Proposed AASHTO Guidelines for Bottom Flange Limits of Steel Box Girders

Prepared for:

AASHTO Committee on Bridges and Structures
AASHTO T-14

Prepared by:

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July 5, 2019

The information contained in this report was prepared as part of NCHRP Project 20-07, Task 415, National Cooperative Highway Research Program.

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Abstract

This report documents and presents the results of a study of the impact of minimum thickness and maximum slenderness limits for steel box girder bottom flanges. Bridge owners and other stakeholders were surveyed to collect data pertaining to existing practices and potential limits. The published and unpublished literature was reviewed for relevant domestic and international research, guidelines, and current practices to determine the state of knowledge on: (1) minimum thickness and maximum slenderness limits for steel box girder bottom flanges, and (2) issues during fabrication, handling, erection, and in service. The results of this task are summarized in tabular form showing thickness ranges, slenderness limit ranges and reported issues. Limited analytical studies were conducted to quantify appropriate limits to bottom flange thickness and slenderness, including the consideration of secondary stress effects. The results are synthesized into proposed AASHTO guidelines.

Summary

Steel box-section bridge girders are highly efficient in their structural function, particularly in applications that benefit from their large torsional stiffness. They have an aesthetically pleasing uncluttered appearance that reduces the exposed surface needing maintenance. Due to their substantial structural efficiency, designers can be tempted to specify very thin and slender bottom flanges, particularly in positive moment regions where the bottom flange is in tension. However, specifying too thin or too slender of a bottom flange can result in potential problems during fabrication, transportation, erection, and during the service life of the bridge, including inspection and maintenance. Excessively thin flanges can lead to concerns regarding strength and fatigue, especially in regions of stress reversal.

Based on surveys of bridge owners and other stakeholders, review of the published and unpublished literature, and limited analytical studies, this report proposes guidelines for bottom flange thickness and slenderness limits in steel box girders. Regarding the minimum flange plate thickness:

- A limit of $t \geq$ ½ inch is a rational minimum requirement, unless otherwise approved by an Owner. Steel box girder flanges with t less than this value are apt to have increasing sensitivity to welding distortions and out-of-plane deflections under self-weight and small concentrated applied loads. Smaller thicknesses are common, however, for box-section members employed in long-span construction, where the expense associated with handling and control of distortion in thin stiffened plates is justified by the savings in weight. Minimum bottom flange thicknesses of ¾ inch to 1 inch are commonly recommended for tub-girder bridges however.
- Regarding thickness requirements to facilitate fabrication and erection, it is clear from the surveys and from the literature that generally thicker is better, particularly when considering potential welding distortion. However, specific quantification of the effects of a thickness limit on ease of fabrication and/or erection is somewhat elusive due to the many factors involved. A limit of $t \geq$ 5/8 inch was cited by one of the survey respondents to accommodate threaded holes for drain pipes, and to avoid problems with "manutention" (i.e., handling) before assembly. A thickness limit of 5/8 inches is required by Article 6.7.3 of the current AASHTO LRFD Specifications for orthotropic decks. However, the bottom flange of a box section is not subjected to the same types of demands as a bridge deck plate. Multiple survey respondents recommended a thickness of at least ¾ inch, and a preference for $t \geq$ 1 inch, to facilitate the control of welding distortion, and generally to facilitate fabrication and erection. However, it is clear from the surveys that numerous bridges exist in service that have smaller thicknesses, and therefore it does not appear that violating a thickness limit of even ½ inches will in itself lead to performance problems during the life of a box-girder bridge. Thicknesses as small as 9 mm = 0.35 inches with panel width-to-thickness ratios, w/t, close to 90 are common and have not presented any in-service performance problems in longitudinally stiffened flanges in long-span bridge construction, where the expense associated with handling and control of distortion from thin stiffened plates is justified by the savings in weight. Plates this thin require attention to established procedures necessary to control welding distortion; however, for panel w/t values less than or equal to 90, buckling distortion (i.e., oil canning and waviness of the plates) due to welding is expected to be very controllable. It should be noted that if the flange plate is longitudinally stiffened, particularly if two or more longitudinal stiffeners are employed, the overall width-to-thickness of the plate will tend to be significantly larger than 90. In these cases, the plate will tend to be sensitive to buckling distortion due to welding, and it

is likely that clamping and restraining of the plate and other control measures will need to be applied to limit welding distortion.

- It is advisable for the compression and tension flange thicknesses corresponding to the box section principal axis direction with the largest bending moment to be never smaller than the thickness of the member webs. Girders with primary flanges thinner than the webs will tend to have plate bending stresses due to the distortion of the box section under torsional loads that are larger in the flanges than in the webs.

  Regarding the maximum flange plate slenderness:

- An overall width-to-thickness limit of $b/t \leq 90$ is advisable for longitudinally unstiffened flanges. A limit on the panel width-to-thickness of $w/t \leq 90$ is advisable for longitudinally stiffened flanges. Flanges with larger b/t (unstiffened) or w/t (stiffened) values are apt to exhibit noticeable oil canning or waviness due to welding residual stresses with or without additional small applied axial compression.

The above combined thickness, b/t, and w/t limits provide some accommodation against buckling under accidental or unintended compression in bottom flange plates that are designed nominally only for tension. The report demonstrates that moments causing compression in the bottom flange of box girders can easily have magnitudes approaching the self-weight simple-span values. In addition, these combined limits restrict the sagging of bottom flange plates under their self-weight plus a transverse concentrated load of 0.3 kips to values less than $b/300$ or $w/300$, which are values that ASCE 7-16 indicates as being visible.

The following behavioral considerations can be correlated approximately with the following b/t or $w/$t limits:

- For welded box sections with a b/t (without longitudinal stiffening) or w/t (with longitudinal stiffening) of the flange plates greater than 100, the fabricator may need to be particularly cautious not to over-weld (i.e., to avoid providing welds that are larger than required for strength and/or minimum size requirements) and to appropriately restrain the plate during welding. Some noticeable distortion of the flange plate may occur due to placement of typical minimal welding of the flange to the webs and/or the welding of any stiffeners to the flange plate if these limits are exceeded.
- Box flanges with b/t or w/t values larger than about 130 will have difficulty maintaining less than 1/300 out-of-plane deflection under self-weight, or under self-weight with a small concentrated transverse load; therefore, plate out-of-plane deflections due to these nominal loads may be noticeable. Therefore, in the recommended AASHTO LRFD provisions, box flanges subject to tension with b/t values exceeding 130 are required to have longitudinal stiffeners.
- Dynamic excitation of a box-section member flange in plate bending can be a potential issue in boxes with b/t or w/t greater than about 210.
- Box section flanges with b/t greater than about 210 may be susceptible to fatigue damage due to plate breathing under cyclic tension. This is due to the out-of-plane bow in the flanges from initial fabrication imperfections and from self-weight deflection being cyclically straightened out and released, causing bending stresses to be developed in the plate.
- It should be noted that the strength of box section flanges subjected to compression becomes smaller relative to F_y as b/t or w/t becomes relatively large. For welded longitudinally unstiffened box-section flanges, the ultimate strength of the plate is approximately $0.8F_y$ at $b/t = 40$, $0.6F_y$ at $b/t = 60$, and $0.4F_y$ at $b/t = 90$. These considerations are addressed specifically in broader recommended provisions for design of box-section members (White, et al. 2019), which have been balloted and approved for the 9th Edition of the AASHTO LRFD Specifications at the time of the writing of this final report (July 2019).

Regarding flange extensions in box-section members:

- A minimum extension of 1 inch is commonly recommended, but with a fabricator option to increase the extension for welding access.
- A maximum width-to-thickness of 20 is recommended for the tension flange extension beyond the web. This is based on original recommendations by Wolchuk and Mayrbaurl (1980) and is considered to be a reasonable practical maximum limit for a tension flange extension. However, this limit appears to be relatively arbitrary, and engineers are not likely to use values this large. The project team recommends that the b/t of the tension flange extensions be limited to 12.0, which is the current practical upper limit on the projecting flange width for I-girder flanges specified in AASHTO Article 6.10.2.2 and for the top flanges of composite tub-girders in AASHTO Article 6.11.2.2.
- A smaller limit equal to the compactness requirement for a flange plate supported only on one longitudinal edge is recommended for flange extensions on compression flanges in box-section flexural members. This ensures that F_y can be developed on the full width of the flange extension in compressive strength calculations. If the flange extension is larger than this value, the compressive strength calculations may be performed neglecting the width of the plate wider than the compact limit. This is a simple rule that avoids the need for complex consideration of effective widths due to local buckling, etc. on flange extensions.

Regarding the overall slenderness of longitudinal stiffeners placed in box-section member tension flanges:

- It is recommended that flange longitudinal stiffeners should satisfy $a/r_s \leq 120$ when the overall b/t of the flange is greater than 90, where a is the longitudinal spacing between transverse stiffeners or diaphragms that provide transverse lateral restraint to the longitudinally stiffened plate and r_s is the radius of gyration of the longitudinal stiffener strut about an axis parallel to the plane of the stiffened plate. This limit ensures against excessive out-of-plane deflection of a longitudinally stiffened plate under the self-weight of the plate plus a small concentrated load.

Finally, for transverse stiffeners:

- A simple minimum requirement (where other Specification requirements do not govern) is that the transverse stiffeners should have a moment of inertia greater than or equal to that of the longitudinal stiffeners (with each moment of inertia including a specified portion of the stiffened plate). This is judged sufficient to ensure against excessive out-of-plane deflection of a stiffened plate under its self-weight plus a small concentrated load for most designs. Commentary language is recommended, referencing this report for further discussion, for specialized cases where the flange width is close to or greater than the largest longitudinal spacing to the adjacent transverse stiffening elements at a given transverse stiffener.

CHAPTER 1

BACKGROUND

1.1 Problem Statement

Steel box girders are highly efficient in their structural function, particularly in applications that benefit from their large torsional stiffness. They have an aesthetically pleasing uncluttered appearance that reduces the exposed surface needing maintenance.

Due to the substantial structural efficiency of steel box sections, designers can be tempted to specify very thin and slender bottom flanges, particularly in positive moment regions where the bottom flange is in tension. However, specifying too thin or too slender of a bottom flange can result in potential problems during fabrication, transportation, erection, or future inspection and maintenance. In addition, excessively thin flanges can lead to concerns regarding strength and fatigue, especially in regions of stress reversal.

Various references pertaining to box girder design provide recommended minimum thickness and maximum slenderness limits for steel box-girder bottom flanges. Industry and owner recommendations on minimum bottom flange thickness range from ½ to 1 inch. Recommendations for bottom flange maximum (not-to-exceed) width-to-thickness (b/t) ratios range from 80 to 120. A prominent design example in the classical reference Designers Guide to Box Girder Bridges (Heins and Hall 1981), pertaining to the estimation of distortional warping and distortional transverse plate bending stresses in box girders, has a bottom flange b/t ratio of 142 without any longitudinal stiffening. Recent flange maximum not-to-exceed limits balloted and approved in 2017 by the AASHTO Committee on Bridges and Structures for design of longitudinally unstiffened noncomposite box-section members are $b/t \leq 90$. These recommendations have been expanded to address general noncomposite box-section members with longitudinally stiffened plate elements in AASHTO LRFD Specification provisions balloted and approved in June 2019. In these provisions, a more general not-to-exceed limit of $w/t \leq 90$ has been specified, where w is the width of the plate between the stiffening elements. The present 8th Edition AASHTO LRFD Specifications (2017) do not specify any limits on the thickness or slenderness (b/t) of composite box-girder bottom flanges.

Specific documentation of quantitative and experiential backing for any of the above limits generally appears to be lacking.

1.2 Project Objective

The objective of this project is to synthesize information from all sources, and additionally to supplement this information with further limited quantitative assessments, to help color the ramifications of different

a. Minimum thickness limits, and

b. Maximum slenderness limits

for steel box-girder bottom flanges. The ultimate objective of the project is recommended AASHTO guidelines for minimum thickness and maximum slenderness limits for these structural elements. Broad criteria to be considered include fabrication, handling and erection, as well as the overall long-term performance of box girders in their final constructed configurations. Recommended guidelines/

specifications for incorporation into the AASHTO LRFD Specifications and/or other pertinent industry documents were developed as project deliverables. These recommended guidelines/specifications have been submitted to the AASHTO Committee on Bridges and Structures, via Technical Committee T-14 Structural Steel Design, and have been balloted and approved for the 9th Edition of the AASHO LRFD Specifications at the time of this writing (July 2019).

1.3 Overview of Project by Tasks

The project was arranged into the following tasks:

- Task 1 – Review literature.
- Task 2 – Survey bridge owners and other stakeholders.
- Task 3 – Execute limited analytical studies.
- Task 4 – Synthesize the existing state of practice and issues and prepare a detailed outline of proposed AASHTO Guidelines.
- Task 5 – Develop draft proposed AASHTO Specifications/Guidelines.
- Task 6 – Revise and refine draft Specifications/Guidelines.
- Task 7 – Present the recommended Specifications/Guidelines to AASHTO Technical Committee T-14.
- Task 8 – Develop and submit the final report.

1.4 Purpose and Organization of This Report

This report provides a synthesis of the existing state-of-practice and issues, and recommended draft proposed AASHTO Guidelines pertaining to the above Tasks 4 and 5. Chapter 2 first explains the Task 2 survey approach, timeline, and specific survey materials. This section then provides a brief summary of the survey results. This chapter is provided before the literature review, since the available literature in this area is relatively sparse, and the final stages of the literature review were driven in part by input received from the surveys. Chapter 3 then discusses the Task 1 literature review. Chapter 4 presents the results from the limited analytical studies aimed at evaluating and supplementing the findings from Tasks 1 and 2. Finally, Chapter 5 synthesizes the existing state of practice and issues and provides specific proposed AASHTO Guidelines.

CHAPTER 2

Survey of Bridge Owners and Other Stakeholders

2.1 Survey Approach, Timeline, and Specific Survey Materials

As anticipated, the supporting background regarding bottom flange limits and the documentation of corresponding issues is relatively sparse. Therefore, the project team focused a significant portion of their work on targeted surveys of bridge owners and other stakeholders (i.e., researchers, design consultants, fabricators, erectors and inspectors having experience with steel box-section members). This was combined with direct discussions with key individuals involved with the development of existing recommended limits and/or individuals having direct knowledge of problems encountered with thin box-girder flanges.

