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Brian Chow, M.Eng., P.Eng.
Chief Engineer
Engineering Branch
Ministry of Forests, Lands, Natural Resource Operations and Rural Development
1520 Blanshard Street
Victoria, BC V8W 3K2

Re: ALTERNATIVE METHODOLOGY FOR DETERMINING THE SHEAR CAPACITY OF END PANELS ON EXISTING GIRDERS

Dear Mr. Chow:

The **Ministry of Forests, Lands, Natural Resource Operations and Rural Development** (Ministry) retained **Associated Engineering** (AE) to develop a methodology for checking the shear capacity of existing girder end-panels that have insufficient shear capacity (LLCF < 1.0) when evaluated using CAN/CSA S6-14 (S6) Section 3 and 14. The purpose of this letter is to a provide a more detailed description of the proposed methodology which is based on Eurocode 3: - Design of Steel Structures – Part 1-5 (EC3) and outlined in AE's April 2020 report, "Steel Plate Girder Shear Design - Anchorage of Tension Field Action".

The following describes the proposed approach for evaluating bridges outside of the warranty period, including a detailed description of the relevant EC3 clauses used to determine the shear capacity of the end panel. For Ministry owned bridges that are within their warranty period use the procedure described below except that the dead and live load demands should be calculated using S6 Section 3 load factors.

1. Determine the factored shear demands using S6 Section 14 assuming:
   - System behaviour – S1 (assumes failure of the girder will result in failure of the of the bridge, i.e. twin girder system)
   - Element behaviour – E3
   - Inspection level – To be confirmed based on the available inspection information
   - Traffic type – PA.
2. Confirm that the bearing and transverse stiffeners conform to S6 requirements (Section 3 Cl. 10.10.6).
3. Confirm the girder end support conditions in accordance with EC3.
4. Determine the shear resistance of the end panel using EC3.
5. Calculated the LLCF as follows:

$$LLCF = \frac{V_{r-EC3} - \sum \alpha_D D}{\alpha_L L(1+I)}$$

Where:

$V_{r\text{-}EC3}$	= Shear resistance calculated in accordance with EC3 (refer to Section 1 of this letter)
α_D and α_L	= load factors calculated in accordance with S6 Section 14
D and L	= Dead and Live Load demands calculated in accordance with S6 Section 14
I	= Dynamic Load Allowance calculated in accordance with S6 Section 3, i.e. no speed restrictions.

If the LLCF ≥ 1.0, there is no need to retrofit the bridge, and the bridge can be posted without any load restrictions.

6. During future visual inspections, confirm that the end panel is not subject to deformation resulting from high shear stresses in the end panel. The expected shear deformation will present itself as buckling of the end panel (for additional information see AE April 2020 report mentioned herein).

1 EC3 END PANEL SHEAR CAPACITY CALCULATION

Clauses 5 and 9 of EC3 present the following methodology for calculating the shear resistance of the girder end panel. For simplicity, we have ignored the EC3 contribution of flanges to the shear strength of the girder. Assuming this, EC3 Equation 5.1 and 5.2 define the shear strength as follows:

$$V_{b,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}}$$

Where:

f_{yw}	= web yield strength
h_w, t	= are dimensions shown in Figure 1-1
η	= 1.2 (for f_{yw} < 426 MPa)
Υ_{M1}	= 1.1 (partial safety factor for resistance to instability)
χ_w	= reduction factor for the shear resistance of the web depending on web slenderness taken from Table 1-1

Further, to determine χ_w EC3 requires the designer identify whether the bearing location can be considered a rigid or non-rigid end post. A rigid end post should comprise two double sided transverses stiffeners that form the flanges of a short beam of length h_w (Figure 1-1 (b)). The strip of web between the stiffeners forms the web of the short beam. Each double-sided stiffener should have a cross sectional are of at least $4h_wt^2/e$, where e is the centre to centre distance between the stiffeners and $e > 0.1h_w$. The girder end plate may act as a double-sided stiffener if it is symmetrical about the centreline of the web, extends the full height of the web and is welded to the web (both sides) and flanges.

