A36 a G ad 50 a

The material test reports for the A36 plates used in my projects consistently conform to material with a yield of 50 ksi. I have two questions: Can I design these plates accounting for the higher yield strength? And should I start specifying all plate material as A572 Grade 50 instead of A36?

[illegible]

La S. M , P.E.

L ca B c

The local buckling limits in AISC *Specification* Table B4.1 are all based on the square root of the inverse of the yield strength. This means for a given shape, a lower-strength steel will result in a compact section while a higher-strength steel will not. How can the shape made from the higher-strength steel have a lower strength, as predicted by the compactness limit?

[illegible]

La S. M , P.E.

D M E d Pa

The plates in an end-plate moment connection have distorted due to the shrinkage of the welds. Is there a tolerance on such distortions?

[illegible]

[illegible]

I have designed end-plate moment connections using the procedure in AISC Design Guide 16, which allows the use of snug-tightened A325 bolts. The erector has installed tension control (TC) bolts, which are pretensioned. Will this cause a problem?

[illegible]

[illegible], P.E.

[illegible]

I have typically used the term *VQ/I* to determine the shear on the welds joining the flanges and web of a built-up girder. When a girder is designed based on the plastic section modulus, the flexural strength is significantly higher than that predicted using the elastic modulus. Is *VQ/I* still appropriate when the plastic section modulus is used?

[illegible] *VQ/I*, [illegible]