PRACTICAL, TECHNICAL AND LEGAL PITFALLS OF EXCESSIVELY RESTRICTIVE STEEL FABRICATION AND ERECTION TOLERANCES THAT EXCEED ASTM A6 MILL TOLERANCE


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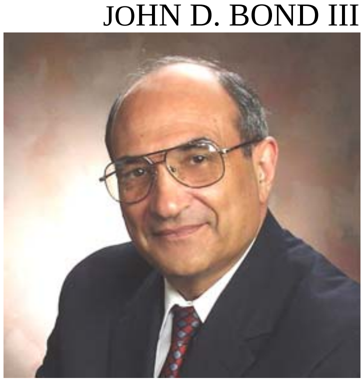
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BIOGRAPHY

John D. Bond III, former president of J.A. Jones Construction Company, is a practicing construction lawyer from Charlotte, North Carolina.

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SUMMARY

The intent of this paper is to bring steel bridge industry focus to the technical, practical and legal consequences of fabrication and erection tolerances that may fail to consider the cumulative implications or be more restrictive than ASTM A6 mill tolerances. Bridge designers often do not pay attention to the tolerances and imperfections inherent in the steel milling process and the specified steel material produced. To avoid the harsh consequences to all contracting parties that often result, contract documents should be prepared in a manner that prominently and clearly alert contractors, structural steel fabricators and erectors to the existence of special or restrictive dimensional tolerances. Examples are cited of the cases in which mil tolerances of ASTM A6, specified by the Contract Documents, contradicted the restrictive tolerances imposed on the fabrication process, based on concerns that are shown by to be unfounded, but resulted in delays and extra costs to the project

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The intent of this paper and conference panel discussion is to bring steel bridge industry focus to the technical, practical and, to a lesser extent, legal consequences of fabrication and erection tolerances that may fail to consider the cumulative implications or be more restrictive than ASTM A6 mill tolerances.

From a legal perspective, it bears note that since the 1918 US Supreme Court decision *U.S. v. Spearin*, project owners in the US and, by extension, their designer consultants have been held to impliedly warrant that their design plans and specifications will be complete, accurate and sufficient to produce the desired project technical results, if followed. *Spearin*, as followed and expanded by many subsequently issued, multi-jurisdictional, judicial decisions, also held that an owner's breach of this implied warranty of design will entitle an aggrieved contractor to recover resulting, provable damages. This legal duty of the implied warranty of design, applies not only to specified methods of construction and fabrication, but also, to the particular materials and components specified for project use.

Since being founded in 1898, the ASTM has been the gold standard for defining the physical properties and testing standards of structural steel material used in the design and construction of steel bridges throughout the world. As a matter of well-educated reflex, bridge designers routinely specify use of ASTM-described steel material grades, such as ASTM A709/A709M-13a, *Standard Specification for Structural Steel for Bridges* to be provided in compliance with the ASTM A6/A6M-13a, *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates and Sheet Piling* ("ASTM A6").

From the platform of ASTM-based development of structural steel bridge specifications, in some cases bridge designs proceed to their often majestically complex conclusion, of height, span and shapes, without sufficient attention being given to the tolerances and imperfections inherent in the steel milling process and the specified steel material produced. To avoid the harsh consequences to all contracting parties that often result, contract documents should be prepared in a manner that prominently and clearly alert contractors, structural steel fabricators and erectors to the existence of special or restrictive dimensional tolerances. The steel bridge industry, as a whole, should remind itself of the fact that structural steel plates and shapes are not dimensionally perfect, and that even in the most perfect of milling conditions, the influence of milling equipment wear, thermal distortion and differential cooling conditions assure that steel plate is not going to be perfectly flat, of perfectly milled thickness and devoid of waviness or surface roughness. These unavoidable steel milling imperfections, mill tolerances, are prescribed in the ASTM A6.