As discussed in the August 31 survey plan, the Task 2 survey consisted of specialized letters of introduction and inquiry, aimed at seeking specific information as well as overall perspectives from key individuals who could potentially assist the project team with valuable inputs. Professor White transmitted a personal letter from himself, Dr. King and Mr. Grubb to each of the individuals listed in Appendix A. A general template was used for writing the letters, to ensure some uniformity of the questions asked. This template was then specialized as the project team saw fit to request specific information from the survey recipients. An example letter with attachments is provided in Appendix B. The letters contained a greeting, a reference to an attached project summary statement, also provided in Appendix B, and a list of several questions including a request for the responder to rank 10 items of investigation/query in terms of their importance from the perspective of the responder. Lastly, a request was made for input on any areas of concern other than the 10 listed. The same list of 10 items and request for ranking of them was transmitted to all of the recipients.

The letters of introduction and inquiry were transmitted via e-mail to the majority of the responders on September 23, 2018, and responses were requested by October 15, 2018. In four of these letters, Professor White contacted the chairs of the following AASHTO/NSBA Steel Bridge Collaboration Task Groups:

- TG2 – Fabrication Specification and AASHTO Technical Committee T-17 on Metals Fabrication,
- TG10 – Erection,
- TG12 – Constructability, and
- TG13 – Analysis,

requesting their input, asking if they would share the survey with their task group members, and asking if they would allow 15 minutes for discussion at their Fall 2018 Task Group meetings on Monday through Wednesday, October 8-10 in Austin, TX. In addition, the Chair for TG11 – Design responded to the letter transmitted to him, offered to share the survey with the TG11 members and invited Professor White to present to this group. Professor White and Mr. Ryan Slein, the GRA on the project, attended the three-days of the Steel Bridge Collaboration meetings in Austin and collected input from a number of meeting attendees.

A significant number of responses were received prior to October 22, 2018. Professor White sent a follow-up e-mail to individuals who had not responded on October 22, resulting in a number of additional responses. In addition, based on inputs received, a number of additional individuals were contacted with specific requests for input up through November 1, 2018. Further follow-ups requesting clarifications and additional inputs were made during November and into December. A number of late responses were returned to the project team in early 2019.

2.2 Survey Results

A total of 63 contacts were made via letters of introduction and inquiry and/or via correspondence initiated via the NSBA Steel Bridge Collaboration Task Groups. These contacts are listed in Appendix A. Fifty responses were received based on these contacts. Of these responses, 25 provided rankings of the listed areas of investigation/query pertaining to bottom flange thickness and b/t limits, and 36 provided specific written input.

2.2.1 Ranked Areas of Survey Investigation/Query

Table 1 shows the 10 areas of investigation/query from the survey in the order of highest to lowest importance, or highest to lowest concern, as ranked by the respondents. The most highly ranked area was given a score of 10, followed by 9, etc. Many of the responders did not provide any ranking to some of the areas. These areas were given a score of zero. The total scores from the 25 rankings were summed and divided by 25 to obtain the average score for each area, listed in Table 1.

Table 1. Specific areas of survey investigation/query, ranked from highest to lowest importance or concern.

Consideration	Average Ranking
1. Welding distortion	7.9
2. Buckling of thin bottom flange plates intended to serve predominantly in tension, due to unanticipated or accidental axial compression in the plates during handling and transportation, erection, and in the final service condition, particularly in the vicinity of inflection points in continuous-span girders or near locations where longitudinal stiffeners are terminated	7.8
3. Localized plate bending stresses induced by handling and erection operations	5.4
4. Localized plate bending stresses due to box-section distortion (i.e., distortion transverse bending stresses) as well as secondary stresses due to out-of-plane deformation of the box-section bottom flange	4.8
5. Potential “oil canning” of thin plates in bottom flanges, i.e., snapping in or out of the plates between edge supporting elements when pushed on by a light force.	4.7
6. Localized out-of-plane deflection due to a concentrated transverse load applied to the bottom flange, or to a plate panel within the bottom flange	4.3
7. Vibration of excessively thin plates due to in-service loadings on the completed bridge	3.4
8. “Bending reluctance” of excessively thin flanges, that is, the tendency for thin flanges to not bend fully along with the overall curvature of the box-section member	2.9
9. Vibration of excessively thin plates during transportation	2.5
10. Perception of vertical vibrations, or general “sponginess”, in bottom flanges of box girders by bridge inspection personnel. These vibrations may be due to transient live loads on the bridge or due to the individual’s movements when walking the inside of the box girder	2.4

Welding distortion and buckling of thin bottom flanges due to unanticipated or accidental compression received similar highest rankings, meaning that these areas were judged to be the most important to consider and/or the areas of highest concern. Consideration of plate bending stresses due to handling and erection operations, and due to box section distortion under torsional loads, potential oil canning of thin bottom flange plates, and localized out-of-plane deflection due to a concentrated transverse load received similar rankings forming a "second priority" group. Vibration of thin plates due to in-service loadings on the completed bridge, bending reluctance of thin flange plates, vibration of thin flange plates during transportation, and perception of flange plate vertical vibrations under in-service conditions received the lowest rankings and comprised the "lowest priority" group.

2.2.2 Other Areas of Concern

The following list summarizes the responses received to the question, "Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?" The complete written responses to the survey are listed in Appendix C:

1. "Web gap fatigue damage." This issue was mentioned by Respondents 2, 7, 18, and 34. However, none of the respondents suggested more than an indirect correlation between the bottom flange thickness or slenderness and web gap fatigue issues, and there does not appear to be any direct correlation with the thickness or slenderness of the girder bottom flange and web gap fatigue in the cases cited. The fatigue cracks were generally near the connection of the internal cross-frame members and the end of the connection plate at the web gap. Provision of a thicker bottom flange plate may even exacerbate the problem, since this provides more restraint from the flange and may further localize the strains within the web gap.
2. "The use of longitudinal and/or transverse stiffeners should be minimized," highlighted by Respondents 15 and 21. Respondent 21 emphasized that in many situations, it is more economical to increase the bottom flange thickness, reducing short-term costs associated with the complexity of the fabrication and long-term costs associated with better fatigue performance, while also providing a lesser potential for corrosion problems, and easier/safer access by inspectors.
3. "Make sure the b/t limits for stiffened flanges are addressed well," highlighted by Respondent 26. This consideration is addressed directly, and more broadly by the panel w/t and longitudinal stiffener a/r_s limits recommended later in Section 4.8, where a is the spacing between the transverse stiffening elements, i.e., diaphragms and/or transverse stiffeners, and r_s is the radius of gyration of the stiffener struts. Requirements for the transverse stiffeners are also addressed subsequently in Section 4.9.
4. "General ease of fabrication." This consideration was discussed by Respondents 1, 3, 10, 12, 13, 21, 25, 28 and 36. Item 2 above also relates to ease of fabrication. In addition, the following aspects were cited by the Respondents:
    a. Respondent 21 indicated that, when the bottom flange has welded longitudinal stiffeners, depending on the width of the bottom flange, the flange thickness should not be less than ¾ to 1 inches because of the deformations generated by the welding of the stiffeners. It was pointed out that these deformations can cause problems of flatness at mechanical joints. This issue can be related to Items 1 and 5 in Table 1.
    b. Respondent 21 also pointed out that if threaded holes are used for drain pipes, a minimum thickness of about 5/8 inches should be employed.
    c. Furthermore, this respondent referred to "manutention" of plates before assembly, i.e., difficulty of controlling local deformations, e.g., at lifting points, and suggested that plates less than 5/8 inches in thickness may be a problem with respect to corresponding local distortions or bowing during the

handling of the plates. Respondent 28 indicated that they have encountered difficulties of this nature with ¾-inch and 1-inch thick plates. It would appear that, generally, thicker is better with regard to this issue. This issue relates to Item 6 of Table 1.

d. Respondent 21 also stated that a *b*/*t* limit of 90 seems reasonable, but a limit on the thickness of 5/8 inches as commonly specified for orthotropic steel decks, also may be appropriate. Respondent 27 used a minimum thickness of 5/8 inches in their prior tub girder designs discussed in response to the survey. Furthermore, Respondent 21 stated that CJP welding of thin plates less than ½ inches in thickness may be more at risk for local deformation.

e. Respondent 25 recommended a minimum bottom flange thickness of ¾ inches to limit the local dishing in the flange when it is subjected to a concentrated jacking force to attach the flange to the cut-curved webs. This can be related to some extent to Item 6 of Table 1. In this regard, it should be noted that Respondent 35 pointed out that minimum thicknesses of 9 mm (slightly less than 3/8 inch) have been employed along with maximum panel *w*/*t* values of 72 in long-span bridge designs such as the Humber, Storebaelt and Canakkale bridges.

f. Respondent 15 pointed out that their comments related to successes of the Texas Steel Quality Council (2015) guidelines are in the context of typical highway bridges using steel tub girders, and that they are not applicable to special bridges with very wide tubs or boxes. It is apparent that engineers should be cautious about imposing simple blanket limits on all types of bridge steel box-section members, and that the various simplified limits being considered by NCHRP 20-07/415 are often associated with somewhat "grey" boundaries in terms of their ability to ensure targeted acceptable behavior.

g. Respondent 36 emphasized that, to control distortions from welding of tub girders, a minimum bottom flange thickness of one inch is preferred. However, his experience is that ¾ inch thickness is generally workable. At ¾ inch thickness, this respondent expected corrections may be needed to satisfy flatness requirements, particularly at bearings. They cautioned that this effort goes up considerably when thinner flanges are used. Again, it should be noted that Respondent 35 pointed out that minimum thicknesses of 9 mm (slightly less than 3/8 inch) have been employed along with maximum panel *w*/*t* values of 72 in long-span bridge designs such as the Humber, Storebaelt and Canakkale bridges. It is apparent that smaller thicknesses are common in long-span bridge construction, where the expense associated with handling and control of distortion in thin stiffened plates is justified by the savings in weight.

5. "Collection of water inside of a box girder due to inadequate drainage and/or ponding, including potential implications with respect to corrosion." This issue was mentioned by Respondents 6, 7, 12, and 21, and can be related to Items 1 and 6 of Table 1. Respondent 12 reported significant ponding issues on a box-girder bottom flange with a thickness of 1 inch. It is uncertain whether this was an actual ponding problem, involving amplification of the out-of-plane plate deflections due to the accumulation of additional water within the plate as it deflected under the weight of the water. If it was, it is expected that it would not be economical to increase the plate thickness further to alleviate the issue. Respondent 12 emphasized that if the plate had been thinner, the section loss due to the concomitant corrosion would have been a greater concern.

6. "Ability to make heat straightening or other repairs in the event of collision damage." Respondents 8 and 12 mentioned this consideration. It is unclear how the bottom flange thickness or slenderness would affect this consideration, although there was an implication that box girders with a thicker flange may fair better with respect to collision damage.

7. "Broad consideration of resilience," which was emphasized by Respondent 12. This is somewhat related both to Items 5 and 6 of Table 1 (resilience to general section loss and to the expected lesser

sensitivity to damage from collision). Respondent 12 also mentioned potential resilience to fire damage; however, the authors of this report anticipate that thicker flanges may not have any significantly greater resilience to fire damage. Thinner plates most definitely would be more sensitive to section loss due to corrosion compared to thicker plates. Respondent 23 highlighted this consideration.

8. "Lateral compression capacity of the bottom flange plate in relation to the compression loads caused by shear in inclined webs of tub girders (i.e., a horizontal component of force applied to the flange due to the web shear)," highlighted by Respondents 20 and 26. This can be related somewhat to Item 2 of Table 1, although Item 2 likely would be interpreted by most as a consideration related to longitudinal compression.

9. "Limits on the bottom flange dimensions based on torsional shear," recommended by Respondent 26. It is expected that satisfaction of demands based on other loading considerations, such as Items 1 through 5 of Table 1, will be sufficient to satisfy these concerns. This consideration is addressed further in Section 4.2 of this report.

10. "Checking that the transverse deflection of a box flange due to self-weight plus a 500 lb concentrated load is less than $L/360$, where L would typically be taken as the plate width." This simple rule, suggested by Respondent 11, directly relates to Item 6 of Table 1 and is an interesting potential way of marrying a demand calculation with a criterion for acceptability borrowed from traditional building floor system design (ICC 2017, Section 1604). Furthermore, it can be indirectly related to Items 3 and 5 of Table 1. This potential criterion is evaluated in detail in Section 4.5 of this report.

11. "Aesthetics and the visibility of distortions and deflections," cited by Respondents 9 and 20. This issue is often referred to as "oil canning" within the broader literature related to metal roofing and metal wall panels in building construction (Metal Construction Association 2013). This issue can be related to Items 1, 2, 5 and 6 of Table 1. It is generally accepted that the thicker the plate, the less likely it is to "oil can." In addition, it is well known that oil canning is commonly more visible on higher gloss surfaces. However, no clear correlation is available between plate thickness or b/t and acceptable lack of visibility of distortions and deflections.

12. "Panning of web and flange plates, due to deformations from weld shrinkage," highlighted by Respondent 19. The authors of this report interpret that the "panning" phenomenon is very closely related to the term "oil canning" mentioned in Item 5 of Table 1, although "oil canning" can also be due to other reasons, e.g., thermal strains, as mentioned by Respondent 23. It is also of course related to Item 1 of Table 1.

13. "Cupping of the flange plates from the web-to-flange welds," highlighted by Respondent 25. This is interpreted to be the same phenomenon as the above term "panning." It also may be referred to as warping, curling or dishing of the plates due to weld shrinkage. As stated above, this phenomenon is believed to be closely related to the term "oil canning" mentioned in Item 5 of Table 1, as well as generally Item 1. Respondent 25 indicated that ¾ inch flanges are preferred for both stiffened and unstiffened flanges to alleviate cupping deformations. This is consistent with Item 4 above from Respondent 21, although Respondent 21 indicated that a 5/8 inch minimum thickness limit may be acceptable in this regard. This is also consistent with the responses from Respondent 36.

14. "Limit the fillet weld size on thin flanges," highlighted by Respondent 25. Respondent 25 also suggested avoiding full penetration welds at all cost. Larger welds cause larger welding distortion; this is obviously related to Item 1 of Table 1.

15. "Satisfy tolerances on dimensional parameters to avoid excessive stress reversals due to a variety of loading conditions, which may result in potential fatigue concerns," from Respondent 3. This relates to Items 2, 4, 6, 7, 8, and 10 of Table 1.

