

DISCUSSION

Ponding Calculations in LRFD and ASD

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(Third Quarter 1999)

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INTRODUCTION

In the Third Quarter 1999 AISC *Engineering Journal* paper, "Ponding Calculations in LRFD and ASD", authors Carter and Zuo discussed the present AISC design criteria for ponding as presented in the ASD and LRFD Specifications. In addition, alternative forms of these criteria as suggested by Milbradt (1995) were examined as possible replacements for the present ponding criteria. The objective of this discussion is to provide notes of caution and further identification of the ponding problem.

Modification of analyses presented in Marino's paper (Marino, 1966) and AISC Commentary yields the following criteria for design against overstress by ponding. Given a system of beams (primary members) and purlins (secondary members) the limits of member flexibility for negation of overstress by ponding may be established. Given f_{op} and C_s , the limit of the beam flexibility is:

$$C_p \leq 1.04 - 0.97C_s - 1.27 \frac{f_{op}}{F_y} \quad (1)$$

or given f_{os} and C_p then:

$$C_s \leq 1 - 1.07C_p - 1.25 \frac{f_{os}}{F_y} \quad (2)$$

These criteria assume an increase in F_b to $0.8F_y$ for temporary ponding plus other live and dead loads.

$$C_p = \frac{32L_s L_p^4}{10^7 I_p}$$

$$C_s = \frac{32SL_s^4}{10^7 I_s}$$

where:

F_b = allowable bending stress, psi

E_p = modulus of elasticity of primary member (beam), psi

E_s = modulus of elasticity of secondary member (purlin), psi

I_p = moment of inertia of primary member, in.⁴

I_s = moment of inertia of secondary member, in.⁴

L_p = span of primary member, ft

L_s = span of secondary member, ft

S = spacing of secondary member, ft

f_{os} = stress in secondary member prior to ponding, psi

f_{op} = stress in primary member prior to ponding, psi

The criteria of Equations 1 and 2 are offered to replace the ASD and LRFD Specification equation of:

$$C_p + 0.9C_s \leq 0.25 \quad (3)$$

and the curves of the appropriate Commentary. The Commentaries introduce the initial stress parameters, f_{os} and f_{op} , to provide a more accurate solution. The present ASD and LRFD Specifications assume $f_{op} = f_{os} = 30$ ksi and Equations 1 and 2, with this assumption reduce to:

$$C_p + 0.97C_s \leq 0.28 \quad (4)$$

$$C_s + 1.07C_p \leq 0.25 \quad (5)$$

The above design equations present a solution to a non-linear problem; the applied loads and resulting stresses are functions of the deflections. One example of the formulation and solution for the simple beam is that by Sawyer (1967). The solution is an eigenvalue for the flexibility of a simple span, $C_s = 1$, the value that defines neutral equilibrium. For $C_s > 1$ instability occurs; for each inch of water load the beam deflects greater than one inch. This result is also implied in the Marino (1966) solution, but in that solu-

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tion, a design stress is introduced to limit flexibility and ponding.

The general ponding problem is soluble by use of an iteration approach using a finite element program; however, obtaining such a solution is not usually within design time constraints.

There are a number of parameters that were not considered in establishing the above inequalities that reduce the apparent factors of safety and these should be considered in application of the criteria.

RESIDUAL STRESSES

A second nonlinear effect in the ponding problem is that associated with residual stresses creating a proportional limit, F_p , at 0.5 to 0.9 of the yield point stress, F_y . At stresses greater than F_p , the modulus of elasticity, E , tends to the tangent modulus, E_t , and $E_t < E$. The incremental flexibility constants become, for $f_b > F_p$,

$$\Delta C_p < \Delta C_{pt} = \frac{93L_s L_p^4}{E_t I_p}$$

and

$$\Delta C_s < \Delta C_{st} = \frac{93SL_s^4}{E_t I_s}$$

For $f_b > F_p$, each unit load increment causes increases in the displacement increment thereby creating loads larger than those assumed in the development of the AISC ponding criteria. The strength of the roof structure is not affected; however, the flexibility requirements, C_p and C_s , for stability are lowered.

Residual stresses are created during rolling processes, welding, forming, etc. The AISC column equations assume the residual stress, F_r , to be equal to $0.5F_y$ for rolled sections. For welding, with no stress relief, the tensile residual stresses in and around weld volumes are on the order of F_y . Welds present in fabricated rolled sections, joists, and joist girders will affect the F_r and F_p values with local plastic deformation before local strain hardening occurs. Obviously, the presence of residual stresses, not calculable, may introduce a detrimental but ignored factor in the AISC calculations for limiting the values of C_p and C_s to prevent ponding.