The situations that cause this steel tolerance issue to be brought to renewed light is that, not infrequently, bridge owners and/or designers impose structural steel fabrication tolerances in cross-sectional and planar configuration and alignment of heavy-thickness structural steel plate and steel shapes components and members, without sufficiently alerting contractors and fabricators that ASTM A6 mill tolerance for flatness, thickness and waviness for the specified steel may exceed imposed fabrication tolerances. A practical consequence is that, in the heat of pricing and planning the work, fabricators and erectors, who are in many instances are not degreed structural engineers, fail to fully comprehend that allowable ASTM A6 mill tolerances in the specified structural steel material may render the consistent achievement of specified tolerances commercially unfeasible or practicably impossible through use standard fabrication and erection practices. When that happens, a likely legal result is that the project design may be found to be defective, exposing the project owner to liability for contractor and fabricator cost overruns and project delays.

On this point, the AISC *Code of Standard Practice for Steel Buildings and Bridges* ("Code of Standard Practice") is instructive. Specifically, the Code of

Standard Practice, AISC 303-10, states its overarching general scope that "[i]n absence of specific instructions to the contrary in the Contract Documents, the trade practices that are defined in the Code of Standard Practice shall govern the fabrication and erection of Structural Steel."

This AISC Code of Standard Practice general scope statement of applicability to the fabrication and erection industry is further explained both internally and by Commentary, specifically:

1- "If a special design concept or system component requires a tolerance that is not specified in this Code, the necessary tolerance should be specified in the contract documents." See *Code of Standard Practice, Commentary to* §9.1.
2- "When the Owner issues design drawings and specifications that are released for construction, the fabricator and the erector rely on the fact that these are the owner's requirements for the project." See *Code of Standard Practice, Commentary to* §4.1.
3- "Normal variations in the cross-sectional geometry of standard structural shapes must be recognized by the designer, the fabricator, the steel detailer, and the erector. Such tolerances are mandatory because of roller wear, thermal distortions…and differential cooling distortions [that occur at the steel mill] are all unavoidable. Geometric perfection of the cross-section is not necessary for either structural or architectural reasons, if the tolerances are recognized and provided for." See *Code of Standard Practice, Commentary to* §5.1.2.
4- "When special tolerances that are more restrictive than those in ASTM A6/A6M are required for Mill Materials, such special tolerances shall be specified in the Contract Documents." See *Code of Standard Practice, Commentary to* §5.1.4.

Taken together, the ASIC Code of Standard Practice requirements and explanatory Commentary acknowledge fabrication and erection limitations and provide reasonably clear guidance on the responsibilities of bridge project designers for special tolerances. Those requirements are written not only with the intention of guiding designers, but also in the express expectation of being relied upon by fabricators and erectors in the performance of their respective work.

By way of a specific example, consider a case involving a mill-to-bear connection at a tied-arch bridge heel assembly, at an end of a truss where the lower edges of the internal stiffeners bear against the top surface of the bottom flange of a box beam. (In the past, the edges of bearing stiffeners had to be milled to provide sufficiently flat and straight surfaces – hence the term mill-to-bear; however, present day cutting methods are sufficiently accurate to provide a straight stiffener edge that will bear satisfactorily against flange plates; therefore, milling is no longer necessary.) Here, the project owner's inspectors interpreted mill-to-bear as requiring contact in the unloaded condition over the entire stiffener-to-flange contact surface and consequently rejected/stopped fabrication of the heel assemblies because they determined that fabricator had not maintained mill-to-bear requirements, as gaps allowing the passage of light were noted along the contact surface. The fabricator reasonably believed that contact was only required over 75% of the contact surface and defined contact to mean a range from no gap to a 0.010 in. gap.

