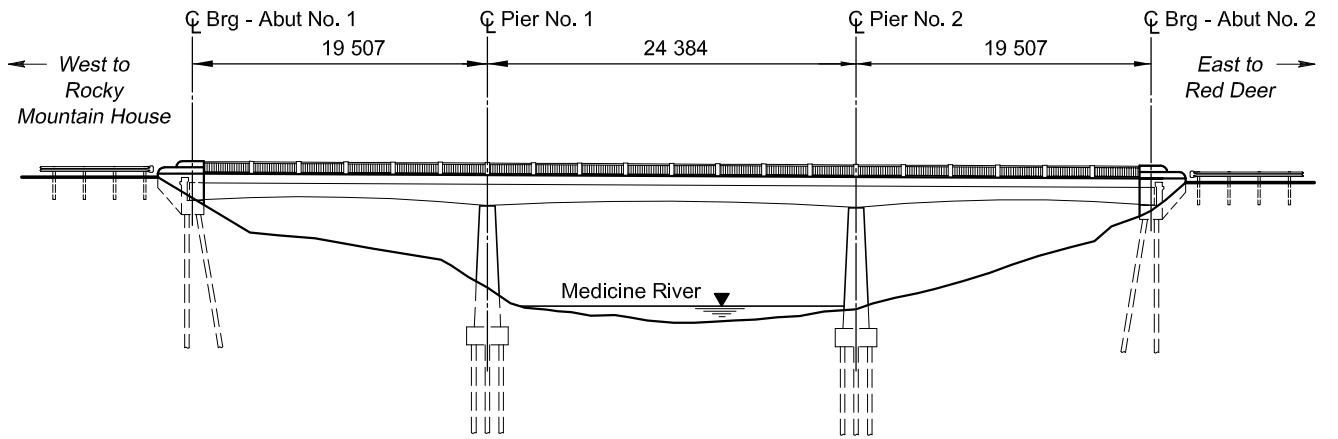


Figure 1 - Bridge Elevation

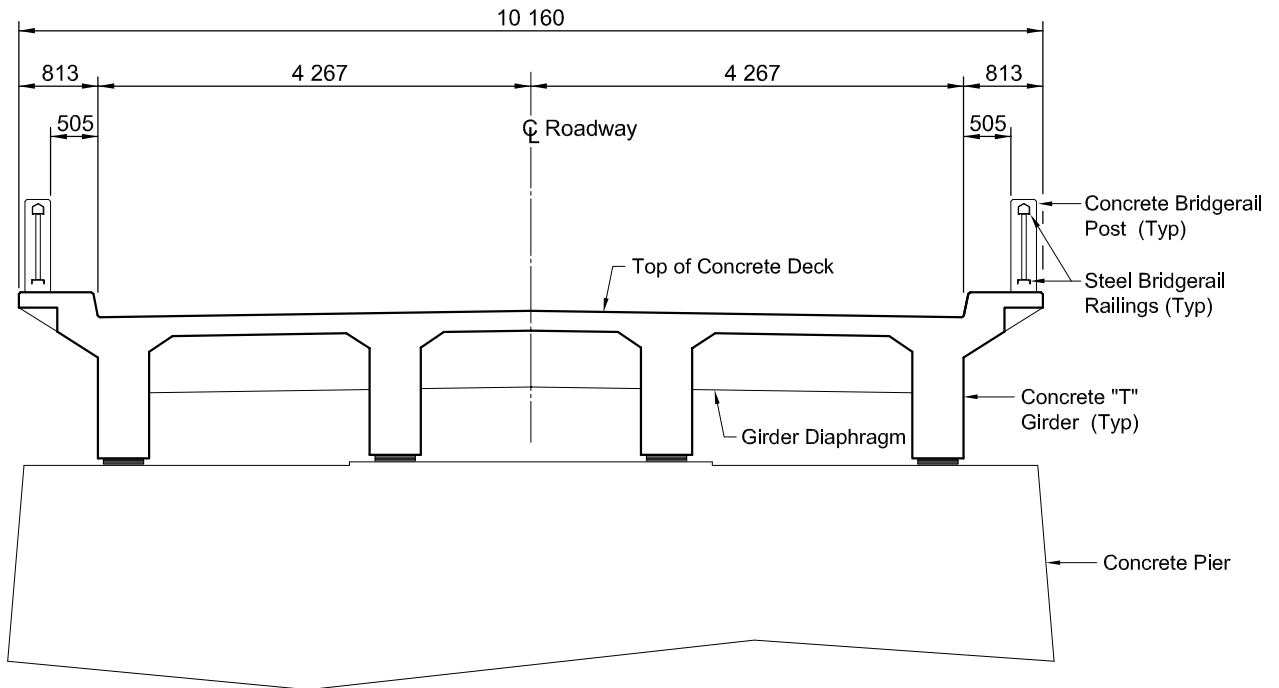
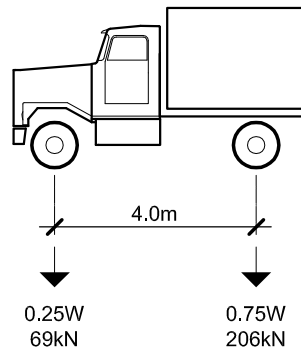
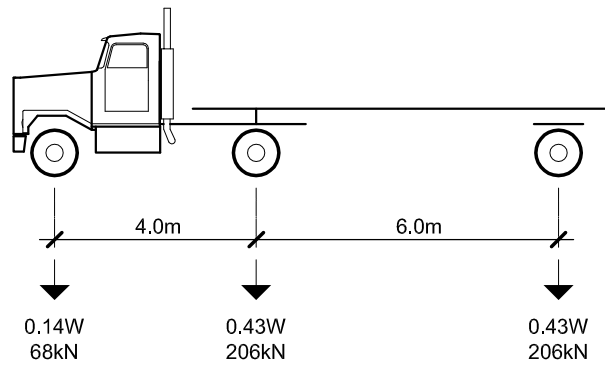