16. "Allowable out-of-flatness fabrication tolerances with respect to ASTM A6, and lack of criteria in AWS D1.5 for fabricated tub girders," recommended by Respondent 11. Respondent 11 also suggested that the recommendations by Asadnia (2018) regarding bottom flange out-of-flatness criteria should be considered. These recommendations involve the following aspects:
    a. The studies by Asadnia seek to build on the prior research by Zhang (2007) and seek to quantify an appropriate out-of-flatness tolerance for tub-girder bottom flanges based on finite element full nonlinear analysis simulations. These simulations target the reductions in the moment at the onset of first yielding, obtained relative to the first-yield moment, considering ideal geometries with very small out-of-flatness of the plates.
    b. Asadnia (2018) references Zhang (2007) extensively for the background to his studies, and proposes the use of the first buckling mode deformations in both the web and the flanges, in combination, as the geometric imperfection pattern scaled to identify the sensitivity of the onset of first yielding to imperfections. In contrast, Zhang (2007) focused on using the first buckling mode, but applied solely to the flange plate or web plate(s) under consideration, when calculating the sensitivity of the onset of first yielding to imperfections. The goal of both researchers was to identify a function for the amplitude of the plate local buckling geometric imperfections that leads to the same percentage reduction in the moment at the onset of first yielding for different values of the plate slenderness.
    c. While identifying rigorous strength-based plate out-of-flatness tolerances for plates subjected to compressive (and shear) loads, and their appropriate measurement, are important state-of-the-art topics, this is somewhat outside the scope of the NCHRP 20-07/415 research. As noted by Respondent 23, one should not mix the issue of a bottom flange loaded predominantly in tension with compression.
    d. Zhang (2007) clearly demonstrates that the actual ultimate strength of I- and box-girders is less sensitive to the amplitude of plate out-of-flatness than the moment at the onset of first yielding. It would appear that the sensitivity of the actual member ultimate strength would be a more appropriate rigorous strength-based criterion for the development of out-of-flatness tolerances.
    e. Regarding box-girder flanges subjected to uniaxial compression, Asadnia (2018) extends the research by Zhang (2007) by analyzing a more comprehensive range of box-girder geometries, focused on the sensitivity of the onset of first yielding in compression to the first-buckling mode imperfection amplitude and the plate slenderness. Both researchers conclude that a constant value of $b/200$ is the most appropriate out-of-flatness tolerance for box-section bottom flanges, both from the viewpoint of impact on the onset of first yielding as well as "average fabrication ability of today's industry" (Zhang 2007). (It should be noted that fabrication effort is considered a more appropriate term in the context of this study, rather than fabrication ability.)
    f. As indicated by Respondent 24, the ultimate local buckling resistance of thin plates can have some sensitivity to initial imperfections; therefore, it is appropriate for the out-of-flatness tolerance to be well defined. It should be noted that plate out-of-flatness imperfections having an amplitude of $b/200$ were considered in the studies documented in White et al. (2019) and Lokhande and White (2018).
17. "Breathing of plates under independent or combined cyclic compression and shear," highlighted by Respondent 34. This can be related to Item 2 of Table 1. Respondent 34 provided two references that address these considerations, Roberts and Davies (2002) and Skaloud and Zörnerovà (2005). The response by Professor White to Respondent 34, listed in Appendix C, discusses this consideration, and the manner in which it is addressed in some depth in current AASHTO (2017) and proposed LRFD provisions (White et al. 2019).

18. "Breathing of an excessively thin plate under self-weight and cyclic tension and consequential fatigue concerns." This issue was pointed out by Respondent 5 in the context of a box-girder bridge having a bottom flange b/t ratio "between 260 and 280" in its positive moment region, where the longitudinal stiffeners were curtailed since the flange was in tension under all load combinations. Clearly, this is an excessive width-to-thickness ratio. It is well beyond what may be expected to be acceptable. However, it is of interest to note that the bridge has been in service since the late 1980's. Some fatigue cracking has been observed near the panel boundary with a transverse stiffener.

19. "Special case of a box designed as two I-girders, but then the bottom flange is a single plate to give the appearance of a box." Respondent 22 highlighted this consideration. It would be expected that although the plate is intended to be non-structural in this case, this could lead to potential fatigue concerns, or at least aesthetic concerns (particularly concern about visible plate breathing from the traveling public), if the b/t of the plate is excessive such as in the above Item 18. This relates also to all of the items of Table 1. The comments regarding aesthetics in the above Item 11 should also be considered for this type of detail.

20. "Handling of boxes with an unbraced open end," mentioned by Respondent 23. The project team views this issue predominantly as a handling consideration during fabrication, although there may be some instances where it may be a consideration during erection. As pointed out by Respondent 23, the solution in this case is to provide proper bracing of the open end, not to increase the flange thickness or decrease the flange slenderness to address this issue. This consideration can be related to Items 3 and 6 of Table 1.

Clearly, there are numerous considerations associated with the definition of a basic not-to-exceed or not-to-be lesser than limits on the dimensions of box-girder bottom flanges loaded predominantly in tension. A number of owner-specified box-girder bottom flange limits were identified via the survey responses. In addition, a number of owners provided summaries of box-girder bottom flange proportions within their bridge inventory, or they provided information about specific box-girder bridges that had exhibited certain concerns. This information provided by the respondents is summarized as part of the literature survey in Chapter 3.

CHAPTER 3

Literature Review

Task 1 of the project focused on the completion of a literature review of relevant domestic and international research, guidelines, and current practices to determine the current state of knowledge on (1) minimum thickness and maximum slenderness limits for steel box girder bottom flanges, and (2) issues during fabrication, erection, and in service. This information has been collected from published and unpublished reports, guideline documents, Specifications, and technical papers.

3.1 Box-Girder Bottom Flange Not-to-Exceed or Not-to-be-Lesser-Than Dimensional Limits Recommended in the Literature

Table 2 summarizes key findings from the literature review regarding recommended thickness limits, slenderness limits, and applicable background. In addition, the table provides information regarding how the different entities have reached their recommended thickness and slenderness limits.

Table 2. Recommended box girder bottom flange limits from review of the literature.

Limit(s)	Source	Background
$t \geq$ ½ inch	FDOT (2018, 5-5)	Standard guidelines/rules of practice.
$t \geq$ ¾ inch, 1 inch preferred, and b/t or $w/t \leq 80$	TSQC (2015, 2-17)	Aid to steer designers toward fabricator- and erector-friendly bridges, based on TxDOT experiences with several steel tub girders with flanges (¾ inch and less) in which plate distortion/oil canning was observed and workers walking inside the girder caused local deflection. TxDOT received fabricator complaints about oil canning and buckling of the bottom flange during handling and storage in the shop.
$t \geq$ ¾ inch and $b/t \leq 120$ for longitudinally unstiffened flanges; thicknesses smaller than ¾ inch might be considered for longitudinally stiffened flanges, after consultation with fabricators	Grubb et al. (2010, 3.1.117)	For ease of handling and to minimize distortion and possible cupping of the flange during welding.

Table 2 (continued). Recommended box girder bottom flange limits from review of the literature.

Limit(s)	Source	Background
b/*t* preferably should not exceed 60 for compression flanges, except in regions of low stress near points of dead load contraflexure [and implied, simply supported girder ends]; should *b*/*t* exceed 45 for compression flanges, longitudinal stiffeners should be considered.	AASHTO (2002, 272)	Guidelines to achieve efficient strength design of structural steel plates subjected to compression, in terms of the magnitude of the design stresses that can be developed.
b/*t* preferably should not exceed 60 for compression flanges; should *b*/*t* exceed 45 for compression flanges, longitudinal stiffeners should be considered.	AASHTO (1993, 1.28)	Guidelines to achieve efficient strength design of structural steel plates subjected to compression, in terms of the magnitude of the design stresses that can be developed.
The *b*/*t* of longitudinally unstiffened tension flanges, or *w*/*t* of longitudinally stiffened tension flanges, shall not exceed 120; The width-to-thickness ratio of tension flange projections on the outside of the webs shall not exceed 20; The slenderness ratio, a/r_s, of the longitudinal stiffeners of a tension flange shall not exceed 120.	Wolchuk and Mayrbaurl (1980)	These are "arbitrary" limits intended to provide the minimum rigidity necessary to overcome the "bending reluctance" of a wide flange, that is, the tendency of the flange to avoid the overall curvature of the box girder, adopting instead a greater radius away from the web. They are also considered as necessary to "check the dynamic excitability" of the flange.
When longitudinal stiffeners are used, it is preferable to have at least one transverse stiffener placed near the point of dead load contraflexure. The stiffener should have a size equal to that of the longitudinal stiffener	AASHTO (2002, 272)	To provide transverse restraint of the longitudinal stiffeners in the vicinity of the dead load contraflexure point.

3.2 Representative Bottom Flange Dimensional Values Ascertained from Survey Responses and from Review of the Literature

Table 3 summarizes the range of bottom flange thicknesses, *b*/*t* and *w*/*t* values reported by the respondents to the survey discussed in Chapter 2, as well as values from a survey of box-girder bridge designs presented by Heins (1978).

Table 3. Summary of bridge box girder bottom flange thickness, *b*/*t* and *w*/*t* values reported by respondents to the survey and by Heins (1978).

Number of Bridges	*b* (in)	*t* (in)	*b*/*t*	*w*/*t*	Flange Classification	Notes
6	68 to 122	5/8 to 1.25	68 to 177	NA	Unstiffened	No problems other than web gap cracking
		11/16 to 1.625	42 to 177	10 to 59	Stiffened	
Not specified	Not specified	Minimum 3/4	Maximum 84	Not specified	Not specified	No records of fabrication or in-service problems
1	83 to 91	Approx. 5/16	260 to 280	NA	Unstiffened, M^+ region	Fatigue cracking due to breathing of flange under cyclic tension
1	Not specified	Min 3/8	204 to 708	51 to 177	Stiffened	Wide multi-celled section with longitudinally and transversely stiffened bottom flanges and three webs; some web gap fatigue cracking
1	111	½ to 1.75	63 to 222	NA	Unstiffened	Difficulties with plate distortion & oil canning during fabrication, visible deflections from workers walking inside the box
28	60 to 81	½ to 2.25	27 to 162	NA	Unstiffened	No record of any performance issues except for cracking at CF connection plate to web welds in one bridge
		½ to 2.00	30 to 122	15 to 61	Stiffened	
32	42 to 112	5/8 to 1.75	36 to 152	NA	Unstiffened, M^+ regions	Bridges seem to be performing well; No bottom flange “sponginess” noticed when walking through boxes
	42 to 92	5/8 to 2.00	35 to 105	33 to 76	M^- regions, Stiffened & Unstiffened	
Not specified	84	¾ to 2.0	42 to 112	NA	Unstiffened	Web gap fatigue cracking encountered in some girders
6	50 to 72	½ to 2.625	21 to 120	Not specified	M^+ regions	Heins (1978) survey, simple-span bridges; unspecified whether any of the flanges were longitudinally stiffened
12	56 to 114	5/16 to 2.625	21 to 247	Not specified	M^+ regions	Heins (1978) survey, two-span continuous bridges; unspecified whether any of the flanges were longitudinally stiffened
		½ to 3.125	18 to 173	Not specified	M^- regions	
10	48 to 114	3/8 to 3.125	18 to 228	Not specified	M^+ regions	Heins (1978) survey, three-span continuous bridges; unspecified whether any of the flanges were longitudinally stiffened
		9/16 to 3.125	18 to 130	Not specified	M^- regions	

3.3 Discussion

In summary, the only documented issues identified within the surveys and the literature review that can be directly correlated with bottom flange thickness or *b*/*t* limits were:

1. Fatigue cracking at the boundary with the transverse stiffener in the third case listed in Table 3, and in Item 18 of Section 2.2.2, where the flange thickness was approximately 5/16 inch and the flange *b*/*t* was between 260 and 280. As noted in Item 18 of Section 2.2.2, the *b*/*t* ratio in this bridge is clearly excessive, resulting in fatigue problems due to "breathing" of the bottom flange plate under cyclic tension stresses from the overall flexure of the box girder acting on the initial out-of-flatness imperfections plus the bowing of the flange under its self-weight. However, one can observe that this slenderness is not significantly larger than several of the other maximum slenderness values indicated in Table 3.
2. Issues associated with difficulty of fabrication and/or with noticeable flange out-of-flatness during fabrication. Several of the guidelines listed in Table 2 are clearly directed at addressing these issues.

Three entries in Table 3 indicate observations of web gap fatigue cracking. However, although one might expect some potential interrelationship between web gap fatigue cracking and bottom flange slenderness, the authors of this report assert that the web gap fatigue cracking problem and the problem of fatigue cracking due to distortion of an excessively slender box flange are distinctly separate phenomena. A number of the above cases involve fatigue cracking near the panel boundary with a transverse stiffener without the existence of a web gap. As noted in Item 1 of Section 2.2.2, there does not appear to be any direct correlation with the thickness or slenderness of the girder bottom flange and web gap fatigue in the cases cited.

Regarding recommendations in international design guidelines and standards, no recommended limits on bottom flange thickness or *b*/*t* were found outside of those listed in Table 2. Respondent 5 confirmed this as a fact in their comment, "No specific guidelines exist in Eurocodes for shape limits of internal tension flanges" (see Appendix C). With the exception of brief discussions provided by Wolchuk and Mayrbaurl (1980), the recommendations in Table 2 appear to be based predominantly on American research and structural engineering practice.

From Table 1, the two considerations relative to steel box-girder bottom flange dimensional limits of greatest concern are:

1. Welding distortion, and
2. Buckling of bottom flange plates.

The following sub-sections provide some detailed discussion of several aspects of the above factors and considerations that have been given notable attention within the literature. The factors tied to the earliest limits specified in Table 2 are addressed first, followed by further discussion of the background to the more recent recommendations listed in this table. This is followed by a broad discussion of welding distortion and buckling considerations.

3.3.1 Dynamic Response and Plate Breathing Response of Box-Girder Bottom Flanges

Wolchuk and Mayrbaurl (1980) specifically state that "Slender and flexible tension flanges which may be subject to dynamic excitation shall possess sufficient rigidity, or be suitably damped, to withstand excitation." In addition, they state that "Tension flanges of multi-box composite girders designed under the provisions of Art. 1.7.203 [i.e., their recommended provisions for bridges of moderate length supported by two or more single-cell composite box girders] shall be deemed to satisfy the dynamic stability requirements." However, there do not appear to be any restrictions on tension flange thickness or *b*/*t* values in this article. Elsewhere, Wolchuk and Mayrbaurl (1980) do specify a maximum value of 120 for the *b*/*t* of longitudinally unstiffened and the *w*/*t* of longitudinally stiffened tension flanges, as well as a slenderness ratio a/r_s limit for longitudinal stiffeners on tension flanges, where a is the spacing between transverse stiffening elements, i.e., transverse stiffeners and/or diaphragms, and r_s is the radius of gyration of the stiffener struts. They indicate that these limits are arbitrary, and that the value of 120 was a proposed limit in a 1975 draft British standard (BSI 1975). However, the project team found no discussion of this consideration after a careful review of various British and European guidelines and standards over the years.