To resolve the impasse, the owner's engineer issued its "recommendations" on the mill-to-bear contact, reiterated that there is no allowance for any gap when articles are to be in contact, directly contradicting the provisions of AWS, which had been incorporated into the contract documents, stating that the "bearing ends of bearing stiffeners…shall have at least 75 percent of this area in contact with the flanges." The commentary of AWS further clarifies that "bearing does not necessarily mean full contact, but is as close as specified." AWS states that contact is to be specified by the designer, yet nowhere in the plans and specifications was "contact" defined. Hence, after challenge, the bridge owner reversed its position and stated that it was permissible to have a gap even exceeding 0.010 in. for up to 25% of the joint area, but the remaining 75% of the area must have a gap of less than 0.005 in. Unfortunately, this new owner position was not based on, and did not consider, the physical reality of ASTM A6 mill tolerances of steel components, which had been incorporated into the contract documents, and ignored the guidance provided by the applicable commentary to governing AWS provisions. Further, the owner did not seek the American Welding Society's interpretation of what constitutes contact at a mill-to-bear connection.

Here, the project specifications only required that the "milled or ground ends of stiffeners" conform to a surface roughness of 500 micro in. This specified roughness is consistent with surfaces resulting from standard cutting processes, as compared to machined (struck or "milled") surfaces which yield not only substantially reduced roughness, but also reduced flatness and waviness, and requires a much more expensive and complicated fabrication process than what was required by the contract documents. Surface waviness is the "widely spaced component of surface texture" that can be caused by "machine or work piece deflections, vibration, and chatter"; see ASME B46.1-2009 Surface Texture, which was incorporated into the project specifications. By comparison, surface roughness is "the finer spaced irregularities…that usually result from the inherent action of the production process or material condition." Waviness can be viewed as similar to ocean swells and roughness can be viewed as the ripples on the surface of an ocean swell. These surface variations are an inherent and unavoidable consequence of the process used to make the plates. Because of this, they have been recognized and accepted by the industry, incorporated in ASTM A6, and should have been anticipated by the design engineers.

Let us now examine the implication of mill tolerances of ASTM A6. The bottom flanges of the box girders were made from 2.25 in. thick by 74.06 in. wide by 19 ft long plates. In the case of the rolled steel plates used for the flanges, their dimensional variations are established in ASTM A6, which was incorporated into the project specifications. These dimensional variations address plate thickness as well as plate surface flatness and waviness. As applied to the bottom flange plate that the 34.5 in. wide edge of the internal stiffeners bear against, ASTM A6 permits the following dimensional variations:

- Thickness of the bottom flange plate near edges can vary between 2.24 in. and 2.35 in. (a 0.015 in. variation), while elsewhere it can be between 2.24 in. and 2.43 in. (a variation of 0.019 in.).
- Flatness of the bottom flange plate can vary up to 0.75 in. across the width of the 74 in. wide plate.
- Waviness of the bottom flange plate can vary from 0.125 in. for 7 waves across a 12 ft length (corresponding to 20.5 in. wavelength). For a single wave over the 12 ft length, the bottom flange waviness over the width of the stiffener has an amplitude of 0.05 in.

The observed gaps at the mill-to-bear connections due to waviness of the bottom flange in the unloaded condition will diminish and possibly disappear once loaded. This is recognized by the commentary to AWS, stating "it is not essential that the parts bear completely before all loads are applied."

Using plates that meet the project specification's dimensional tolerances as stated above, the contact surface tolerance required by the project owner could only be achieved if the fabricator undertook exceptional machining efforts to essentially sculpt the bearing surface/edge of the stiffeners to conform to the non-uniform, yet ASTM A6 compliant, profile of the flange plate's surface, or, most likely, machine both surfaces perfectly flat. This level of effort was beyond the scope of the project specifications and was not contemplated by the American Institute of Steel Construction's fabrication provisions, as contained in the Steel Manual. The Steel Manual states that "planing or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the design documents or included in a stipulated edge preparation for welding" and assumes that the edge or surface is not sculpted to match the uneven profile of a flange plate's surface.