Figure 2 - Bridge Cross - Section

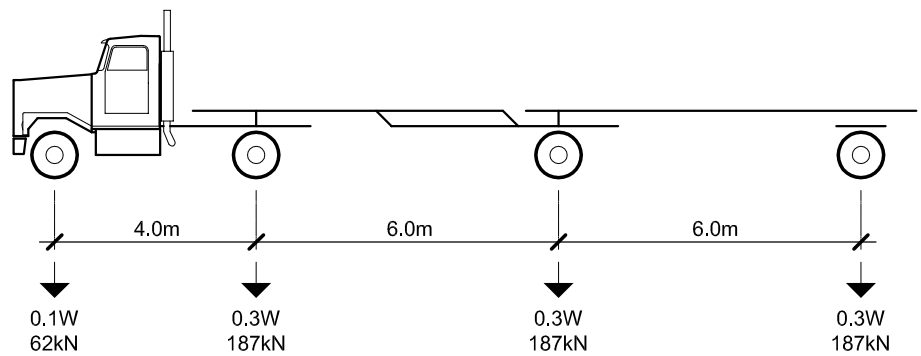
CS1 Truck
 $W = 275\text{kN}$ (28t)



CS2 Truck
 $W = 480\text{kN}$ (49t)



CS3 Truck
 $W = 623\text{kN}$ (63.5t)