Wolchuk and Mayrbaurl (1980) state that their recommended tension flange slenderness limits are intended to limit the dynamic excitability of the flange. Wolchuk and Mayrbaurl reference an unpublished study by Mattock for a quantitative study of the dynamic excitation of box-girder bottom flanges. The project team was able locate an obscure commentary document, Mattock (1967) that presents this apparent work. In summary, the steps involved with this dynamics study were:

1. Estimation of the natural frequency of a given bridge.
2. Estimation of the natural frequency of the bottom flange plate elements. This calculation is based on an analysis of the composite box-girder bottom flange and webs as a frame, with the webs providing minor rotational restraint to the longitudinal edges of the bottom flange.
3. Estimation of the maximum vertical acceleration of the bridge girders due to traffic loading.
4. Estimation of the plate bending moments due to the imposition of the above vertical acceleration.
5. Conservative estimation of the dynamic amplification of the plate bending moments by assuming a steady-state forcing of the plate elements at a frequency equal to that of the bridge and 2 % viscous damping.

The example worked by Mattock (1967) involved a 50 ft girder span and a 106 inch wide x ½ inch thick bottom flange plate (b/t = 212). For this problem, the bridge natural frequency was calculated as f_b = 5.8 Hz, and the natural frequency of the bottom flange plate was estimated as f_p = 7.1 Hz, which is relatively close to the natural frequency of the bridge. The resulting estimated dynamic amplification was 2.65. The corresponding bottom flange maximum plate bending stresses were located at the mid-width of the flange and were only approximately 3 ksi. Mattock (1967) concluded the example by stating that, for spans greater than 50 ft, lower stresses are expected since the fundamental bridge frequency and consequently the bridge girder vertical accelerations decrease. Furthermore, he stated that the plate elements would usually be stiffer, and therefore, the base plate bending stresses due to the imposition of the vertical acceleration as well as the dynamic amplification factor would be smaller. Wolchuk and Mayrbaurl (1980) refer very briefly to the above attribute for bridge spans larger than 50 ft.

Given that the above example involved a plate with a b/t significantly beyond the "arbitrary" limit of 120 recommended by Wolchuk and Mayrbaurl (1980), it can be interpreted that significantly larger b/t values are possible without dynamic excitation being a problem. Anecdotally, it would appear, very roughly, based on the single field result cited in Table 3 and in Item 18 of Section 2.2.2, that the flange b/t limit certainly should be smaller than 260. Also, given the other bridges reported by the survey respondents and by Heins (1978) plus the example from Mattock (1967), one might, very roughly, surmise that a b/t up to 247 might be possible (at least in typical bridges with spans greater than or equal to 50 ft) without inducing a plate breathing or a dynamic excitation issue.

The area of vibration fatigue in mechanical engineering, and sophisticated finite element tools employed in this area such as FE-SAFE (Simulia 2019) and ANSYS nCode DesignLife (ANSYS 2019), are very applicable to analyze the forced vibration of a box girder bottom flange under the service loads a bridge may see, and to assess the influence of the flange slenderness on fatigue life. With these types of tools, the excitation is characterized within the frequency domain via a Power Spectral Density. In addition, a key part of the solution (which is the same as in the finite element solutions discussed by Murray et al. (2016))

is a modal analysis, which quantifies the natural modes and frequencies of the vibrating component and enables accurate prediction of the local stress responses for a given excitation. Extensive documentation relating to forced vibration and fatigue can be found in references such as Lalanne (2014), Wirsching et al. (1995) and Crandall and Mark (1963).

Unfortunately, the use of these types of tools to evaluate the dynamic excitability and vibration fatigue of a bottom flange was determined to be well beyond the scope of the current project. Furthermore, given that vibration concerns were ranked very low, and given that it would appear that the thickness and/or *b*/*t* limits needed to facilitate the fabrication and erection of steel box-girders are much more restrictive than the above-discussed values, it can be surmised that this type of sophisticated dynamics study is unnecessary. It would only bolster the findings from Mattock (1967) that vibration of the bottom flange plates is not typically a significant concern.

3.3.2 Limits Intended to Facilitate Fabrication and Erection of Box Girders

Substantial guidance regarding the proportioning of box-section flanges is summarized in the NHI Course No. 130095 Reference Manual, Analysis and Design of Skewed and Curved Steel Bridges with LRFD, Section 3.1.5.4.2 (Grubb et al. 2010). This document recommends a minimum thickness limit of ¾ inch for unstiffened box flanges, "for ease of handling and to minimize distortion and possible cupping of the flange during welding," and points to Goldberg and Leve (1957) for a recommended maximum *b*/*t* limit of 120. As part of the NCHRP 20-07/415 literature review, the project team located the Goldberg and Leve (1957) reference. Unfortunately, this reference only discusses the application of folded plate theory to the analysis of structural members. No mention is made of any recommended *b*/*t* limits in the paper [the authors have since recognized that this reference citation was made in error].

This unfortunately leaves the *b*/*t* maximum limit of 120 as indeed being "arbitrary," as stated by Wolchuk and Mayrbaurl (1980).

Grubb et al. (2010) suggests that a smaller thickness might be considered for longitudinally stiffened box flanges, but that fabricators should be consulted before utilizing box flange thicknesses below ¾ inch. In addition, this document provides insightful discussions of the material savings that should be gained to warrant introducing a shop splice to step-down the thickness of a box-section member bottom flange.

In addition, Grubb et al. (2010) discusses box flange requirements to accommodate the placement of a composite concrete deck. Use of a composite lightweight concrete layer on a box-section member bottom flange could be one option to consider as a future innovative option to optimize the economy of bottom flanges in box girders, particularly in regions of high compression. This is permitted for top flanges of closed-box sections subjected to flexure in the current AASHTO LRFD Specifications, but has not been implemented in the U.S. with the exception of the work by Yen et al. (1986) and Sen and Stroh (2010). In addition to top flanges, this was permitted for bottom flanges in Article 10.4.3 of the AASHTO (2003) Guide Specifications for Curved Steel Girder Highway Bridges (LFD). Example calculations for this option in a negative moment region were given on page 358 of that specification. This provides one other option to stiffen, and dampen the vibrations, of box-girder bottom flanges. Addressing the questions associated with this option is beyond the main scope of the NCHRP 20-07/415 project, however.

The Texas Steel Quality Council (2015) document, Preferred Practices for Steel Bridge Design, Fabrication and Erection, provides numerous practical recommendations for all types of steel bridge construction. Regarding the thickness and *b*/*t* limits on bottom tension flanges of tub girders, this document states, "Check with fabricators when using bottom tension flange plates less than 1 inch thick to determine whether practical stiffness needs are met. Bottom tension flanges should never be less than ¾ inches thick," and that "Bottom tension flanges should have a *w*/*t* ratio of 80 or less." Based on the input from Respondents 12 through 15, it is clear that these limits were based on significant concerns pertaining to plate distortion/oil canning as well as noticeable flange out-of-plane deflections caused by workers walking inside of steel box-girders. Input from several survey respondents indicates there was a range of opinions regarding the above thickness limit during the development of this document. Coletti et al (2005) reference a year 2000 version of this document in which the recommended absolute minimum thickness was specified as ½ inch. Sections 4.1, 4.5, and 4.6 of this report endeavor to quantify the implications of different thickness and *b*/*t* limits.

3.3.3 Welding Distortion

Extensive guidance can be found in the literature pertaining to processes and procedures to alleviate welding distortion (Mandal 2004 and 2009; Feng 2005; Michaleris 2011; Radaj, D. 2012; Canadian Metalworking 2014; Gray et al. 2014; CAMM Metals 2017; Scholler and Ringer 2018a and b; KOBELCO 2019; Lincoln Electric 2019; TWI 2019; Welding Answers 2019). Welding distortion is caused by non-uniform heating and cooling in and near the weld, which produces residual stresses in the metal. The form and magnitude of the distortion depends on a number of factors, such as the component geometry, initial fit-up of the parts, weld design, welding process heat input, welding time, welding sequence, and tooling. Welding heats a very specific location, often the edges of the plates, and this causes the material to expand unevenly. The metal is restricted in the amount it can expand by the cooler metal further away from the weld. This causes compression in the expanding metal near the weld. Upon cooling, this compressed metal shrinks, pulling the weld into tension and placing the initially cooler metal into compression. This leaves the welded components distorted. The type and magnitude of distortion depends on numerous factors, including thickness. Thinner plates have the greatest tendency for "buckling distortion," which is commonly characterized as "oil canning" and "waviness." "Angular distortion," i.e., a deviation in the intended angle between welded plates, is typically the more significant issue for plates with larger thicknesses. "Bowing distortion" occurs due to eccentricity of the welds relative to the center of stiffness of the components being welded in a given configuration during fabrication. Proper clamping or restraint of the workpiece is often crucial to resist and reduce angular as well as bowing distortion.

Thicker plates tend to have less distortion due to their greater inherent ability to resist the residual stresses that cause distortion, and plates with smaller slenderness, *b*/*t*, have lesser tendency for buckling distortion. It is commonly stated that the simplest way to alleviate welding distortion is not to over-weld (i.e., to avoid providing welds that are larger than required for strength and/or minimum size requirements). Respondent 25 emphasized this in their comments to the survey. With this said, there are various control procedures that may be used in various combinations to alleviate welding distortion. The technology and general understanding certainly exists to control weld distortion when welding thin plates. However, many variables are involved. Also, the general impact of the necessary controls to achieve geometries that are within expected or required tolerances on plate out-of-flatness, and to eliminate visible oil-canning, on the cost of fabrication is not easily quantified. As stated by Respondent 21, "The answer is not easy … Many parameters are involved… We often do a mock-up (even when not contractually required) to validate our assumptions for distortion control."

It is clear from the NCHRP 20-07/415 survey and the literature review at large that welding distortion by itself is not the only consideration pertaining to ease of fabrication. Bending of the bottom flange plate under its self-weight and other loadings such as jacking of the bottom flange plate against cut-curved webs to place the longitudinal welds, workers walking inside the box, etc. are also factors. These considerations are addressed further in Section 4.7 of this report.

As noted in the discussion associated with Item 16 of Section 2.2.2, Zhang (2007) and Asadnia (2018) have recommended a bottom flange plate out-of-flatness tolerance of *b*/200 based on consideration of the impact of out-of-flatness imperfections on first yielding in plates subjected to compression as well as the "average fabrication ability of today's industry" (Zhang 2007). Zhang (2007, 47-49) reviews several other research studies, namely Koral and Thimmhary (1984) and Thimmhary and Koral (1987a; 1987b). These studies establish some statistical correlation between measured box-section bottom flange plate out-of-flatness and plate thickness, and show that thinner bottom flange plates in box girders statistically tend to have larger out-of-flatness. However, beyond a certain extent, Zhang (2007) and Asadnia (2018) also show that thinner (or more slender) flange plates also tend to have smaller sensitivity of their ultimate strengths to out-of-flatness.

3.3.4 Buckling of Thin Bottom Flange Plates

A wide range of buckling solutions are available that can be related directly to the slenderness, b/t, of the bottom flange plate in a box-section member. A specific focus in this research study is on the buckling of flange plates intended to serve predominantly in tension due to unanticipated or accidental axial compression. Accidental axial compression during lifting and interim stages of erection is of particular interest. Table 2 lists the traditional AASHTO suggestions for the maximum b/t of flange plates loaded predominantly in compression, aimed at ensuring a reasonably efficient use of the steel plates. There do not appear to be any focused studies in the literature that delve into the above key consideration of unanticipated or accidental axial compression in bottom flange plates, and the relationship to potential recommended not-to-exceed b/t limits. Section 4.1 provides general references and evaluates these considerations in detail.

3.3.5 Steel Box Section Distortion under Torsional Loading

The fourth highest priority consideration listed in Table 1 is localized plate bending stresses due to box-section distortion. Regarding the calculation of local plate bending stresses due to box-section distortion, the Beam on Elastic Foundation (BEF) analogy developed by Wright and Abdel-Samad (1968) has long been the primary calculation approach referenced within US practice. The solution by these authors relies centrally on the selection of major parameters via charts. Haaijer (1981) developed a simple modeling technique that implements these concepts in a local analysis of the cross-section using a two-dimensional frame element idealization. Yoo et al. (2015) presented a detailed development and use of computational tools for analysis for box-section distortional stresses. Their approach also employs a two-dimensional frame element idealization for the local analysis. Fan and Helwig (2002) and Helwig et al. (2007) have presented comprehensive developments of closed-form equations for estimation of internal cross-frame forces associated with the restraint of cross-section distortion in bridge tub girders. These authors point to Dabrowski (1968) for the specific calculation of box-girder distortional warping and distortional plate bending stresses. Appendix B of BS5400 (BSI 2000), the British Code of Practice for Design of Steel Bridges, provides detailed closed-form equations for calculation of plate distortional stresses in trapezoidal box-section members.

Recently, extensive test simulation studies on noncomposite box section members have been conducted as part of the development of noncomposite box-section member design provisions balloted and approved by the AASHTO Committee on Bridges and Structures (Lokhande and White, 2018). These studies indicate that cross-section distortion of box-section members generally has a small influence on the ultimate strengths associated with axial compression, flexure, shear and biaxial bending where there is no significant applied torsion. However, it is well established that internal cross-frames and/or diaphragms are essential to limit the distortional warping and plate transverse bending stresses in box-section members subjected to significant applied torsional loads. In the above provisions, the factored transverse plate bending stresses due to cross-section distortion are limited to 20.0 ksi at the strength limit state in box-section members subjected to large torques. This is the same as the conventional AASHTO requirement for composite box-section members.

The BEF procedures recommended by Wright and Abdel-Samad (1968), illustrated in detail in Grubb et al. (2010), are viewed as a reasonable established first-level approach for evaluating the relationship of potential bottom flange thickness and/or *b*/*t* limits on the behavior associated with distortion under torsional loads. Therefore, it was decided to focus on how the bottom flange thickness, *t*, and slenderness, *b*/*t*, enter into the mathematical equations of this approach. In basic terms, it can be observed that two bottom flange factors have an influence on the plate bending stresses due to distortion. These are:

1. The plate bending rigidity

$$D = \frac{Et^3}{12\left(1-\nu^2\right)} \quad \text{Eq. 1}$$

2. The plate bending section modulus for a unit width plate strip associated with the plate bending distortion

$$S = \frac{(1)t^3}{6} \quad \text{Eq. 2}$$

The plate bending rigidity, *D*, of the bottom flange enters broadly into the calculation of the overall box-section stiffness properties associated with distortion under torsional loading, along with the corresponding rigidities for the two webs and that for the top flange. As one might expect, the section modulus of the bottom flange, *S*, can be tied more directly to the actual estimated plate bending stress in the bottom flange due to cross-sectional distortion.