Alternatively, it could be argued that the contact area on the flange could be machined down to a flat surface where the stiffener would come into contact with it. However, there was nothing on the drawings or in the project specifications that specifically required such a processing of the flange plates. Additionally, even if such a provision existed, the act of machining a flat, straight contact surface into an ASTM A6 compliant flange plate would potentially cause a substantial reduction of the local thickness of the flange plate. If full contact was required, the designer should have specified final dimensions, special tolerances, and the machining required to accomplish such tolerances in accordance with the Code of Standard Practice.

In summary, in this example, owner inspectors and subsequently its engineer imposed tolerances at mill-to-bear connections that were nearly impossible because of inconsistent and contradictory requirements in the drawings and specifications as interpreted by the owner. In the design document, standard mill tolerances were specified without imposing enhanced tolerances or requiring special machining for stiffener installation, yet the inspectors and engineer called for stringent contact tolerances which were not possible using components having the standard mill tolerances. The owner inspectors and engineer improperly interpreted the mill-to-bear tolerance, resulting in the imposition of a far more stringent interpretation of mill-to-bear than is warranted and more than what is recognized and required in the industry or feasibly possible to achieve in the context of the standard ASTM A6 steel mill tolerances. Finally, the inspection results had not been scrutinized considering the effect of expected dead load, which would have substantially reduced or even eliminated much of the gaps that the inspectors observed.

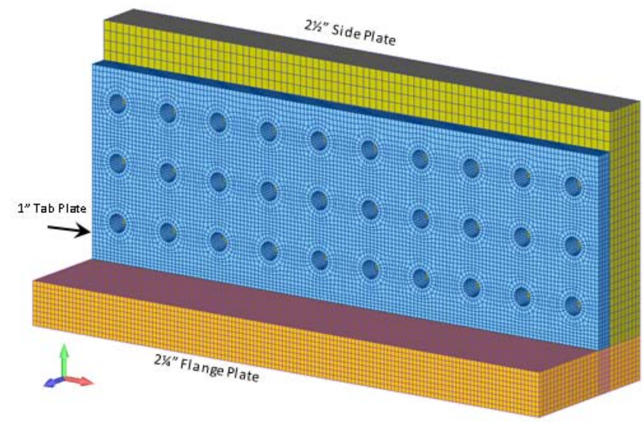

Figure 1 – Finite element model of the connection.

By way of a second example, consider the case of a box girder, as shown in Figure 1, made of three basic steel plate components: 2.25 in. thick top and bottom flange plates, 2.50 in. thick side or web plates, and 1.00 in. thick tab plates. These plates met the required ASTM A6 mill tolerances and were cut using standard cutting procedures. No special machining of the faying surfaces or special gap tolerances was specified. The assembly of the box girder began by positioning the tab plates from the centerline of the bottom flange plate, fixing them in position, and then welding them to the bottom flange plate using double bevel welds applied in multiple passes. The resulting welding distortion was then removed by heat straightening, and the excess weld metal on the outside of tab plate was ground off to form a square profile. The web plates are then placed on the outside surface of the tab plates and bolted to the tab plate using 7/8 in. diameter A325 bolts torqued to produce 39 kip nominal compression between the faying surfaces. The connection thus produced is a slip-critical connection and relied on the friction between the clamped plates for slip resistance. The slip resistance is a function of the clamping force and coefficient of friction. The design is based on a Class A (clean mill scale) surface with a minimum coefficient of friction of 0.33 based on Specification for Structural Joints Using High-Strength Bolts, Appendix B, § B5.4 (2009).

During fabrication of the box girders, the owner inspectors initially interpreted the specifications as requiring <u>full contact</u>, and then, based upon an interpretation of AWS 1.5 Article 3.5.1.14 subsequently required a gap tolerance of no greater than 1/16 in., over the entire faying surface and consequently refused to accept the assemblies as gaps were noted along the faying surfaces. Unfortunately, the drawings and specifications, and the special provisions to the standard specifications, were silent on such required tolerances. The basis of the decisions or indecisions made by the inspectors and owner personnel were as follows: (1) Pre-bolt contact, or that not exceeding 1/16 in. was not present between the faying surfaces in bolted connections; (2) Gaps compromised the slip-critical connection of the design by materially decreasing the clamping force between the faying surfaces; (3) Bolt forces cause yielding and large plastic strains at the base of the tab plates; such plastic strains compromised the ability of the tab plates to resist the loads between the connecting members and components.