$W =$ Gross Vehicle Weight

Figure 3 - Alberta Transportation Load Evaluation Trucks

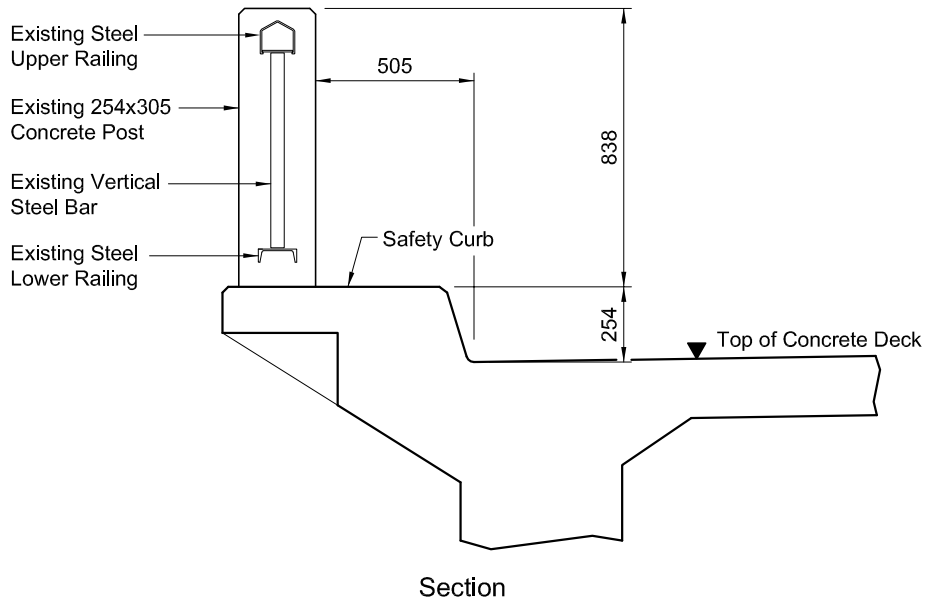
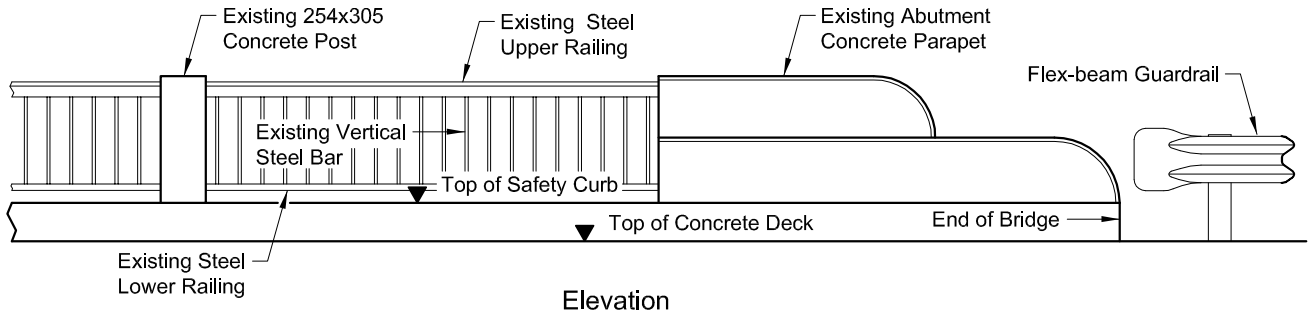