This basic assessment leads to the conclusion that the most important direct bottom flange factor relating to distortion of the box cross-section is the section modulus of the bottom flange relative to that of the webs. The influence of the bottom flange thickness and *b*/*t* on the cross-section distortional stiffness or flexibility is relatively indirect. However, at the risk of stating the obvious, one can conclude that for a given overall "system" stiffness of the box cross-section, the bottom flange section modulus should be greater than that of the webs, to avoid having distortional plate bending stresses in the bottom flange larger than those in the webs.

Therefore, the simple and obvious conclusion from considering the distortional analysis of the box-girder cross-section using the Wright and Abdel-Samad (1968) equations is that the thickness of the compression and tension flanges corresponding to the box section principal axis direction with the largest bending moment should be greater than or equal to the thickness of the box-section webs.

CHAPTER 4

Analytical Studies

This section presents a number of analytical investigations intended at complementing and supplementing the findings from the targeted surveys discussed in Chapter 2 and the Literature Review presented in Chapter 3.

4.1 Elastic Plate Buckling Under Accidental or Unintended Longitudinal Axial Compression

The theoretical elastic buckling resistance of a rectangular plate with a width b and thickness t may be expressed as

$$F_{cr} = \frac{\pi^2 Ek}{12\left(1-\nu^2\right)\left(b/t\right)^2} = \frac{0.90Ek}{\left(b/t\right)^2} \qquad \text{Eq. 3}$$

where E is the elastic modulus of the material, commonly taken as E = 29,000 ksi for steel plates in US practice, ν is Poisson's ratio, commonly taken as 0.3, and k is the plate buckling coefficient. For simply supported rectangular plates, k varies as a function of the plate length-to-width ratio, a/b, exhibiting a minimum value of 4.0 at a/b = 1, 2, 3, and for "long" plates as a/b theoretically approaches infinity. The minimum value of k = 4.0 is commonly used for design. This equation also applies to a rectangular panel in a stiffened plate, as long as the stiffeners are assumed sufficient to restrain the out-of-plane deflections along the panel edges. In this case, the panel width w is substituted for b.

Figure 1 shows a plot of the corresponding elastic buckling stress, F_{cr}, determined from the above equation versus b/t. The solutions for a simply supported rectangular plate with a linearly varying axial stress ranging from a maximum at one edge to zero at the other edge (f_1/f_2 = 0), where the minimum k is 7.8, and for a simply supported rectangular plate with a linearly varying axial stress ranging from a maximum compression at one edge to an equal maximum tension at the other edge (f_1/f_2 = -1), where the minimum k is 23.9, are also shown in the plot.

Several b/t values along the solid line curve associated with uniform axial compression are worthy of note:

The AISC 360-16 (AISC 2016) Specification and numerous other design standards for steel construction commonly recommend a practical maximum effective slenderness of KL/r = 200 for column members. AASHTO (2017) LRFD Article 6.8.4 limits L/r to 200 for primary tension members not subject to stress reversals, and Article 6.9.3 limits the slenderness of primary compression members to KL/r = 120. Using the theoretical column elastic buckling equation, KL/r = 200 corresponds to a nominal buckling resistance of

$$F_{cr} = \frac{\pi^2 E}{\left(KL/r\right)^2} = 7.2 \text{ ksi} \qquad \text{Eq. 4}$$

If one were to adopt a philosophy that the theoretical elastic plate buckling resistance – based on $k = 4.0$, corresponding to simply supported edge conditions – should also be at least equal to this value, the corresponding plate width-to-thickness ratio (or slenderness) must satisfy

$$b/t \leq 120 \qquad \text{Eq. 5}$$

Interestingly, this limit is the same as that recommended by Wolchuk and Mayrbaurl (1980). Therefore, the above development can be employed as a simple justification of the Wolchuk and Mayrbaurl (1980) maximum b/t recommendation.

A value of stress equal to 10 ksi (20 % of the yield strength for Grade 50 steel) is a common estimate for the residual compression induced within the main area of rectangular plates, away from the heat affected zones, due to welding along their longitudinal edges (Lokhande and White 2018). Based on the solid-line curve in Figure 1, simply supported plates more slender than $b/t = 100$ are apt to experience theoretical buckling under these residual compression stresses. If some additional compression is applied to the plates, such as due to shifting of true inflection point locations from nominal positions calculated in design, buckling distortion of the plates associated with "oil canning" or "waviness" may be likely. To alleviate these concerns, the project team recommends that the b/t of box-section flanges should be limited to

$$b/t \leq 90 \qquad \text{Eq. 6}$$

Based on the solid line curve in Figure 1, this slenderness eliminates theoretical elastic plate buckling up to a combined axial compressive stress due to welding residual compression plus applied loads of 13 ksi.

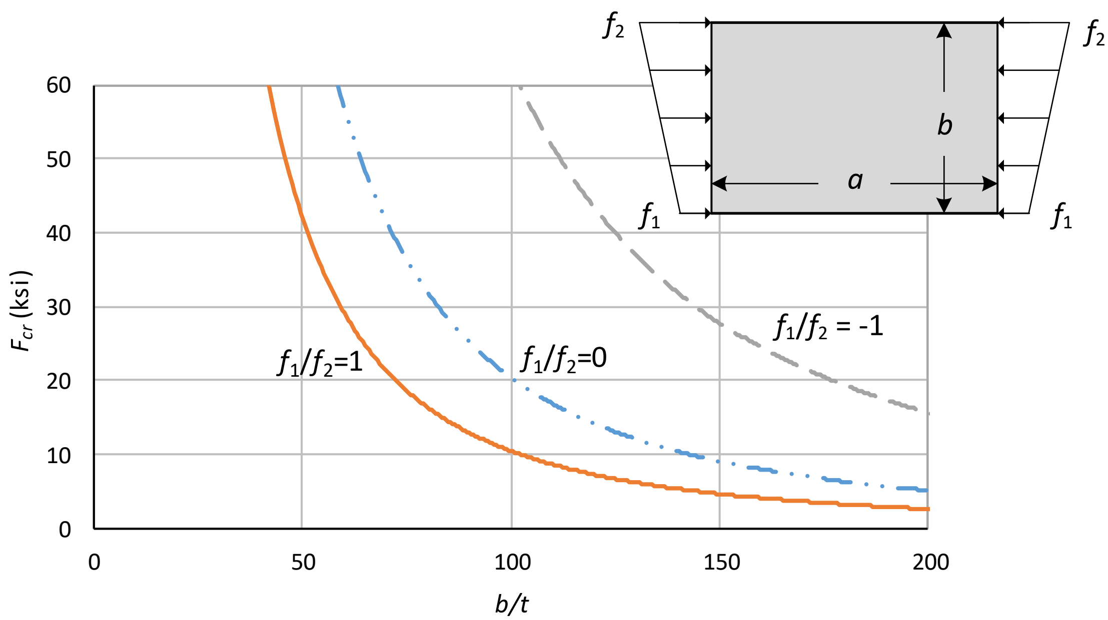

Figure 1. Simply supported plate elastic buckling stress F_{cr} versus b/t for uniform longitudinal axial compression, and for axial compression varying linearly from an edge maximum compressive stress to zero and from an edge maximum compressive stress to an equal and opposite edge tensile stress.

It is important also to consider the problem of buckling under accidental or unintended axial compression from the demand side. Figure 2a shows the basic shear and moment diagrams for a simply supported prismatic girder subjected to uniformly distributed transverse load, q_F, in its final constructed condition while Figure 2b shows the corresponding diagrams corresponding to lifting of the girder under its self-weight, q_D, during construction. The pick points on the girder are located at a distance αL apart, where L is the final span length. The positive and negative self-weight bending moments are minimum, equal to

$$0.172(q_D L^2/8) \qquad \text{Eq. 7}$$

if the girder is lifted from two pick points via a spreader beam or two cranes, such that $\alpha = 0.586$, or $(1 - \alpha)/2 = 0.207$. If the girder is lifted from one pick point at its mid-span ($\alpha = 0.0$), the negative dead load moment is equal to

$$q_D L^2/8 \qquad \text{Eq. 8}$$

which is the same as the simply supported maximum self-weight moment within the span. Therefore, the negative bending moment, causing "accidental" compression in the bottom flange during lifting of the girder, is significant in typical cases, and is as large as the span's self-weight positive bending moment.

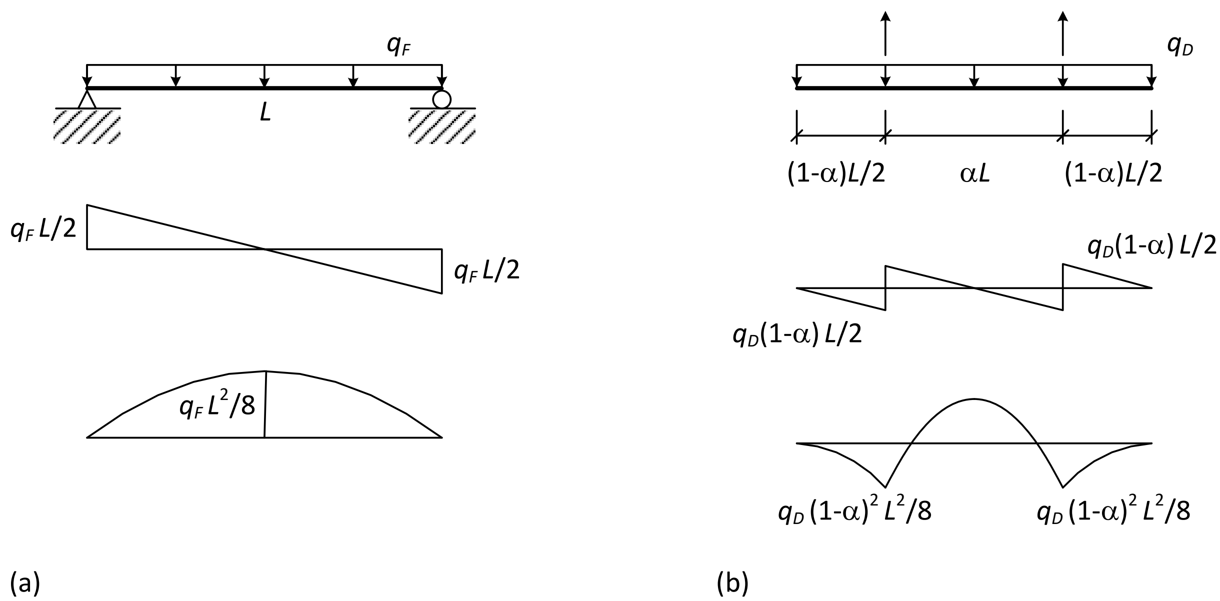

Figure 2. Shear and moment diagrams for a simply supported prismatic girder subjected to uniformly distributed transverse load, q_F, in its final constructed condition and due to its self-weight, q_D, during lifting of the girder.

Figure 3 shows another case involving negative bending moment during construction. In this case, the prismatic member has a cantilever overhang of length βL at the right-hand end of its main span of length L. The member is subjected to its self-weight, q_D, plus an additional load of $\Gamma(q_D L/2)$ at the free end of the cantilever. For this loading and geometry, the negative moment at the right-hand support is

$$(q_D L^2/2)\,(\Gamma\beta + \beta^2) \qquad \text{Eq. 9}$$

as shown in the figure. Figure 4 plots the combinations of β and Γ resulting in a negative moment at the right-hand support equal to the simple-span maximum moment $q_D L^2/8$. One can observe the corresponding β value ranges from 0.5 at $\Gamma = 0$ to 0.207 at $\Gamma = 1.0$. Therefore, as noted above, negative moments, causing compression in the bottom flange, can be significant during construction. They can easily be of a magnitude comparable to the simple-span maximum self-weight positive bending moment. If the bottom flange of a girder cross-section is relatively slender (i.e., large b/t in Figure 1), the flange can easily be overstressed during construction.

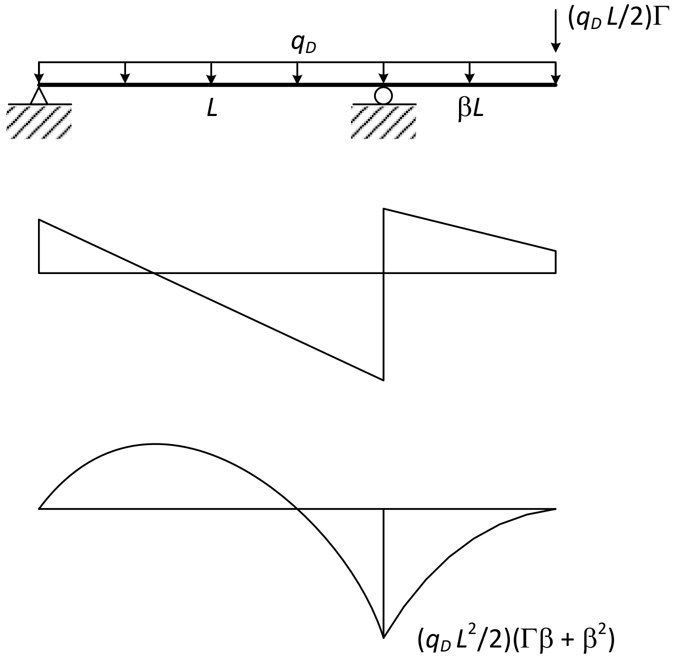

Figure 3. Shear and bending moment diagrams for a prismatic girder with an overhang, subjected to its self-weight, q_D, plus an additional load of $\Gamma(q_DL/2)$ at the free end of the cantilever.