As applied to the above-described box girders, ASTM A6 permits the following variations in thickness and flatness for the high-strength low-alloy rolled plates:

For the 2.50 in. thick by 7 ft wide by 19 ft long side plates:

- Thickness can vary from 2.49 in. to 2.68 in.
- Flatness can deviate from a perfectly flat plane up to 3/4 in. over its entire length and it can have waviness of up to 1/8 in. over shorter wavelengths.

For the 1.00 in. thick by 9.75 in. wide by 19 ft long tab plates:

- Thickness can vary from 0.99 in. to 1.11 in.
- Flatness can deviate from a perfectly flat plane up to 9/16 in. over its entire length and it can have waviness of up to 1/8 in. over shorter wavelengths.

ASTM A6 addresses tolerances for plate waviness, which is the maximum deviation of the surface relative to a plane formed by the adjacent wave peaks. As the number of waves increase along an assumed 12 ft long portion of the plate, the maximum allowed deviation of a valley relative to the adjacent peaks decreases. Taking the 2.50 in. thick side plate as an example with 7 waves in 12 ft distance (one wave every 21 in. or so), the maximum deviation allowed between any valley and its adjacent peaks decreases to 1/8 in.

To investigate the behavior of the connection when the tab plate was not in contact with the side flange, a three-dimensional finite element (FE) model of the connection was generated and analyzed. The overall geometry of the model was based on the provisions of ASTM A6, which establishes standard mill tolerances for plates, including limits on flatness and waviness. These mill tolerance provisions were applied to the 1 in. thick tab plate with a deviation from flatness of 1/8 in. assuming conservatively that other components of the model, namely the 2.25 in. flange plate and the 2.50 in. side plate, were perfectly flat and straight.

The FE model, as shown in Figure 1:

- Simulates a 30 in. long portion of the connection,
- Contains three rows of ten 7/8 in. bolts each,
- Permits non-linear yielding and strain hardening behavior when the plate stress reaches 50 ksi, which is the minimum specified yield strength of the steel, and
- Simulates the 44-kip clamping force at each of the 30 bolts.

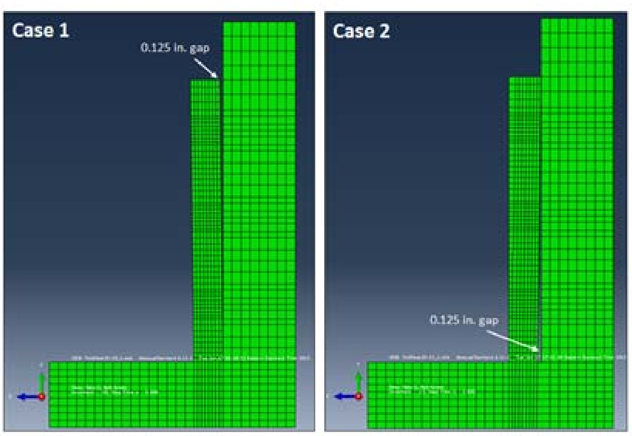

Figure 2 – Cross sections showing gaps assumed in Cases 1 and 2

Two types of tolerance geometry for a maximum limited "gap" of 1/8 in. were studied, as shown in Figure 2:

- In Case 1, the 0.125 in. maximum gap is at the upper edge of the tab plate (away from the flange plate) at the mid-width of the model. The gap decreases linearly to zero at the weld attaching the tab plate to the flange plate. Across the 30 in. width of the model, the gap is assumed to have a cosine wave shape, decreasing to zero at the left and right edges of the model.
- In Case 2, the 0.125 in. maximum gap is at the lower edge of the tab plate (where it is welded to the flange plate) at the mid-width of the model. The gap decreases linearly to zero at the upper edge of the tab plate. Across the 30 in. width of the model, the gap is assumed to have a cosine wave shape, decreasing to zero at the left and right edges of the model.