Figure 4 - Existing Bridgerail

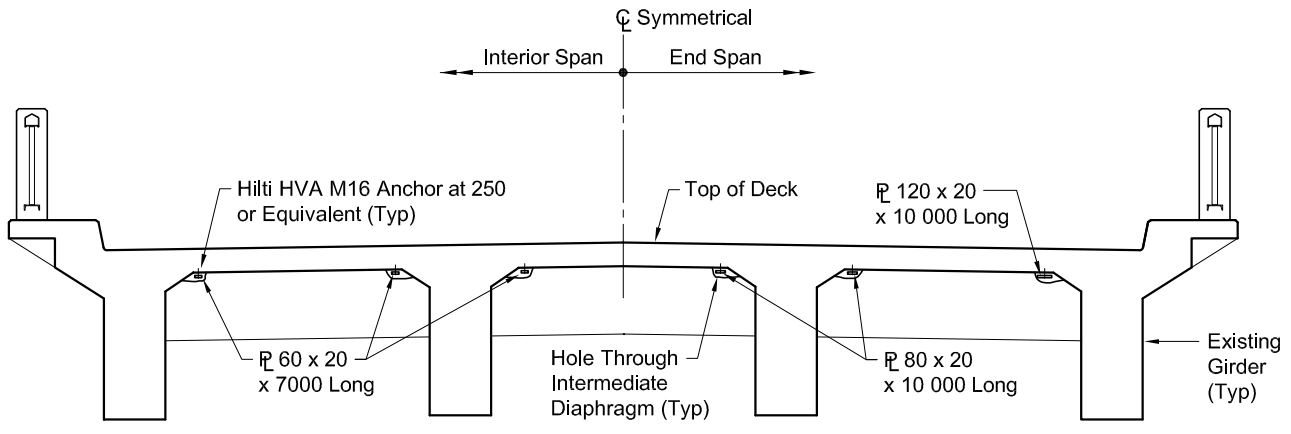


Figure 5 - Girder Strengthening with Steel Plates

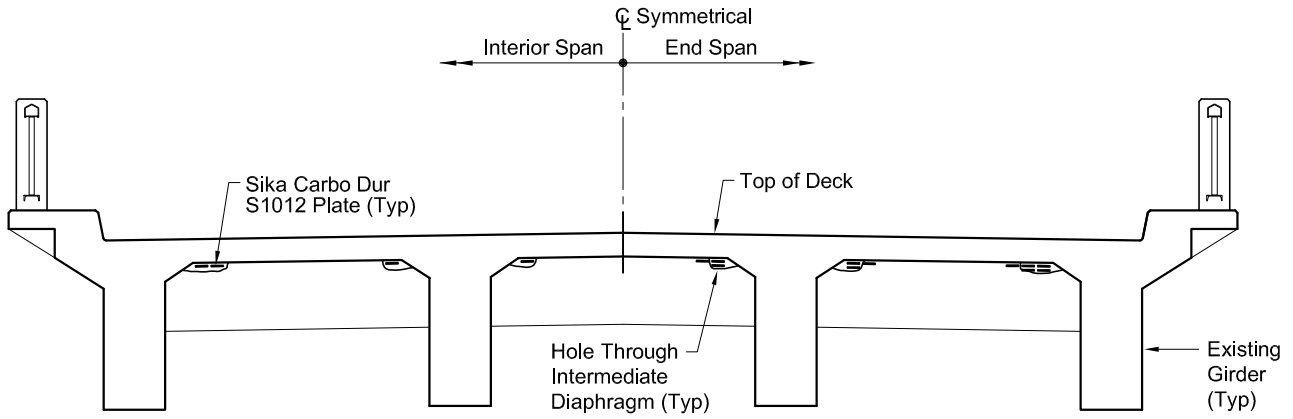