A non-rigid end post would consist of a single bearing stiffener as shown in Figure 1-1 (c).

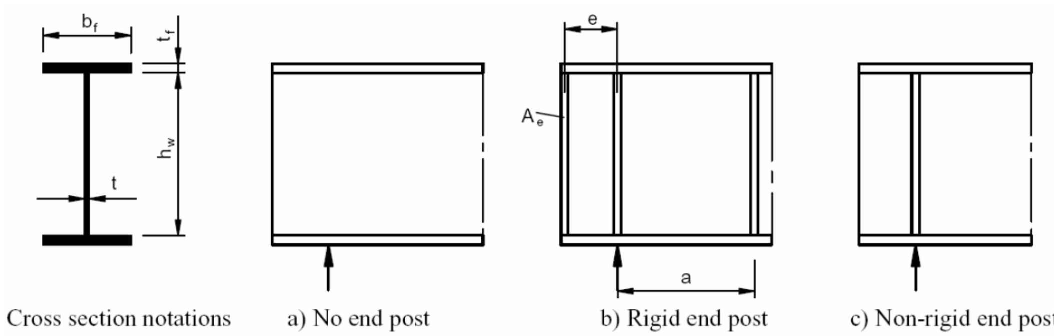

Figure 1-1
Cross Section Notations and End Stiffeners

Table 1-1
Contribution of Web χ_w to Shear Buckling Resistance

	Rigid End Post	**Non-Rigid End Post**
$\lambda_w < 0.83/\eta$	η	η
$0.83/\eta \le \lambda_w < 1.08$	$0.83 / \lambda_w$	$0.83 / \lambda_w$
$\lambda_w > 1.08$	$1.37 / (0.7 + \lambda_w)$	$0.83 / \lambda_w$

For stiffened webs (intermediate transverse stiffeners)

$$\lambda_w = \frac{h_w}{37.4 \times t \times \varepsilon \times \sqrt{k_\tau}}$$

Where:

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

$$k_\tau = 5.34 + 4 \times \left(\frac{h_w}{a}\right)^2 \ when \ {}^{a}/_{h_w} \ge 1.0$$

$$k_\tau = 4 + 5.34 \times \left(\frac{h_w}{a}\right)^2 \ when \ {}^{a}/_{h_w} < 1.0$$

Appendix A provides guidance on the determination of whether rigid end post conditions are present for various configurations of web heights and bearing / end plate configurations. Appendix B provides two worked examples illustrating the described methodology.

Closure

This memorandum was prepared for the Ministry of Forests, Lands, Natural Resource Operations and Rural Development to provide a methodology for an alternative check for existing girders designed incorporating tension field action in end panel shear design.

The services provided by Associated Engineering (B.C.) Ltd. in the preparation of this memorandum were conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty expressed or implied is made.

Respectfully Submitted
Associated Engineering (B.C.) Ltd.

Yours truly,

PROFESSIONAL PROVINCE OF U. A. WEXLER # 33760 BRITISH COLUMBIA ENGINEER

Uri Wexler

May 12, 2020

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Bridge Engineer

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Attachments:

- Appendix A – Methodology for Verifying EC3 Rigid End Post Compliance
- Appendix B – Example Shear Capacity Calculation to Eurocode 3: 2006

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APPENDIX A - METHODOLOGY FOR VERIFYING EC3 RIGID END POST COMPLIANCE

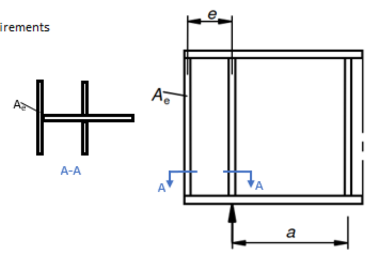
MINIMUM AREA NEEDED FOR END STIFFENER TO COMPLY WITH EC3 RIGID END POST REQUIREMENTS