DETERMINATION OF f_o

The value of f_o is defined as that maximum stress in the roof elements associated with all loads other than those created by ponded water associated with deflections. Thus dead loads, ice, snow, accumulated water (due to roof geometry, ice dams, blockage or inadequate drains,

hydraulic gradient for discharge flow) are all used in the computation of f_o . The rain intensity and duration may provide water volume input that exceeds the primary drainage capacity. For any particular location the accurate prediction of maxima for these rain variants is not likely; secondary positive drainage should be provided. Naturally, the hydraulic gradient for flow to the primary or secondary drainage should be considered (1 in.-3 in.) in the calculation of f_o . "Free" flow over the edge of the roof implies that the height of the gravel stop, or other geometric barriers, are to be added to the hydraulic gradient head for the calculation of accumulated water load.

VALUES FOR MOMENT OF INERTIA

The I values for rolled beams are readily available; however, those for bar joists and joist girders are not published with the general load tables for same. These latter I values, for the same design load capacity, vary due to fabrication with different chord and web elements. The *SJI Tech. Digest* No. 3, "Ponding Loads" contains a list of I values for joists and an equation for computation of I_e for joist girders to be used in the AISC Ponding Specifications. For joists the *Digest* No. 3 uses the equation:

$$I_e = \frac{0.85A_t A_b d^2}{A_t + A_b}$$

A_t and A_b are the areas of the top and bottom chords and d is the depth between the top and bottom chord center of gravities.

Computations were performed to compare the values of the moment of inertia given in the *SJI Tech. Digest* No. 3 to those obtained from the measured geometry and areas of fabricated joists and joist girders. A computer analysis for load-deflection relationships for the actual structural elements allowed computation of the effective moment of inertia, I_e . The results indicate the effective moment of inertia for the joists to be approximately 15 to 20 percent higher than those listed in the *SJI Tech. Digest* No. 3. Moreover, the 0.85 factor for the web deformations varied from 0.78 to 0.90. This correction factor was used to correct the moment of inertia calculation based on the chord areas; or $I_e = 0.85I$. The 0.85 correction factor is an approximate average value but not necessarily a conservative value.

Results of the same computations for some joist girders indicated that the SJI equation yielded I_e values 2 to 10 percent greater than those calculated using the geometry of fabricated joists. Obviously, the number of samples for comparison were limited but did indicate possible inaccuracy of results when using these SJI values for ponding computations.

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A ROOF COLLAPSE ANALYSIS

The following data and analysis results are presented for a low-slope roof that collapsed, to illustrate the sensitivity of the AISC Commentary procedure to the input value of f_o .

A roof structure design based on ASD criteria for allowable stresses as the only criterion, resulted in values of $C_p = 0.21$, $C_s = 0.25$, and $f_o = 26$ ksi (accumulation of water in valleys) with $F_y = 50$ ksi. Using Equation 3 the design $C_p \leq 0.03$, and by the Commentary $C_p \leq 0.14$ which are both less than the given flexibility of $C_p = 0.21$. The provided elements do not meet the AISC criteria. By Milbradt's Equation 4, $C_p \leq 0.04$ for $f_{op} = 30$ ksi = assumed f_{op} value for Equation 3. For $f_{op} = 26$ ksi, the C_p limiting value is $C_p \leq 0.14$.

Obviously, the results of Equation 3 and Commentary do not differ significantly from the results given by Equations 1 and 2.

Iterative analyses indicate that, for the given data, collapse would occur. Actual structure collapse did occur for two separate roofs with the primary members initiating collapse. The secondary members were not as critical as the primary members.

If $C_p = 0.21$, $C_s = 0.25$, and $F_y = 50$ ksi, then the AISC Commentary criteria and Milbradt's equation indicate a limit of $f_{op} \leq 23$ ksi for a design solution. The 23 ksi is only 3 ksi less than that causing collapse. The 3 ksi difference in f_{op} changes the design value of C_p from 0.21 to 0.14, or a 33 percent reduction in flexibility.

COMMENTS AND SUGGESTIONS

It is suggested that Equations 1 and 2 be used to replace those in the AISC Specifications. The replacement will provide a simplification of the ponding criteria of the Commentary and make more apparent the influence of f_o in the ponding phenomenon. Numerous factors for determination of f_o may be listed in the Commentary with short discussions of the roles of residual stresses in collapse causation and the problems in determination of moments of inertia for joists and joist girders.

The accuracy of the suggested replacement equations is sufficient, given the approximations in the development of AISC criteria for prevention of collapse by ponding.

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