Figure 6 - Girder Strengthening with CFRP Plates

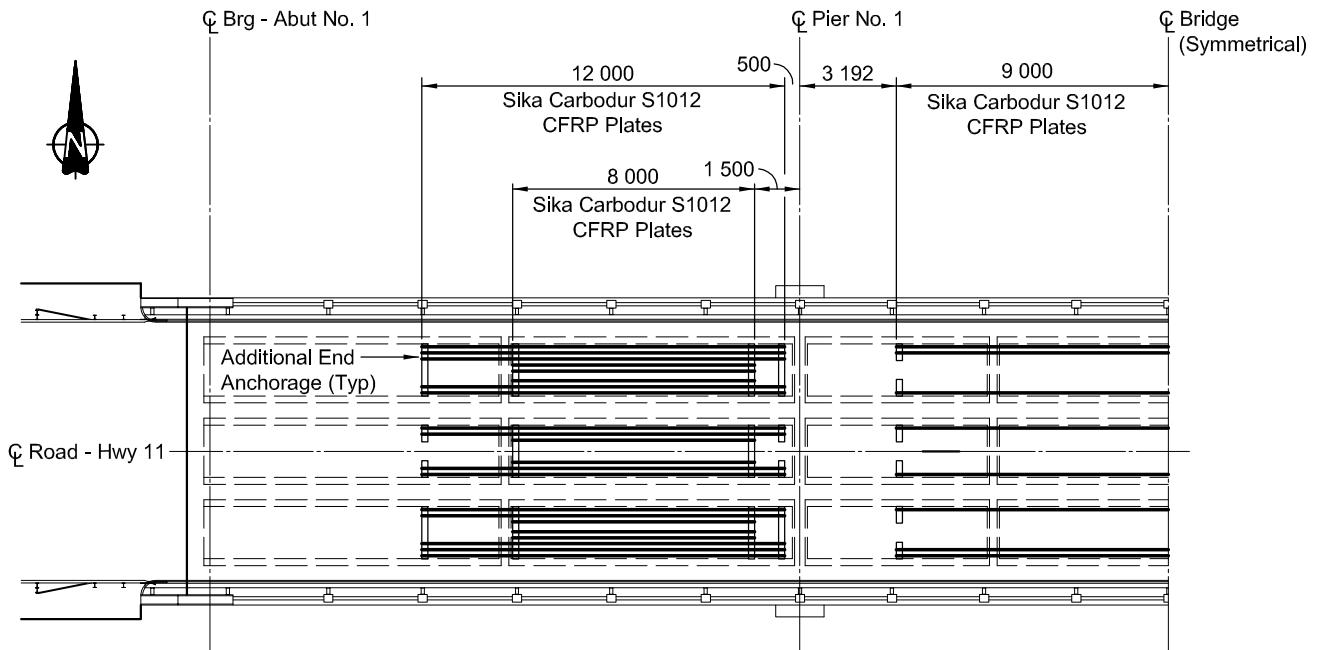
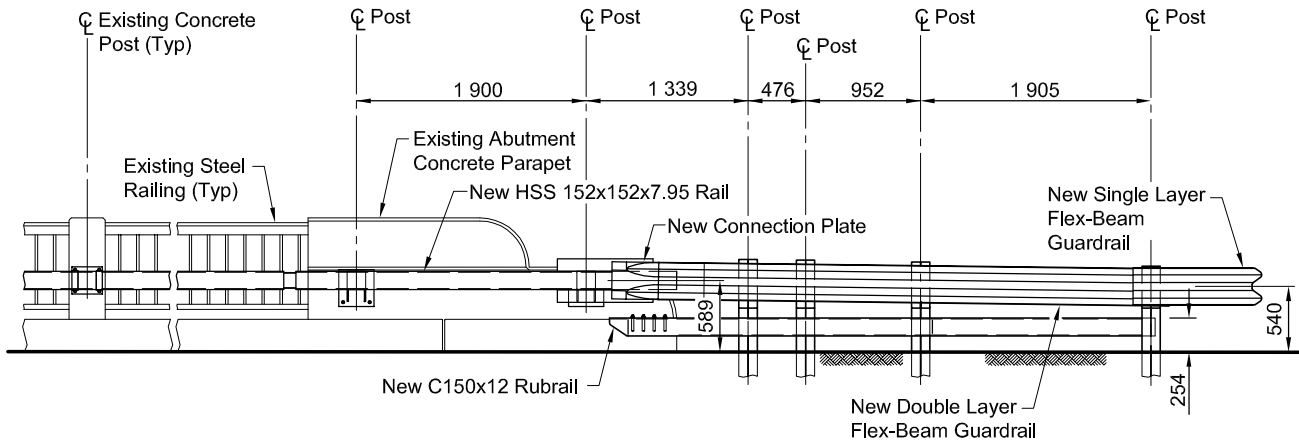
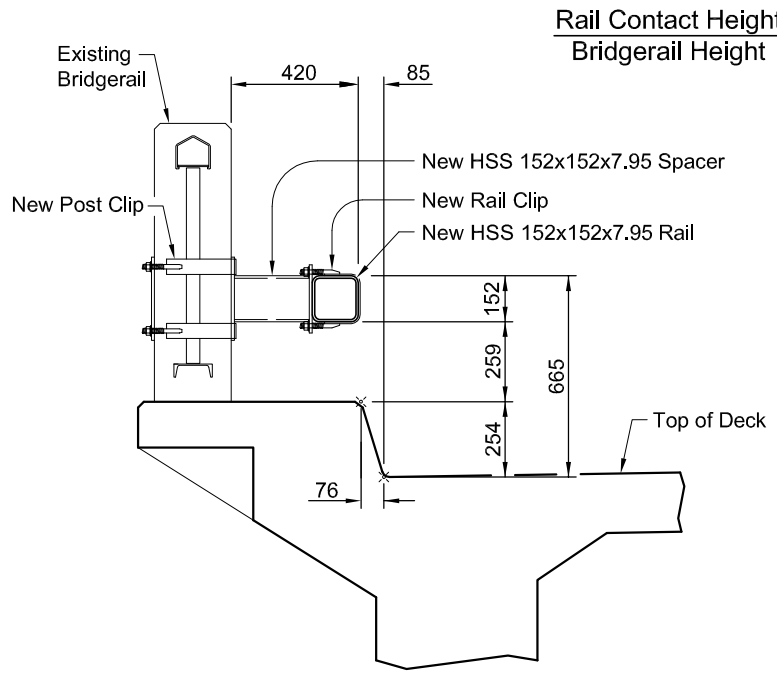