The results of the analysis are shown for Case 2 in Figures 3, 4, and 5. Figure 3 shows the shape of the tab plate (the extent of gap) before and after application of the bolt forces for Case 2. The 0.125 in. initial gap at the bottom of the contact surface reduce to 0.026 in. over a limited area and closes completely elsewhere after application of the bolt forces.

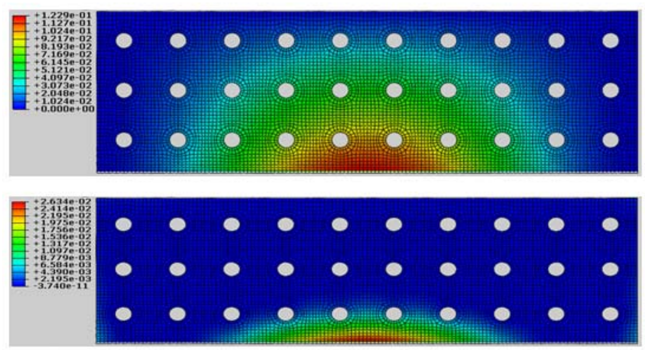

Figure 3 – Contour of gap width before (top) and after (bottom) torqueing bolts, for Case 2.

The contact pressures between the faying surfaces are shown in Figure 4 for Case 2. Note that the contact pressure is non-uniform throughout the contact surface in both cases, with larger pressures distributed closer to the bolts. The sum of pressure over the faying surface of the tab plate for both cases show that the entire bolt clamping force is transferred across the plate faying surfaces, with no part of it going into the 2.25 in. thick flange plate through transverse shear at the base of the tab plate. Note that the mean force developed by the bolt is 13% larger than the design value; therefore, we conclude that the presence of a 0.125 in. gap at localized areas of the faying surface does not compromise the frictional capacity of the connection.

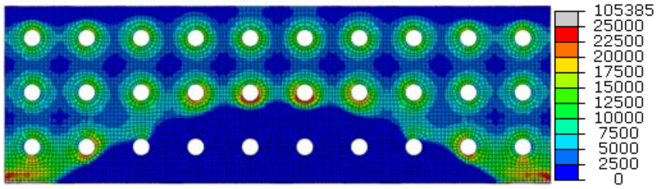

Figure 4 – Contour of the contact pressure at the faying surfaces for Case 2.

The tab plate bends as the bolts are tightened in the presence of a gap between the tab plate and side plate. The resulting plastic strains in the vertical direction are shown in Figure 5 for Case 2. The results show that, in Case 1, there is no yielding on the contact (faying) surface of the tab plate. On the inside surface of the tab plate, the maximum plastic strain is only 0.49% (beyond elastic strain of 0.17%), has limited penetration of 0.125 in. through the depth of the plate, and is located at the bottom edge of the tab plate. In Case 2, the maximum plastic strain in the tab plate is over a limited area located around bolt holes, and has magnitude of 0.71% (beyond the elastic strain of 0.17%), with limited penetration through the thickness of the tab plate. In both cases the plastic strains are small and does not compromise the long-term performance of the connection.

