# REVIEW OF THE MEMORANDUM "PROPOSED REVISIONS TO CAN/CSA-S6-00 TO FACILITATE THE DESIGN OF MONO-SYMMETRIC PLATE GIRDERS – REV 3", PREPARED BY ASSOCIATED ENGINEERING ON JUNE 14, 2004, FOR THE MINISTRY OF FORESTS

## Description of Works/Services

David C. Stringer Engineering Inc. undertakes to carry out an independent review and submit comments on the suitability and applicability of the proposed methodology for CAN/CSA-S6-00 to facilitate the design of mono-symmetric plate girders – Rev. 3, June 14, 2004.

## Terms of Reference

Review the memorandum prepared by Associated Engineering outlining a methodology for the evaluation of the non-composite flexural capacity of mono-symmetric steel plate girders subject to construction loads and provide written comment on the suitability and applicability of the proposed methodology. Upon approval by the Ministry of Forests, this methodology will be included in the *Forest Road Bridge Design and Construction Manual.*

The methodology will cover the design of typical twin girder forestry bridges with pre-cast concrete decks where excavators or front-end loaders are used to place the deck panels. This load is evenly distributed between the two girders. Load factors are specified in CAN/CSA-S6-00 (Steel = 1.1, Concrete = 1.2, Live load = 1.445, Dynamic Load Allowance is not included). Typically though the minimum load factor of 1.25 x (total load effect) governs as required by CL 3.5.1.

## Comments on the Proposed Methodology

We have reviewed the memorandum prepared by Associated Engineering and the relevant portions of the referenced report "Evaluation of CAN/CSA-S6-00 (2000 Canadian Highway Bridge design Code)" and our comments are as follows:

### 1.0 Background

For mono-symmetric steel girders with transverse stiffeners, S6-88 has similar limits on web slenderness to those of S6-00. For example, Clause 7.19.6.2 of S6-88 and Clause 10.10.4 of S6-00 have the same slenderness limit for the web, viz. $2D_c/w \leq 3150/\sqrt{F_y}$. Therefore it does not seem correct to say that the use of S6-00 requires longitudinal stiffeners, which were not required with S6-88.

### 2.0 Limiting web slenderness ratio during construction (AASHTO LRFD (1998))

We are in agreement that this section states the AASHTO 1998 requirements, except that there is a typographical error in the expression for $f_{cw}$ where "=" should be replaced by "$\leq$".

If $f_c$ were equal to $f_{cw}$, equations 6.10.2.2-1 and 6.10.3.2.2-1 would give the same value for web slenderness when the load factor $\alpha$ is equal to 1.41. For values of $\alpha < 1.41$, the most limiting web slenderness will be derived from equation 6.10.3.2.2-1, viz. $2D_c/w \leq \sqrt{\alpha} \times 2551/\sqrt{f_{cw}}$. It is expected that this equation will always control the maximum web slenderness (unless the upper limit of 200 is exceeded) since the overall load factor $\alpha$ would not exceed 1.41 during construction.

## 3.0 Modifications to AASHTO LRFD criteria

By increasing the critical flexural buckling stress by the $\alpha$ factor of 1.25, AASHTO LRFD is essentially checking the stress in the web during construction at the service limit state. Thus the maximum flexural compressive stress in the web due to service loads is allowed to equal, but not to exceed, the theoretical elastic bend-buckling stress. Since the check is carried out under service loads, it is implied that the elastic bend-buckling of the web does not cause failure of the girder, i.e. the girder has post-buckling strength after the bend-buckling of the web occurs. Test results (Ref. 1) have demonstrated that this is the case since the load at failure is approximately twice the load required to cause the theoretical elastic bend-buckling of the web.

As pointed out in the memo, there are some differences in the load factors during construction between S6-00 the 1998 AASHTO-LRFD as noted below:

| | S6-00 | AASHTO-LRFD |
|---|---|---|
| Max DL factory produced components | 1.10 | 1.25 |
| Max DL cast-in-place concrete | 1.20 | 1.25 |
| Construction LL | 1.445 | 1.50 |

In addition, S6-00 Clause 3.5.1 requires that a minimum load factor of 1.25 be applied to the combination of load cases. For cast-in-place bridge decks where the construction live load is relatively small, the 1.25 minimum load factor usually governs rather than the sum of the individual load factors. This may not be true, however, for heavy construction live load equipment such as front-end loaders or excavators placing pre-cast deck panels.

Although the differences in load factors between S6-00 and 1998 AASHTO-LRFD are quite small, it is justified to recognize that these differences exist. The proposal in Section 3.0, to use the maximum service load compressive stress in the web is one option for accounting for the differences in the load factors. In terms of consistency with other parts of the Code however, we are of the opinion that it is preferable to use the factored compressive stress in the web and to use a $\alpha$ factor, which is a weighted average of the S6-00 dead load and the live load factors

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## 4.0 Proposed revision to CAN/CSA-S6-00

As was pointed out in Section 2.0 above, it is expected that the equation $2D_c/w \leq \sqrt{\alpha} \times 2551/\sqrt{f_{cw}}$ will control the maximum web slenderness since the overall load factor $\alpha$ will not exceed 1.41 during construction. This equation for web slenderness is derived from the equation for flexural buckling stress of the web.