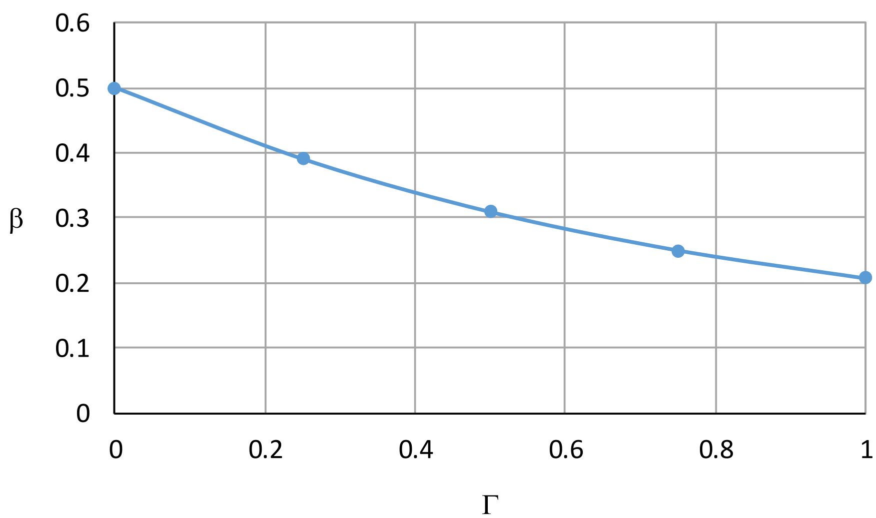

Figure 4. Values of β and Γ associated with the development of a moment of $q_DL^2/8$ over the right-hand support in the prismatic girder with an overhang shown in Figure 3.

As noted above, and targeted within the project survey (see Table 1, Consideration 2), one can also have "accidental" or "unintended" compression in an unstiffened bottom ("tension") flange near typical field-splice locations in continuous-span box-section members (i.e., near dead-load contraflexure points) due to stress reversal resulting from the compressive stress due to the minimum live load plus impact moments. This can be particularly problematic when the span balance is such that longitudinal stiffeners utilized in the adjacent field section over the pier are terminated at or near the field-splice locations. Grubb et al. (2010, 3.2.158-3.2.160) discuss the checking of an unstiffened tub girder bottom flange on one side of a bolted field splice where the flange on the opposite side of the splice is longitudinally stiffened in a continuous span. Grubb et al. (2010, Section 5.2.9.4.1.2) provides an example design check for this case.

4.2 Plate Shear Buckling

The minimum shear buckling coefficient for long (unstiffened) simply supported plate is k = 5.34 (Ziemian 2010). Equation 3 still applies with this coefficient, but the buckling stress corresponds to the uniform shear stress within the plate. Figure 5 shows the elastic shear buckling stress versus b/t.

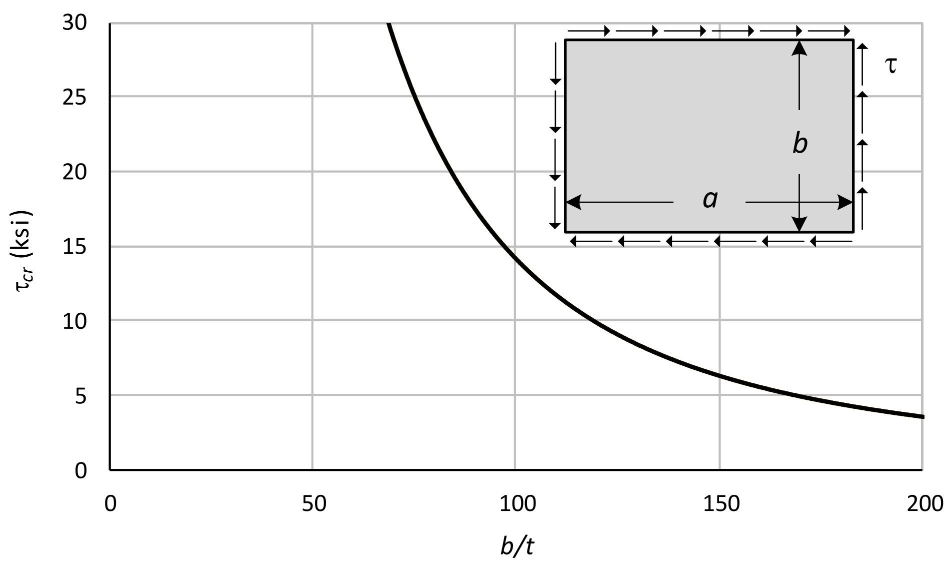

Figure 5. Simply supported plate elastic shear buckling stress τ_{cr} versus b/t.

For $b/t = 90$,

$$\tau_{cr} = \frac{0.90Ek}{\left(b/t\right)^2} = 17 \text{ ksi} \qquad \text{Eq. 10}$$

This level of shear stress is roughly 60 % of the corresponding plate yield strength in shear for Grade 50 steel. It is not expected that typical applications would exceed this level of shear stress under construction, service or fatigue loading conditions.

4.3 Transverse Compression on Bottom Flange from Inclined Webs, and From Other Potential Actions

Figure 6 shows a simply supported rectangular plate subjected to uniformly distributed compressive load in the direction perpendicular to the longitudinal axis of the member. This type of loading can be taken as an idealization of the demands on a box girder bottom flange plate due to the inclined webs in a trapezoidal cross-section. In this case, the transverse compression demand on the bottom flange per unit length along the girder may be estimated conservatively as

$$\frac{q}{2}\tan\phi \qquad \text{Eq. 11}$$

(Fan and Helwig, 1999), where q is the uniformly distributed load supported by the girder, and ϕ is the angle of inclination of the web.

One assumption in the above calculation is that each web supports one-half of the transverse load. In addition, Eq. 11 is considered to provide a reasonable approximation of the uniform transverse tension in the top flange, i.e., the flange on the side of the member receiving the applied load q. However, considering beam theory, the bottom flange on the opposite side provides only a small fraction of the total shear, V, resisted by the overall box cross section. Since $dV/dx = -q$ from fundamental equilibrium considerations, the above transverse tension in the top flange will be equilibrated predominantly by a variation in the shear flow tangent to the inclined web, $dV\,Q/I$, not by a transverse compression in the bottom flange. Transverse compression in the bottom flange can be induced locally near box-section member supports; however, internal cross frames or diaphragms will assist substantially in resisting the transverse compression at these locations. Nevertheless, there can be some limited transverse compression in the bottom flange, and therefore it is of interest to quantify the resistance of the bottom flange to this loading.

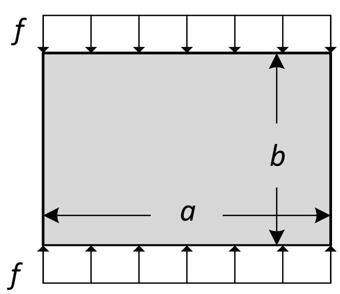

Figure 6. Simply supported plate subjected to a uniformly distributed compressive load perpendicular to the longitudinal axis of the member.

In the limit of a long simply supported plate, the bottom flange plate buckles under the above transverse compression essentially as parallel column strips of unit width. The corresponding buckling stress is

$$F_{cr} = \frac{\pi^2 E}{\left(1-\nu^2\right)\left(KL/r\right)^2} = \frac{\pi^2 E}{12\left(1-\nu^2\right)\left(b/t\right)^2} = \frac{0.90E}{\left(b/t\right)^2} \qquad \text{Eq. 12}$$

The term $(1 - \nu^2)$ in the denominator accounts for the Poisson effect of a wide column.

By equating the transverse compression load from Eq. 11 to the above buckling stress, multiplied by t, and solving for t, one obtains

$$t \geq \frac{q\tan\phi\left(b/t\right)^2}{1.8E} \qquad \text{Eq. 13}$$

If $\tan\phi$ is taken as the maximum slope of the webs considered in AASHTO LRFD for ordinary tub girders, i.e., ¼, this equation becomes

$$t \geq \frac{q(b/t)^2}{7.2E} \qquad \text{Eq. 14}$$

Upon substituting the maximum *b*/*t* limit associated with the above studies, $b/t = 90$, and $E = 29{,}000$ ksi, this becomes

$$t \geq 0.039q \qquad \text{Eq. 15}$$

with *q* and *t* expressed in consistent units of kips/inch and inches. Alternatively, for a given *t* value, the transverse distributed load that can be supported is

$$q = 26t \qquad \text{Eq. 16}$$

Therefore, for a girder with $t = 0.5$ inches for the bottom flange (the minimum discussed in the responses to the survey, and the minimum value cited in Table 2) and $b/t = 90$, the bottom flange can support a transverse distributed load of 13 kips/inch = 160 kips/ft. This is larger than any practical value expected for a trapezoidal box girder.

In addition to the fact that the above analysis indicates little chance of bottom flange buckling due to transverse compression, it should be emphasized that the analysis is conservative in that (1) it neglects the resistance provided by the shear flow in the inclined webs as well as (2) it ignores the resistance from internal K frames. The internal K frames, in conjunction with the webs and web stiffeners, effectively form a system with two "upside down" A-frame type trusses assisting in resisting transverse compression forces.

4.4 Bottom Flange Elastic Buckling Resistance to Concentrated Transverse Edge Loads

It may be of interest to consider the maximum concentrated transverse load (i.e., perpendicular to the longitudinal axis of the member) that a bottom flange plate with $t = 0.5$ and $b/t = 90$ can support. Timoshenko and Gere (1961) provide the following solution for this case, for simply supported rectangular plates with $a/b > 2$ loaded by diametrically opposed concentrated loads at the mid-width of their long edges (Figure 7):

$$P_{cr} = \frac{\pi E t^3}{3(1-\nu^2)b} = \frac{\pi E t^2}{3(1-\nu^2)(b/t)} \qquad \text{Eq. 17}$$

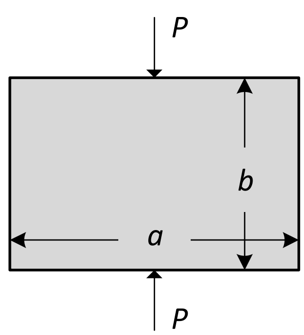

Figure 7. Simply supported plate subjected to a concentrated compressive load perpendicular to the longitudinal axis of the member at the mid-width of the long edges.

The plate is actually stronger if only one side is subjected to a concentrated transverse load, such as a concentrated load due to a vehicle collision. However, the authors were not able to find any simple analytical solution for this case. Substantial studies have been devoted to this problem in the context of web crippling, or web buckling under a transverse "patch" load (Ziemian 2010); however, these solutions include the influence of a flange on the distribution of the load within a web plate.

Given Eq. 17, if one substitutes t = ½ inch and b/t = 90 (b = 45 inches), a buckling load of P_{cr} = 93 kips is obtained. This is a reasonable magnitude for a bottom flange plate transverse buckling resistance, e.g., the bottom flange elastic buckling resistance associated with a concentrated impact load from a vehicle collision. This can be compared to the required transverse load resistance of 54 kips from vehicle impact on barrier rails under the TL-4 railing test level, "taken to be generally acceptable for the majority of applications on high-speed highways, freeways, expressways and Interstate highways with a mixture of trucks and heavy vehicles" (AASHTO 2017, Table A13.2-1). It is acknowledged that the load demand from a vehicle collision on the bottom flange of a box girder is different from the barrier rail load demand; however, it is submitted that the above value is a reasonable rough approximation of this load demand. Specific design of box girders for vehicle or vessel/ship collision may require larger thicknesses and smaller b/t values than the respective minimum and maximum limits considered in this research.

4.5 Out-of-Plane Deflection of the Bottom Flange Plate Subjected to Its Self-Weight Plus a Concentrated Load at Its Mid-Width

Respondent 11 recommended that the out-of-plane deflection of the bottom flange of a box girder should be held to less than or equal to $b/360$ under the plate self-weight (0.49 kcf) plus a concentrated load of 0.5 kip as a design criterion for a box-girder bottom flange plate. One may consider the 0.5 kip load as a live load associated with bridge inspection. This requirement can potentially be placed either on the total out-of-plane deflection of a longitudinally unstiffened plate, or on the out-of-plane deflection within a panel of a stiffened plate. All of the solutions are shown below in the context of an unstiffened plate of width b. They may be applied to a panel of a stiffened plate by substituting the panel width w for b.

The maximum out-of-plane deflection of a long simply supported plate due to its self-weight may be calculated by simply considering a unit strip of the plate across its width:

$$\delta = \frac{5qb^4\left(1-\nu^2\right)}{384EI} = \frac{5\left[t\left(0.490\text{ kcf}\right)/\left(12\text{ in/ft}\right)^3\right]b^4\left(1-\nu^2\right)}{384E\left(t^3/12\right)} \qquad \text{Eq. 18}$$

Solving for the ratio of the out-of-plane deflection to the plate width, one obtains

$$\frac{\delta}{b} = 1.39\times10^{-9}\left(\frac{b}{t}\right)^3 t \qquad \text{Eq. 19}$$

where t is expressed in consistent units of inches. As expected, this solution matches with the corresponding plate solution given for $a/b = \infty$ in Young and Budynas (2002, 502), which is based on Timoshenko and Woinowsky-Krieger (1959).

The corresponding maximum out-of-plane deflection in a long simply supported plate subjected to a concentrated load at its mid-width may be expressed as (Young and Budynas, 2002, 502)

$$\delta = \frac{0.1851P\left(b/t\right)^2}{Et} \qquad \text{Eq. 20}$$

Upon substituting for E and dividing through by the plate width, b, one obtains

$$\frac{\delta}{b} = \frac{6.38\times10^{-6} P\left(b/t\right)}{t^2} \qquad \text{Eq. 21}$$

where t is expressed in consistent units of inches and P is expressed in kips.

It is of interest also to determine the maximum plate bending stress, located at the middle of the plate, for the above cases. For a long simply supported plate subjected to its self-weight, the plate bending stress in the direction of the plate width may be calculated from the beam solution as

$$\sigma = \frac{\left(qb^2/8\right)\left(t/2\right)}{I} = \frac{\left[t\left(0.490\text{ kcf}\right)/\left(12\text{ in/ft}\right)^3\right]\left(b^2/8\right)\left(t/2\right)}{\left(t^3/12\right)} = 2.13\times10^{-4}\left(\frac{b}{t}\right)^2 t \qquad \text{Eq. 22}$$

Again, as expected, this matches with the solutions from Young and Budynas (2002, 502) and Timoshenko and Woinowsky-Krieger (1959).