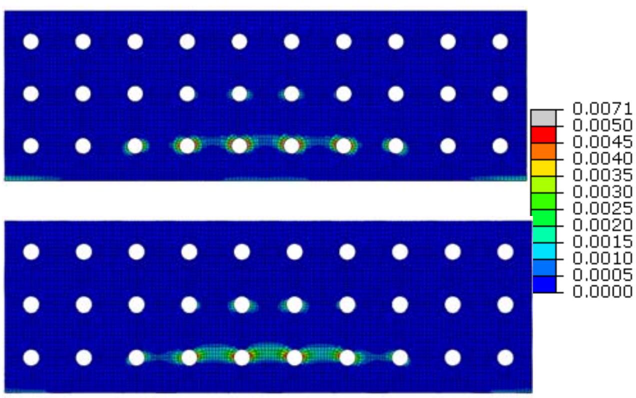

Figure 5 – Plastic strain in the tab plate for Case 2 on the contact surface (top) and on the inside surface (bottom).

In this example, the presence of isolated gaps along the faying surfaces at the tab plate to side plate connections of box girders that are consistent with the ASTM A6 tolerances, are unavoidable. As shown, such gaps do not affect the slip resistance of the connection noticeably.

In rejecting the assemblies, the owner representatives misinterpreted the requirements of governing AWS standards and imposed unrealistic tolerances on the fabricator, which failed to fully and comprehend the influence of mill tolerances on the assemblies and was simply not able feasibly fabricate the members to the imposed tolerances using standard bridge steel practices. Without any focus on the inherent dimensional properties of the steel material specified, structural concerns arose about the gap and its role in reducing the ultimate strength of the assemblies and the slip resistance of the connection, which was based on an argument that a gap between the faying surfaces results in a portion of the bolt tension to be used to bend the tab plate rather than resulting in clamping force in the vicinity of the bolts and was supported by unrealistic and simplistic analysis that did not capture the true three-dimensional performance of the connection. The analysis made proved that the resulting gaps were unavoidable with the steel and fabrication methods prescribed. A three-dimensional finite element analyses performed show that applying bolt torque to a typical tab plate to side plate connection exhibiting a waviness corresponding to a gap of 1/8 in. and conforming to ASTM A6 does not result in excessive strain in the tab plate, or more than a negligible decease in clamping force.

From the practical viewpoint, insufficient notification within the contract documents alerting the contractor and fabricator that required tolerances may exceed allowable ASTM A6 mill tolerances (and/or allowable AWS welding distortion tolerances) and the failure to recognize the cumulative effect of those allowable deviations on the overall members can have devastating consequences on the project.

In the real world of steel bridge fabrication and erection, the subtlety of fabrication tolerances exceeding ASTM A6 mill tolerances doesn't typically surface in discussions between highly trained structural engineers who should know the ins and outs of governing codes and standards. Instead, it generally surfaces in often heated disputes between non-engineer shop inspectors representing the owner and the fabrication shop production personnel who are also typically not engineers. During fabrication, or more damagingly in the inspection of completed fabricated structural steel, interim measurements or final surveys sometimes reveal that actual geometric measurements don't conform to specified tolerances. In those problematic instances, the fabrication shop may have cut steel components using standard or best practice technology, used appropriate practices to provide a dimensionally true lay down and assembly workstations to establish control of components and members that have been carefully positioned, anchored and tacked in position for final welding, then properly sequenced and applied full penetration welds or filet welds with care to hold the weld size and distortion to a minimum. As in the examples above, even with the employment of best practices fabrication techniques and full compliance with AWS fabrication and welding practices, fabricators still may not be able to achieve required tolerances that are more restrictive than ASTM A6 mill tolerances.