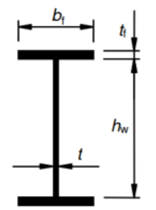
H_w [mm]	700		800		900		1000		1100		1200		1300		1400	
t_w [mm]	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7
e [mm] 100	2530	4520	2890	5165	3250	5810	3610	6455	3975	7100						
150	1685	3015	1930	3445	2170	3875	2410	4305	2650	4735	2890	5165	3130	5595	3370	6025
200	1265	2260	1445	2585	1625	2905	1805	3230	1990	3550	2170	3875	2350	4195	2530	4520
250	1015	1810	1160	2065	1300	2325	1445	2585	1590	2840	1735	3100	1880	3355	2025	3615
300	845	1510	965	1725	1085	1940	1205	2155	1325	2370	1445	2585	1565	2800	1685	3015
350	725	1295	830	1475	930	1660	1035	1845	1135	2030	1240	2215	1345	2400	1445	2585
400	635	1130	725	1295	815	1455	905	1615	995	1775	1085	1940	1175	2100	1265	2260
500	510	905	580	1035	650	1165	725	1295	795	1420	870	1550	940	1680	1015	1810
600	425	755	485	865	545	970	605	1080	665	1185	725	1295	785	1400	845	1510
e_{min} [mm]	70		80		90		100		110		120		130		140	

H_w [mm]	1500		1600		1700		1800		1900		2000		2100	
t_w [mm]	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7	9.5	12.7
e [mm] 100														
150	3610	6455												
200	2710	4840	2890	5165	3070	5485	3250	5810	3430	6130	3610	6455		
250	2170	3875	2315	4130	2455	4390	2600	4650	2745	4905	2890	5165	3035	5420
300	1805	3230	1930	3445	2050	3660	2170	3875	2290	4090	2410	4305	2530	4520
350	1550	2765	1655	2950	1755	3135	1860	3320	1960	3505	2065	3690	2170	3875
400	1355	2420	1445	2585	1535	2745	1625	2905	1715	3065	1805	3230	1900	3390
500	1085	1940	1160	2065	1230	2195	1300	2325	1375	2455	1445	2585	1520	2710
600	905	1615	965	1725	1025	1830	1085	1940	1145	2045	1205	2155	1265	2260
e_{min} [mm]	150		160		170		180		190		200		210	

The bearing and end stiffener must comply with all S6 stiffeners requirements
End stiffener must be fully welded to girder webs and flanges

A_e= end stiffener area
H_w= web height
t_w= web thickness
e= distance between bearing stiffener and end stiffener

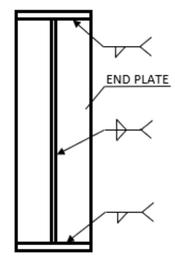




Cross section notations

$$e = 4h_w t^2/e$$

$$e_{min} \geq 0.1h_w$$



Minimum weld requirements

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APPENDIX B – SHEAR CAPACITY EXAMPLE CALCULATION TO EUROCODE 3: 2006

$h_w := 1500\ mm$ web height
$t := 9.5\ mm$ web thickness
$b_f := 375\ mm$ top flange width
$t_f := 19\ mm$ top flange thickness
$a := 2208\ mm$ End Panel Stiffener Spacing
$e := 468\ mm$ distance from support to edge stiffner (girder edge)
$t_e := 12.7\ mm$ beam end stiffener thickness
$b_e := 375\ mm$ beam end stiffener width
$E := 200000\ MPa$ modulus of elasticity
$f_y := 350\ MPa$ Yield Stress
$\nu := 0.3$ Poisson Ratio
$\gamma_{M1} := 1.1$ resistance of members to instability assessed by member checks. EC3-2 Steel Bridges recomended value
$\gamma_{M0} := 1.0$ resistance of cross sections to excessive yielding including local buckling
$\eta := 1.2$ 1.2 recomended for steel grade up to S460, 1.0 for higher grade

$$\varepsilon := \sqrt{\frac{235\ MPa}{f_y}} = 0.819$$

Section 9.3.1 minimum requirements for Rigid End Post

min e required:

$$e_{c1} := 0.1 \cdot h_w = 150\ mm$$

min. rigid end post cross section area required:

$$e_{c2} := \frac{4 \cdot h_w \cdot t^2}{e} = \left(1.157 \cdot 10^3\right)\ mm^2$$

if both checks=OK->Rigid End Post

$$\begin{array}{l} \text{if } e_{c1} < e \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NG”} \end{array} = \text{“OK”}$$

$$\begin{array}{l} \text{if } e_{c2} < t_e \cdot b_{e1} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NG”} \end{array} = \text{“OK”}$$

$$b_{e1} := \left\| \begin{array}{l} \text{if } \dfrac{b_e}{2 \cdot t_e} < 14 \cdot \varepsilon \\ \quad \| b_e \\ \text{else} \\ \quad \| 14 \cdot \varepsilon \cdot 2 \cdot t_e \end{array} \right. = 291\ mm$$

Use only area confirming to Class3 Section

Section 5 End Panel Shear:

$$k_\tau := \left\| \begin{array}{l} \text{if } \dfrac{a}{h_w} \ge 1.0 \\ \quad \left\| 5.34 + 4 \cdot \left(\dfrac{h_w}{a}\right)^2 \right. \\ \text{if } \dfrac{a}{h_w} < 1.0 \\ \quad \left\| 4 + 5.34 \cdot \left(\dfrac{h_w}{a}\right)^2 \right. \end{array} \right. = 7.186 \qquad \text{(A.5)}$$

$$\left\| \begin{array}{l} \text{if } \dfrac{h_w}{t} < \dfrac{31}{\eta} \cdot \varepsilon \cdot \sqrt{k_\tau} \\ \quad \| \text{“no need to check”} \\ \text{if } \dfrac{h_w}{t} \ge \dfrac{31}{\eta} \cdot \varepsilon \cdot \sqrt{k_\tau} \\ \quad \| \text{“check shear buckling”} \end{array} \right. = \text{“check shear buckling”}$$

$$\sigma_E := \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot (1-\nu^2) \cdot {h_w}^2} = 7.251 \; MPa \qquad \text{(A.1)}$$

$$\tau_{cr} := k_\tau \cdot \sigma_E = 52.103 \; MPa \qquad \text{(5.4)}$$

$$\lambda_w := 0.76 \cdot \sqrt{\frac{f_y}{\tau_{cr}}} = 1.97 \qquad \text{(5.3)}$$

TABLE 5.1 RIGID END POST

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7+\lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 0.513$$

TABLE 5.1 NON-RIGID END POST

$$\chi_{w1} := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{0.83}{\lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 0.421$$

χ_w - contribution of the web to the shear buckling resistance
λ_w - slenderness parameter

WEB CONTRIBUTION TO SHEAR:

$$V_{bwRD} := \frac{\chi_w \cdot f_y \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}} = 1343.3 \; kN \qquad (5.2)$$

shear capacity when rigid post

$$V_{bwRD1} := \frac{\chi_{w1} \cdot f_y \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}} = 1103 \; kN$$

capacity when non-rigid post
(when there isn't an end plate)

FLANGE CONTRIBUTION TO SHEAR (NOT USED):

$Med := 1557 \; kN \cdot m$ max demand moment within checked panel

$$c := a \cdot \left(0.25 + \frac{1.6 \cdot b_f \cdot {t_f}^2}{t \cdot {h_w}^2}\right) = 0.574 \; m$$

$$M_{fRD} := t_f \cdot b_f \cdot (h_w + t_f) \cdot \frac{f_y}{\gamma_{M0}} = 3788 \; kN \cdot m$$

$$V_{bfRD} := \frac{b_f \cdot {t_f}^2 \cdot f_y}{c \cdot \gamma_{M1}} \cdot \left(1 - \left(\frac{Med}{M_{fRD}}\right)^2\right) = 62.3 \; kN \qquad \text{(5.8)}$$

EC3 allows for flange contribution to shear. it's usually small.

$$V_{bRD} := min\left(V_{bwRD} + V_{bfRD}, \frac{\eta \cdot f_y \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}}\right) = 1405.6 \; kN \qquad \text{(5.1)}$$

EC3 shear capacity