The solution for the maximum plate bending stress due to a concentrated loading at the plate mid-width requires the assumption of a circular area over which the concentrated force is applied as a uniform load. Taking this area to have a radius of $r_o = 1$ inch, which is greater than or equal to $0.5t$ for the thicknesses up to 2 inches considered in this study, this stress may be expressed as

$$\sigma = \frac{3P}{2\pi t^2}\left[\left(1+\nu\right)\ln\frac{2b}{\pi}+1.0\right] \qquad \text{Eq. 23}$$

for the long simply supported plate case considered in this work (Young and Budynas, 2002, 502; Timoshenko and Woinowsky-Krieger, 1959). This equation can be simplified to

$$\sigma = \frac{0.477P}{t^2}\left[1.3\ln\left(0.637\frac{b}{t}t\right)+1.0\right] \qquad \text{Eq. 24}$$

Figure 8 plots the total maximum δ/b from the above two solutions versus the thickness for $P = 0.5$ kips and for several b/t values, and compares these solutions to the recommended limit of 1/360. Figure 9 shows the corresponding solutions for the total maximum plate bending stress at the middle of the plate. The following observations can be gleaned from these plots:

1. For the recommended consideration of deflection under self-weight plus 0.5 kips, the suggested deflection criterion is satisfied by limiting the plate b/t to a maximum of 120 and maintaining $t \geq 0.5$ inches as long as the thickness does not become larger than 1 inch. For $t > 1$ inch, the deflection of the plate due to its self-weight starts to dominate and the 1/360 limit is violated. However, for $b/t = 120$, $t > 1$ inch corresponds to a plate width larger than 120 inches, which is unusual for an unstiffened bottom flange on a steel box girder. Excluding this unusual case, one can conclude that $b/t = 120$ and $t \geq 0.5$ inches are acceptable limits based on the satisfaction of the load-deflection criterion recommended by Respondent 11.
2. Clearly, from Figure 8, all practical cases of box girder bottom flanges with say $b \leq 120$ inches and $b/t \leq 120$ give acceptable performance based on the recommended criterion for $t \geq 0.5$ inches.
3. For $t < 0.5$ inches, it is clear that the behavior is unacceptable based on the recommended deflection criterion regardless of the plate b/t.
4. Considering the total maximum plate bending stresses shown in Figure 9 for $P = 0.5$ kips, combined with the effects of the plate self-weight, one can conclude that the maximum stresses (which are local to the concentrated load and are in the direction of the plate width) are limited to a maximum of 7.0 ksi by maintaining $t \geq 0.5$ inches and $b/t \leq 120$ for all practical bottom flange plate widths $b \leq 120$ inches.

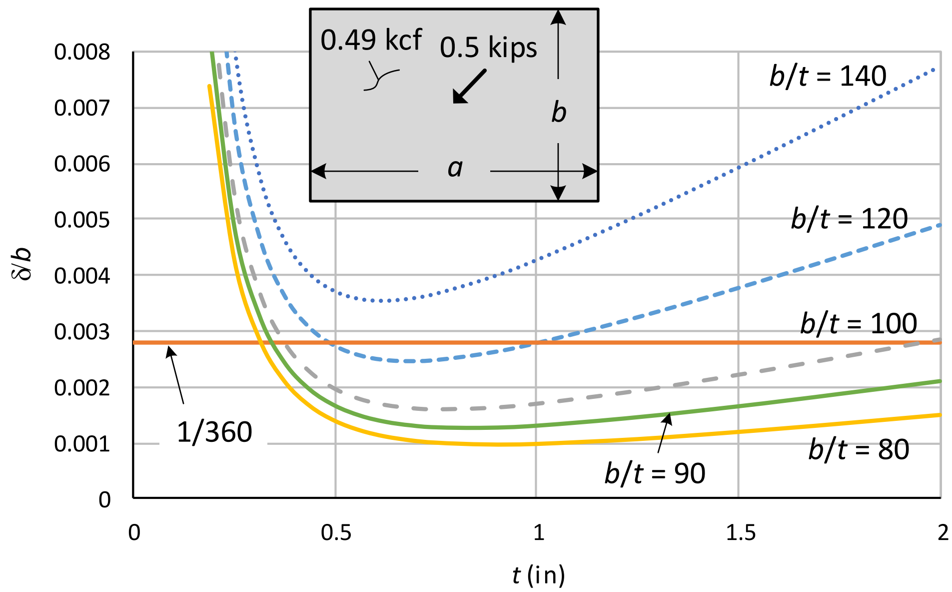

Figure 8. Maximum simply supported plate normalized out-of-plane deflection, δ/b, due to self-weight plus a concentrated load of 0.5 kips, versus the plate thickness, t, for several plate b/t values, and comparison to a limit of 1/360.

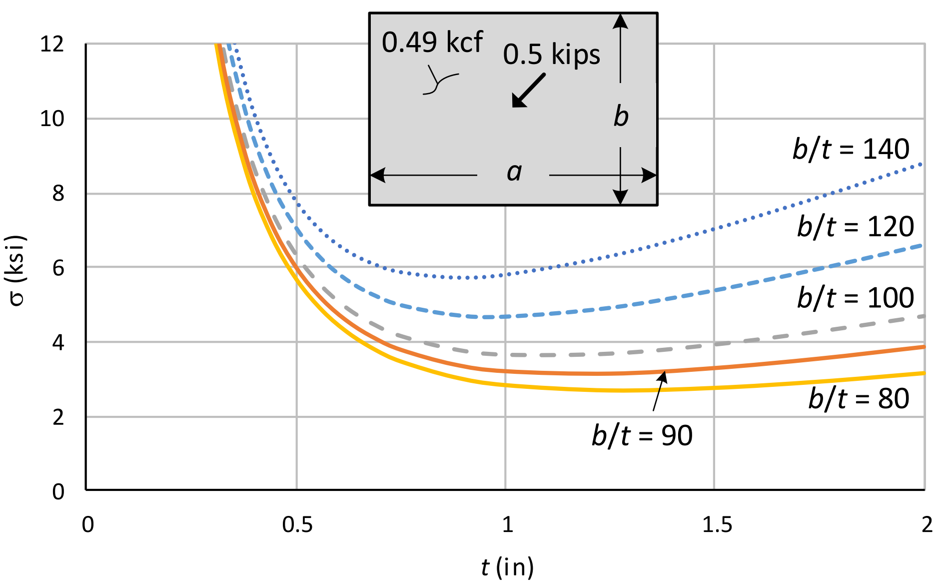

Figure 9. Maximum simply supported plate bending stress due to self-weight plus a concentrated load of 0.5 kips, versus the plate thickness, t, for several plate b/t values.

The selection of a concentrated load of 0.5 kips is a matter of judgment. Figure 10 shows the solution for the case of combined self-weight and $P = 1$ kip and Figure 11 shows the solution for combined self-weight and $P = 2$ kips. Figure 12 compares the normalized deflections for self-weight plus a live load of 0.3 kips at the middle of the plate to a normalized deflection limit of 1/300. Figures 13 through 15 show the corresponding solutions for the total maximum plate bending stress at the middle of the plate.

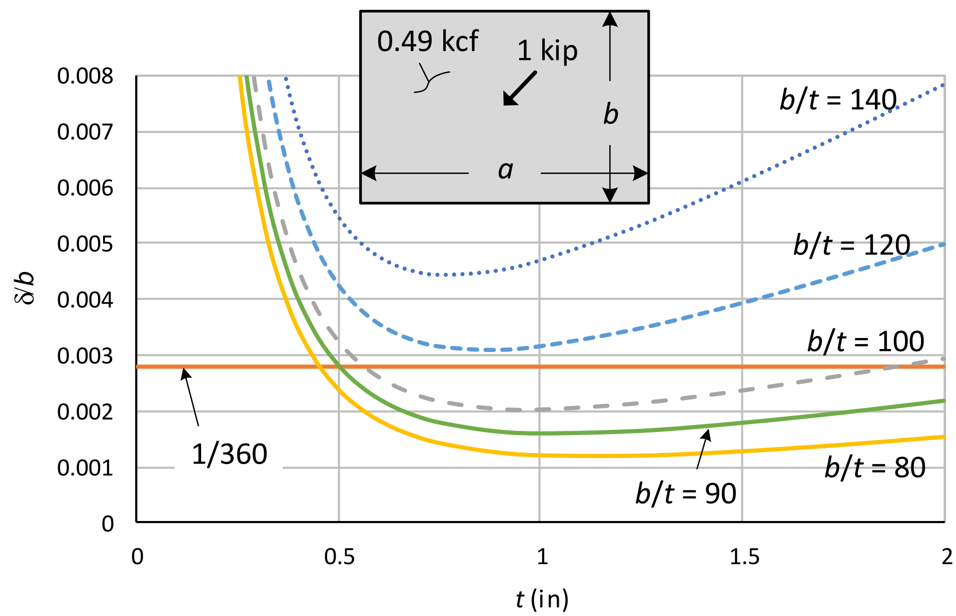

Figure 10. Maximum simply supported plate normalized out-of-plane deflection, δ/b, due to self-weight plus a concentrated load of 1 kip, versus the plate thickness, t, for several plate b/t values, and comparison to a limit of 1/360.

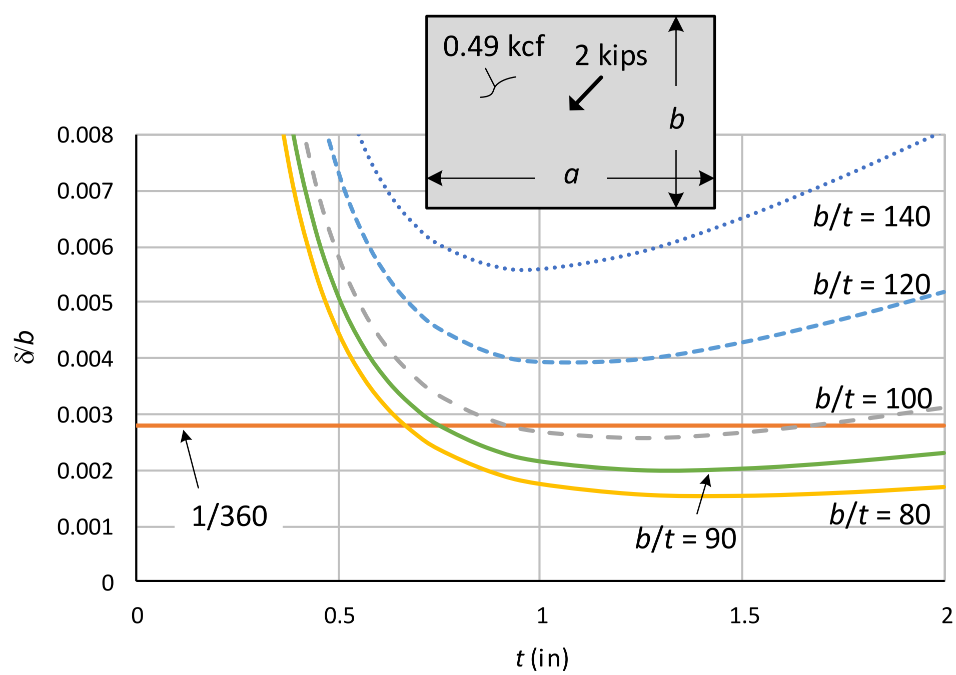

Figure 11. Maximum simply supported plate normalized out-of-plane deflection, δ/b, due to self-weight plus a concentrated load of 2 kips, versus the plate thickness, t, for several plate b/t values, and comparison to a limit of 1/360.

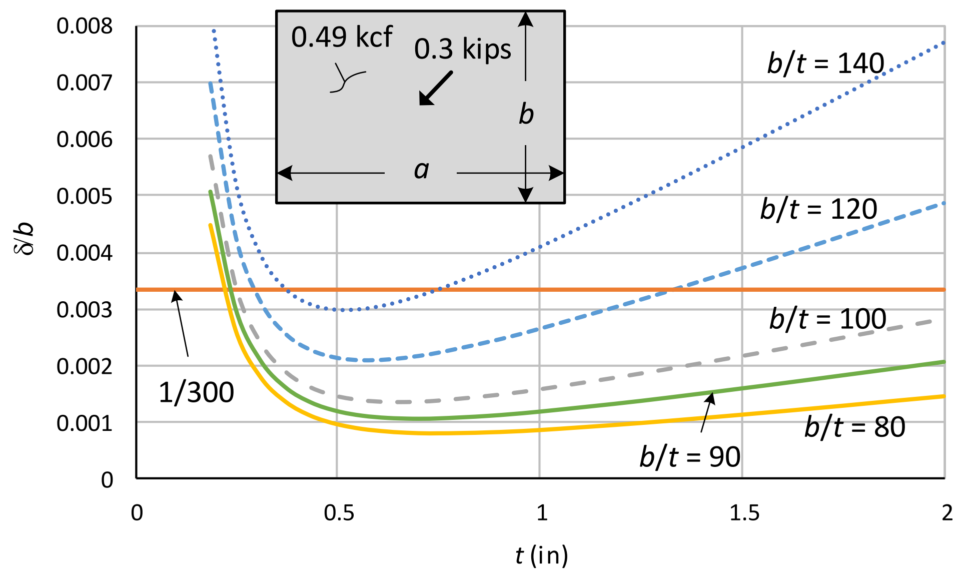

Figure 12. Maximum simply supported plate normalized out-of-plane deflection, δ/b, due to self-weight plus a concentrated load of 0.3 kips, versus the plate thickness, t, for several plate b/t values, and comparison to a limit of 1/300.

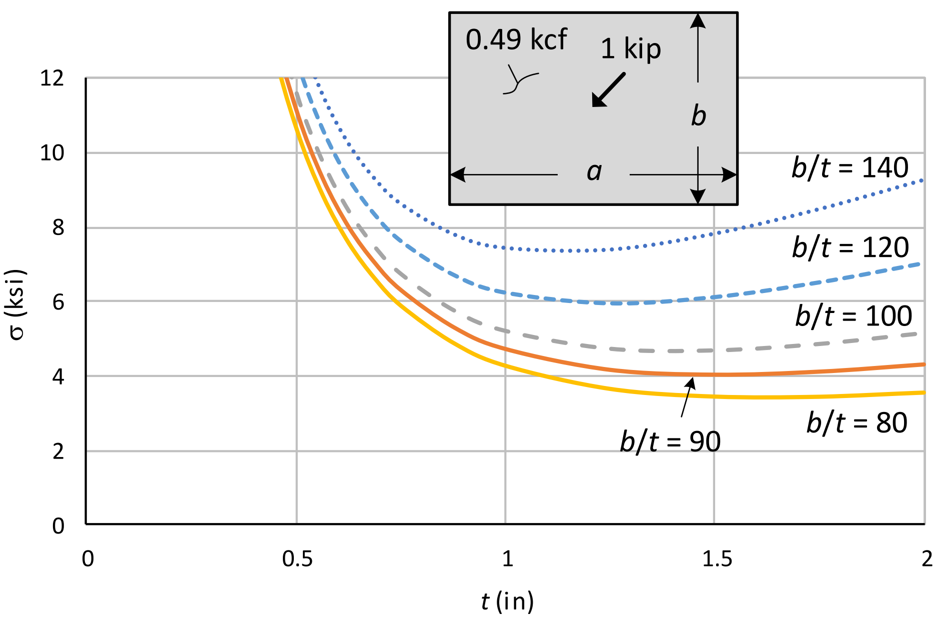

Figure 13. Maximum simply supported plate bending stress due to self-weight plus a concentrated load of 1 kip, versus the plate thickness, t, for several plate b/t values.