Most steel fabrication tolerance disputes such as described in the examples above, more likely than not begin with non-engineer, owner inspectors rejecting members or components based upon their own interpretation of the acceptability of specified tolerance requirements, or in the case of the example cited above, based upon their own interpretation of applicable design codes and fabrication standards. Without fully analyzing the root cause of the fabrication tolerance failure, by careful consideration of the ASTM A6 mill tolerances, the disputes often escalate by more senior owner project personnel being too quick to ratify the inspector's position of rejection. Once ratification of the inspector's interpretation and rejection is committed to writing by the owner, positions harden and an inalterable dispute path develops. Adding fuel to this dispute development dynamic, often non-engineer fabrication shop personnel simply don't understand the subtle limiting effects that allowable mill tolerances, perhaps in combination with welding heat distortion, may have on their ability or inability to achieve specified tolerances. A predictable fabrication tolerance dispute path is that owner personnel tend to take the position that the fabricator and contractor submitted a bid based upon a stated set of tolerance requirements and that the owner should not have to settle for anything less than what was specified. The dispute fermentation evolution then likely ropes in the general contractor, which in all likelihood does not understand the intricacies of milled steel properties or the steel fabrication process, which then demands that the fabricator, no matter the cost, comply with owner tolerance requirements. At that point, all too often, neither the fabricator, general contractor or owner comprehend that, utilizing the ASTM A6 steel material specified, consistent achievement of tolerances more restrictive than allowable mill tolerances, is either impossible to achieve under standard fabrication practice or commercially impracticable or unfeasible. The common results are that fabrication falls behind, follow-on erection activities are delayed, fabricator and general contractor costs increase beyond anything estimated or budgeted, and all contracting parties assume combative claims assertion and claims defense postures. When that happens the opportunity for timely and meaningful, collaborative problem solving often evaporates.

In these circumstances all project stakeholders suffer. Owner project personnel are criticized by the trafficking public and held accountable internally for slipped opening dates. The general contractor incurs substantial unforeseen delay-related and/or acceleration costs, in many instances on top of liquidated damages. The fabricator faces increased labor costs from unproductive idle periods and forced supplementation in the attempt to remedy the "defective work" and accelerate fabrication completion. More often than not, the fabricator has little practical choice but to keep on working in the effort to remedy this "defective" work and perform work on the balance of the project fabrication in a highly inefficient manner. The planned benefit of the fabricator's expected learning curve typically gained over the life of the project may be completely lost.

After the fabricator has incurred tremendous costs attempting to "fix" the problem, delay costs are incurred by both the fabricator, erector and general contractor, all too often the owner is ultimately forced to relax its impossible or impracticable-to-achieve tolerance requirements, which in and of itself may be evidence of a defective specification. From that point, claims arise and highly expensive and time-consuming litigation ensues. As in the example cited above, the relaxation of fabrication tolerances may not adversely affect the design strength and structural integrity of the bridge structure is not compromised. The results are damaged relationships, damaged reputations, extensive cost overruns, lost profit, and project delays frustrating the taxpayer traveling public, and damage to the reputation of the construction industry as a whole.

In the end, there are seemingly two apparent solutions that are worthy of consideration. The first is for project designer to focus more on the constructability of the specified steel tolerances in relationship to the physical properties of the materials required. In so doing the designer must evaluate the specified dimensional tolerance requirements, in a cumulative sense, considering ASTM A6 and the AWS D1.5 code. Then, if any specified tolerances appear questionably restrictive, those “special tolerance” requirements should be prominently noted as such in the plans and specifications. Special fabrication tolerance requirements should be discussed with prospective contractors and fabricators prior to bid and not, for the first time, in pre-fabrication or preconstruction conferences or worse after rejection of fabricated steel. While contractors and fabricators are expected to perform reasonable pre-bid study of all design requirements, in the generally short time allowed during the bidding process, they cannot be reasonably expected to, and they are not legally required to, confirm the constructability of the owner’s design unless a special tolerance(s) is set out prominently and clearly in the documents.

Failing that pre-bid analysis, when specified fabrication tolerances are not met, project designers, owners representative, general contractor and fabricators need to immediately ask the question of, whether, with the material specified, the required tolerances can be consistently achieved utilizing standard fabrication practices. Claim and claim defense perspective should be forced aside and all project stakeholders should collaboratively search for solutions, which don’t compromise structural design integrity. This effort may very well involve the incurrence of short-term additional costs, study of underlying mill tolerance and unavoidable welding distortion through problem-solving analysis such as inquiry to industry specialists and performing a closer look through finite element analysis of the affected components and members.