Figure 7 - Plan of CFRP Girder Strengthening



Elevation



$$\frac{\text{Rail Contact Height}}{\text{Bridgerail Height}} = \frac{(152\text{mm} + 259\text{mm})}{665\text{mm}} = 0.61$$

Section

Figure 8 - Upgraded Bridgerail and Approach Rail Transition

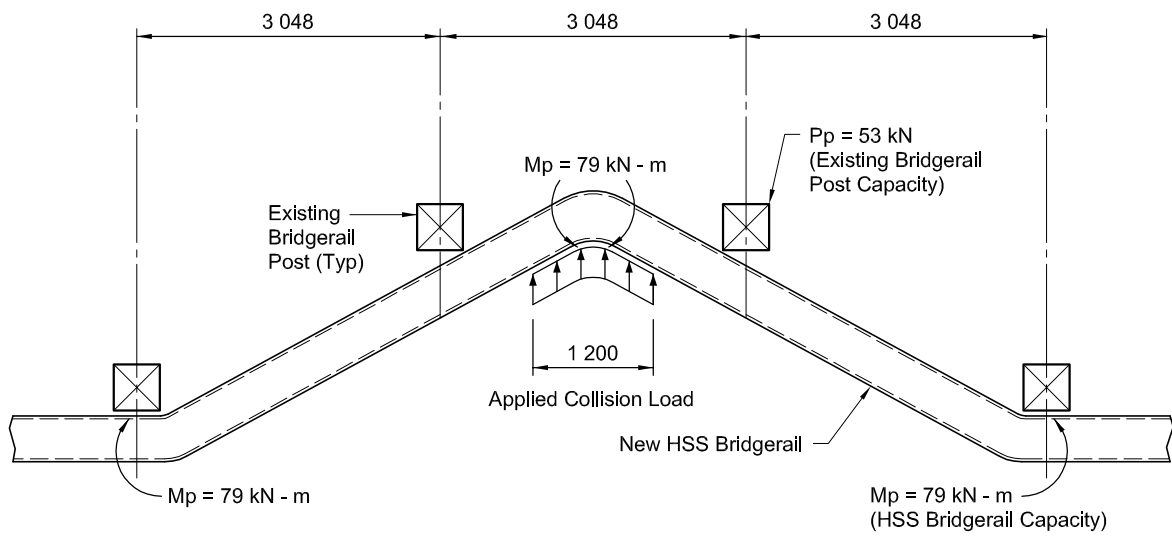


Figure 9 - Upgraded Bridgerail Failure Mode