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steel interchange

Historical Requirements for Secondary Members

Until the 1978 AISC *Specification*, allowable stresses for bracing and secondary members with *l/r* ratios greater than 120 were provided with a higher allowable stress than main members by dividing the allowable compression stress by the factor [1.6–(*l*/200*r*)]. The justification was based on these members being relatively unimportant and also because of the greater effective end restraint likely to be present at their ends.

The stress increase was no longer allowed beginning with the 1989 AISC *Specification*. It seems that any secondary members in structures evaluated under the older specifications would be deemed unsafe when analyzed using the newer editions of the specification. Is this correct?

No. It is likely that the provision was removed in part because of the difficulty in defining a secondary or bracing member and also relative unimportance. That said, there is no Commentary describing the reason for the change, so all I can offer is my approach to the situation.

There seems to have been three independent provisions in the 1978 AISC *Specification*:

1. when $Kl/r < C_c$, when $Kl/r > C_c$
2. when $l/r > 120$
3. when the member is a secondary or bracing member

However, the third case is also a subset of the other two and is really a simplification that allows the designer to assume in certain cases that $K = 1.0$. This increases the allowable stress to account for "greater effectiveness of end restraint likely to be present at their ends." As stated in the Commentary in 1969: "The formula should be restricted to members that are more or less fixed against rotation and translation at braced points."

The earlier editions of the *Specification* do not prohibit the use of the first two approaches for secondary members.

With this in mind, the comparison should not be made to the current equations with $K = 1.0$, but rather to the current equations with $K = 0.65$. The existing condition would also have to be evaluated to ensure that the original intent was met—i.e., the member was "more or less fixed against rotation and translation at braced points." I have made the comparison (in the table below) between the increase allowed for the secondary members to the increase resulting from the change in *K* from 1.0 to 0.65. Note that this comparison is likely not exactly what you are interested in, since the increase for secondary members would be from the older column equations and the increase due to *K* values is based on the current AISC *Specification*. However, it does reflect the conservative nature of the antiquated provision for secondary members. Even if the restraint were more towards the "less" side of more or less, you could probably still justify the strength of the member.

As indicated in the earlier commentary, since the member is flexible it would not take much of a connection to justify a fixed condition.

l/r	$0.65l/r$	$1/(1.6–(l/200r))$	Table 4-22 ASD stress		Increase Due to K
			l/r	$0.65l/r$	
120	78	1.00	10.1	15.6	1.54
140	91	1.11	7.67	13.9	1.81
160	104	1.25	5.78	12.2	2.11
180	117	1.43	4.64	10.5	2.26
200	130	1.67	3.76	8.86	2.36

Larry S. Muir, P.E.

Short-Term Corrosion

We have a project with minor rusting that has started since the steel was erected about two months ago. The building will be exposed for another four months and then will be enclosed. We are particularly concerned about the effect on the bolts, since they have a smaller cross-sectional area than the members. Should we be concerned about the corrosion?

Albrecht and Hall (2003) compiled atmospheric corrosion data for carbon and weathering steels, with graphs of thickness loss versus exposure time for rural, industrial and marine environments. The graphs can be used to predict the surface material loss due to uniform corrosion. In the presence of oxygen and moisture and the absence of contaminants, Xanthakos (1996) noted that unprotected steel has a uniform corrosion rate of about 0.008 in. per year. If the element is continuously wet or exposed to chlorides found in deicing salts and marine environments, then pitting or local corrosion can occur at a rate of 0.012 in. per year. Using the worst case from both publications, the corrosion loss for six months of exposure is much lower than the bolt dimensional tolerances. Therefore, any light uniform corrosion incurred during normal erection of uncoated steel is typically acceptable.

Additionally, if the joints were designed and installed according to 2010 AISC *Specification* (available at **www.aisc.org/2010spec**) Section J3, any corrosion should be reduced at the areas of the bolt that are highly stressed. After the bolts are installed, the bolt shank and engaged threads will tend to be protected from moisture. For sealing against the penetration of moisture, the geometry of bolted joints must provide tight contact between faying surfaces. This is accomplished by limiting the fastener spacing and edge distance according to AISC *Specification* Section J3.5. This is also an important though often-forgotten reason that bolts are required to be installed to at least a snug-tightened condition as required in *Specification* Section J3.1. The snug-tightened condition is defined as the tightness required to bring the connected plies into firm contact.

Here are the details on the references we cited:

- Albrecht, P. and Hall, T.T. (2003), "Atmospheric Corrosion Resistance of Structural Steels," *Journal of Materials in Civil Engineering*, Vol. 15, No. 1, February, pp. 2-24.
- Xanthakos, P.P. (1996), *Bridge Strengthening and Rehabilitation*, Prentice Hall.

Bo Dowswell, P.E., Ph.D

Percent Composite Action

The Commentary to Section I8 of the *Specification* states: "The degree of composite action, as represented by the ratio $\Sigma Q_n/F_yA_s$ (the total shear connection strength divided by the yield strength of the steel cross section), influences the flexural strength." Must the percent composite action always be greater than 50%?

The AISC *Specification* does not specify a minimum percent composite. The section you have highlighted is part of general discussion in the Commentary. It is not a requirement, so you are allowed to use a lower percent of composite action based on your own engineering judgment.

The 50% recommendation has raised a number of questions, and the AISC committee that oversees the composite design provisions has spent a fair amount of time in the last few years discussing and evaluating minimum composite requirements. There will be some new language on this topic in the 2016 AISC *Specification* and Commentary. I'll try to give you the brief synopsis of what you'll see in the near future without overwhelming you with too much detail.

Historically, a minimum of 25% composite action has been recommended (but not required) for composite beams. However, certain research over the years has indicated that in some scenarios, low percentages of composite behavior could result in a non-ductile failure of the headed studs at the beam ends with the potential for a "zippering" effect of stud failures and a drastic reduction in member capacity. As a result of this research, the language that you highlighted was added to the Commentary, recommending a minimum of 50% composite action.

The current Commentary recommendation implies all beams should meet the 50% composite minimum, but this could be excessive in many situations. The 2016 AISC *Specification* will still not require a minimum percentage of composite action, but it will include a new requirement to "consider ductility."

The 2016 Commentary to the *Specification* will remove the current language that discusses 25% composite as a minimum and include new language that gives guidance on how to "consider ductility." Within this discussion it identifies three exceptions where ductility need not be evaluated:

- Beams spanning 30 ft or less in length
- Beams with 50% composite action
- Beams with an average of 16 kips/ft shear connector capacity (this equates to roughly 1 stud/ft but can also be used when looking at beams with skewed deck or similar conditions where 1 stud/ft cannot be installed)

For beams that do not meet these criteria, ductility will need to be more carefully considered. The Commentary indicates that data obtained from numerical analysis can be used as one method of considering ductility, and provides references to a few different analytical approaches, including the June 1995 *Journal of Structural Engineering* article "Composite Beams with Limited-Slip-Capacity Shear Connectors" by Oehlers and Sved.

Susan Burmeister, P.E.

Multiple Conditions in a Single WPS

Is it permissible for one prequalified welding procedure specification (WPS) to list multiple combinations of variables?

Yes. AWS D1.1 gives latitude to the fabricator relative to the form the WPS takes. Annex Q provides some guidance. The Commentary to AWS D1.1, C-Table 3.8 – Item 3 addresses and permits multiple combinations of variables in a single WPS.

Larry S. Muir, P.E.

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