In the present S6-00, there is no similar equation for web slenderness based on the flexural buckling stress in the web. The web slenderness limit in S6-00 and in S6-88 is $2D_c/w \leq 3150/\sqrt{F_y}$, and is similar to AASHTO-LRFD equation $2D_c/w \leq 3027/\sqrt{f_c}$ when $f_c = 0.9F_y$. These limits on web slenderness define an upper bound below which fatigue due to excessive lateral bending of the web is not a consideration. For many years, the upper bound on $2D_c/w$ was set at 170 but AASHTO-LRFD 1998 has now raised the value to 200.

Because $2D_c/w \le \sqrt{\alpha} \times 2551/\sqrt{f_{cw}}$ (or the equivalent $2D_c/w \le 2551/\sqrt{f_{const}}$ in terms of service stress) will probably control rather than $2D_c/w \le 3150/ F_y$, the proposed revision to S6-00 imposes a more stringent limit on web slenderness than the present S6-00 for values of $f_{cw}$ which approach $F_y$.

It is logical, however, to include an equation for web slenderness, which is based on the actual flexural compressive stress in the web rather than on a limit involving only the yield stress. It is also logical to use the factored compressive stress in the flange, rather than the yield stress, to limit web slenderness as per the first AASHTO-LRFD equation $2D_c/w \le 3027/ \sqrt{f_c}$. Therefore we are in agreement with the proposal in Section 4.0 that CAN/CSA-S6-00 should be revised to include this double check on web slenderness limits. As discussed in the comments on Section 3.0, however, we suggest that the second check be carried out using factored stress and a $\alpha$ factor, which is a weighted average of the S6-00 dead load and the live load factors

In the last paragraph of Section 4.0, we are of the opinion that "an alternative rational analysis" may not be the best solution when the "above defined slenderness limit or moment of resistance calculated in accordance with S6-00 is exceeded". Instead we recommend that in such cases, the design engineer follow the options in C6.10.3.2.2 of AASHTO-LRFD. These options include providing a larger top flange or a smaller bottom flange to decrease the depth of the web in compression or providing a thicker web.

Conclusions

From the review of the memorandum prepared by Associated Engineering, we are of the opinion that the proposed methodology, modified according to the above comments, is suitable and applicable for the design of mono-symmetric plate girders during the construction stage.

Example

The Daisy Lake FSR @ 1.0km Cheakamus River Bridge is used here to illustrate the proposed methodology, modified according to the above comments.

Section properties:
$A = 32345\ mm^2$, $c = 866.7mm$, $I = 1.44E10mm^4$

$2D_c/w = 2(866.7-19.05)/9.5 = 178$
$3150/ \sqrt{F_y}, = 3150/\sqrt{350} = 168 < 178$ $\therefore$ the web slenderness limit of the current S6-00 (and S6-88) is exceeded and the girder would not pass this code requirement.

With the proposed methodology, modified according to the above comments, it is necessary to compute the maximum factored compressive stress from the construction loading.

With all the pre-cast concrete panels in place and un-grouted and a front-end loader traversing the bridge to place the last panel, the maximum factored moment per girder is computed as follows:

For the steel self-weight including 15% allowance for miscellaneous steel and ignoring the bottom flange taper;
$M_{f1} = (1.1)(1.15)(32345)(77.0)(31.71)^2 / 8x10^6 =$ 396 kN.m

For the concrete deck panel self-weight;
$M_{f2} = (1.2)(225+175)(4268)(23.5)(31.71)^2 / (4)(8x10^6) = 1512$ kN.m

For the front-end loader and one pre-cast panel, assuming a point live load of **230** kN per girder at mid-span;
$M_{f3} = (1.445)(\mathbf{230})(31.71) / 4 = \mathbf{2643}$ kN.m

Thus the maximum moment is $M_f = \mathbf{4551}$ kN.m using the sum of the individual load factors.

If a minimum load factor of 1.25 is applied to the combination of load cases, the maximum moment is $M_f = \mathbf{4311}$ kN.m < **4551**

Therefore, in this example, the maximum moment is controlled by the sum of the individual load factors.

Then the maximum factored compressive stress in the flange from the construction loading is
$f_c = (\mathbf{4551})(866.7)(10^6) / 1.44E10 = \mathbf{274}$ MPa.

$3027/ \sqrt{f_c} = 3027 / \sqrt{\mathbf{274}} = \mathbf{183} > 178$ but $< 200$ OK

The maximum factored compressive stress in the web from the construction loading is
$f_{cw} = (\mathbf{4551})(866.7 - 19.05)(10^6) / 1.44E10 = \mathbf{268}$ MPa.

The weighted load factor $\alpha = \mathbf{4551} / (396/1.1 + 1512/1.2 + \mathbf{2643}/1.445) = \mathbf{1.32}$

$\sqrt{\alpha} \times 2551/\sqrt{f_{cw}} = \sqrt{\mathbf{1.32}} \times 2551/\sqrt{\mathbf{268}} = \mathbf{179} > 178$ OK

(It should be noted that this same value for $2D_c/w$ would be obtained from $2551/\sqrt{f_{const}}$ using service load stresses and $\alpha = 1$)

Therefore both checks on web slenderness are satisfied. Hence the girder meets the requirements of the proposed methodology, modified according to the above comments.

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David C. Stringer Engineering Inc.
August 16, 2004 **Revised August 23, 2004 as shown above in bold underlined type**

Reference

1. Basler, K., Yen, B.T., Mueller, J.A., and Thurlimann, B. (1960), “Web Buckling Tests on Welded Plate Girders”, Welding Research Council Bulletin No. 64, September.