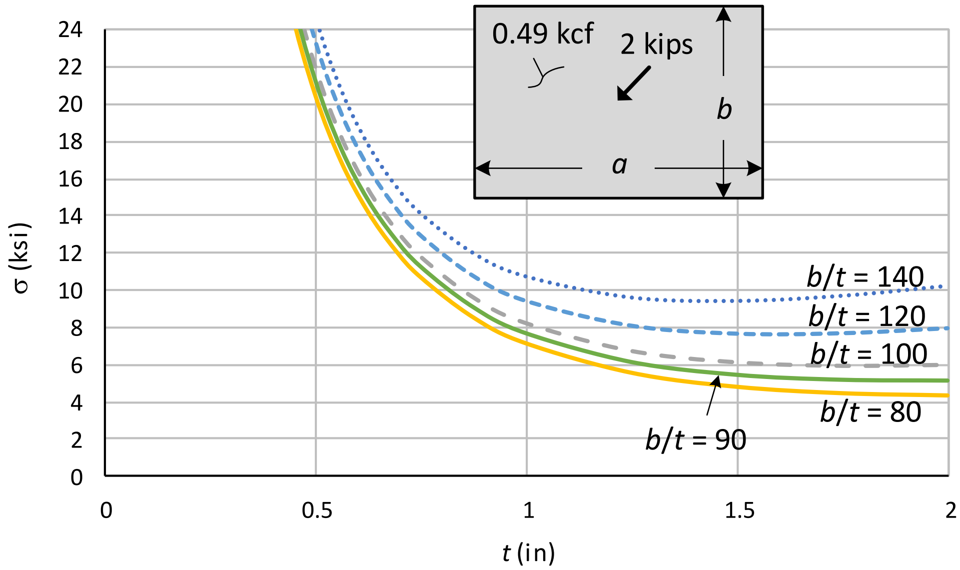

Figure 14. Maximum simply supported plate bending stress due to self-weight plus a concentrated load of 2 kips, versus the plate thickness, t, for several plate b/t values.

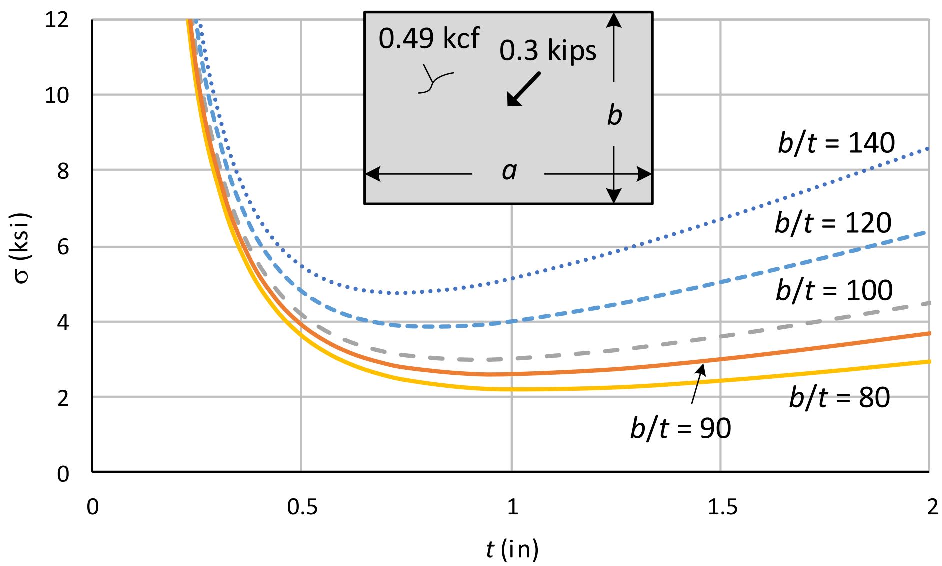

Figure 15. Maximum simply supported plate bending stress due to self-weight plus a concentrated load of 0.3 kips, versus the plate thickness, t, for several plate b/t values.

The following load requirements from Table 4-1 of ASCE 7-16 (ASCE 2016) are the most closely related to the live load requirements that might be considered for the bottom flange of a steel box girder:

1. Catwalks for maintenance access, 0.3 kip concentrated force or 40 psf distributed over the area.
2. Light manufacturing, 2 kip concentrated force or 125 psf distributed over the area.
3. All roof surfaces subjected to maintenance workers, 0.3 kip concentrated force.

If one assumes a maximum practical bottom flange width of 10 ft, the total live load on a 1 ft strip across the width of the plate is 0.4 kips for 40 psf distributed over the area, and 1.25 kips for 125 psf distributed over the area. The above concentrated loads of 0.3 and 2 kips would give a somewhat larger deflection and bending moment than these distributed loads for a long simply supported plate.

Regarding the deflection limit, Appendix CC of ASCE 7-16 states, "Historically, common deflection limits for horizontal members have been 1/360 of the span for floors subjected to full nominal live load and 1/240 of the span for roof members. Deflections of about 1/300 of the span (for cantilevers, 1/150 of the length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than 1/200 of the span may impair operation of movable components such as doors, windows, and sliding partitions."

Regarding the loading criterion, Appendix CC of ASCE 7-16 states, "For serviceability limit states involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short-term effects, the suggestion load combinations are $D + L$ …" That is, the combination of the nominal dead and live load, using load factors of 1.0, is recommended by ASCE 7-16 for checking these serviceability conditions.

In the context of a service deflection criterion, the above concentrated live load of 2 kips combined with a 1/360 deflection limit is judged to be a relatively extreme/conservative requirement. The plates have difficulty limiting the maximum plate bending stress essentially for all the b/t values under consideration for this loading (see Figure 14). This live load requirement is intended by ASCE 7 to apply to a floor system of a light manufacturing facility, which is certainly not the case for the bottom flange of a box girder in its service condition. Therefore, it can be argued that the self-weight plus 0.3 kips concentrated live load (in combination with the 1/300 deflection limit) is the most appropriate of the above criteria per ASCE 7-16. Unstiffened plates with $t \geq 0.5$ inches, b/t less than about 130, and practical widths of less than about 10 ft, clearly do not have any difficulty in satisfying the 1/300 deflection limit under this loading (see Figure 12). Furthermore, the local plate bending stress near the concentrated load is limited to 5.3 ksi (see Figure 15). The normalized out-of-plane deflections of these plates are in fact less than approximately 1/300 in all practical cases (for flange widths less than about 10 ft), and in fact for plate thicknesses between 0.5 and 0.75 inches, are close to 1/360. It can be argued that this smaller deflection limit is desirable, since the deflections under load are additive with the initial plate imperfections (assumed within a recommended out-of-flatness limit of 1/200 as discussed in Chapters 2 and 3).

One can observe that practical bottom flange plates with $b/t = 90$, the maximum limit on b/t recommended in Section 4.1, and t between 0.5 and 1.375 inches (resulting in flange widths between 45 and 124 inches), will have δ/b values due to self-weight plus a 0.3 kips load (not considering any initial imperfections associated with the fabrication of the girder) smaller than approximately 0.0016 = 1/600.

In addition, one can observe that given a maximum limit of $b/t = 90$, the ASCE 7-16 recommended deflection limit of $b/300$ is not exceeded for plate thicknesses as small as ¼ inch. As noted previously, the above results can also be applied to assess panels of stiffened flanges. This implies that plate thicknesses of 9 mm = 0.35 inches combined with $w/t \leq 90$, which have been employed commonly in signature long-span bridges, are not a problem. Plates this thin require attention to established procedures necessary to control welding distortion, but with $w/t < 90$, buckling distortion (i.e., oil canning and waviness of the plates) due to welding is expected to be very controllable.

It should be emphasized that all of the above solutions are heavily idealized. For instance, one can expect that some rotational restraint is provided at the longitudinal edges of the bottom flange plate from the box section webs. This will tend to reduce the maximum out-of-plane deflections and the local maximum plate bending stresses at the mid-width of the plate to some extent. In addition, in many cases the aspect ratio of the plate, a/b, is small enough such that the displacements and maximum plate bending stresses are reduced to some extent due to the plate bending in the two orthogonal directions of the member as well as due to the torsional resistance of the plate. As noted by survey Respondent 5 (see Appendix C), the maximum plate bending stresses may actually be at the boundaries of the plate, e.g., at a plate boundary with transverse stiffeners. Obtaining refined estimates considering these aspects is often best addressed by sophisticated finite element analysis models. Nevertheless, sophisticated finite element models are no better than the idealizations of the many potential geometries, loadings and boundary conditions behind them. The above solutions provide a useful simplified rational estimate of the magnitudes of the plate out-of-plane bending deflections and plate bending stresses.

4.6 Other Plate Out-of-Plane Deflection or Out-of-Plane Stiffness Checks – Dishing of a Plate due to Applied Edge Moments

Various references such as those listed in Section 3.3.3 provide substantive guidance regarding welding procedures and control of welding distortion. Guidance can be identified regarding stiffening of plates, by use of strong backs, stiffeners and/or clamping, to avoid distortion during welding. However, as might be expected, there are no specific limits in these references regarding the thickness or *b*/*t* for the plate components being welded.

Section 4.1 of this report has provided recommendations to avoid potential buckling distortion of welded rectangular plates. A simplistic assessment of the sensitivity of thin plates to curling or dishing due to shrinkage forces can be developed by considering the out-of-plane deflection of a simply supported thin plate strip due to applied end moments (see Figure 16). This deflection may be calculated as

$$\delta = \frac{(M / EI)\left(1-\nu^2\right)b^2}{8} = \frac{M\left(1-\nu^2\right)b^2}{8Et^3/12} \qquad \text{Eq. 25}$$

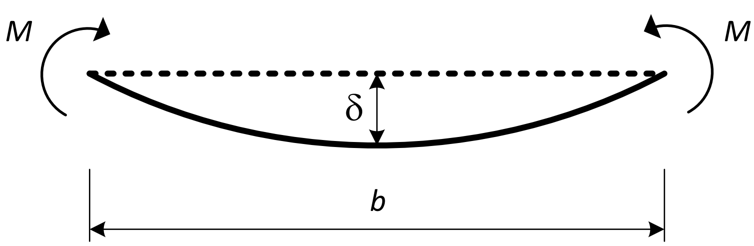

Figure 16. Out-of-plane dishing deflection of a plate strip subjected to applied end moments.

Upon dividing by the plate width and simplifying, one obtains

$$\frac{\delta}{b} = \frac{3M\left(1-\nu^2\right)(b/t)}{2Et^2/12} = \frac{3M}{2}\frac{\left(1-\nu^2\right)}{E}\frac{(b/t)}{t^2} \qquad \text{Eq. 26}$$

From this simplistic analysis, one can conclude that the normalized plate distortion due to welding is roughly inversely proportional to the square of the plate thickness, and directly proportional to the plate *b*/*t*. Of course, the eccentricity of welds relative to the neutral axis of the plate potentially could be larger for thicker plates, influencing the value of the simplistic moment term in the above expression. If these eccentricity effects are minimized, then one might consider the above expression to be of some value in terms of relating the potential of weld distortion to the plate thickness and slenderness. In this case, one can conclude that bowing or curling of plates due to weld shrinkage can be alleviated more effectively by increasing the thickness limit rather than decreasing the *b*/*t* limit. This conclusion is consistent with the comments from the survey respondent summarized under Item 4 of Section 2.2.2.

4.7 Other Plate Out-of-Plane Deflection or Out-of-Plane Stiffness Checks – Plate Bowing During Handling and Girder Fabrication in the Shop

As a simplistic quantification of the sensitivity of a bottom flange plate to bowing during handling and girder fabrication in the shop, consider the idealized deflection of a plate strip under its self-weight, supported solely by a concentrated reaction at its mid-width (Figure 17). The corresponding bowing deflection of the plate can be estimated as

$$\delta = \frac{q(b/2)^4(1-\nu^2)}{8EI} = \frac{\left[t(0.490\text{ kcf})/(12\text{ in/ft})^3\right]b^4(1-\nu^2)}{128E(t^3/12)} \qquad \text{Eq. 27}$$

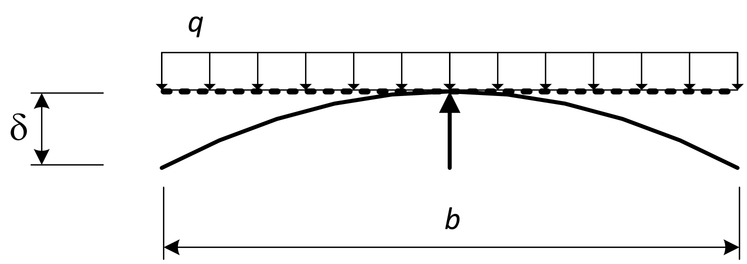

Figure 17. Bowing deflection of plate strip in which the plate is supported only at its mid-width.

Solving for the ratio of the out-of-plane deflection to the plate width, one obtains

$$\frac{\delta}{b} = 8.34\times10^{-10}\left(\frac{b}{t}\right)^3 t \qquad \text{Eq. 28}$$

This estimate is very similar to the estimate for the normalized out-of-plane deflection of the simply supported plate under its self-weight, determined previously. One can observe that the out-of-flatness in this case is 60 % of that for the simply supported plate under its self-weight. Therefore, satisfaction of the deflection criterion from Section 4.5 effectively ensures that the bowing of the plate across its width is reasonably limited during handling in the fabrication shop.

One might consider limiting the idealized plate bending stresses during handling and fabrication operations to less than say the yield strength of the material. Similar to the prior estimate for the simply supported plate geometry, the idealized plate bending stresses for the conditions shown in Figure 17 may be calculated as

$$\sigma = \frac{\left[q(b/2)^2/2\right](t/2)}{I} = \frac{\left[t(0.490\text{ kcf})/(12\text{ in/ft})^3\right](b^2/8)(t/2)}{(t^3/12)} = 2.13\times10^{-4}\left(\frac{b}{t}\right)^2 t \qquad \text{Eq. 29}$$

If this stress is equated to a yield strength, F_y = 50 ksi, one can write

$$\left(\frac{b}{t}\right)^2 t \le 235{,}000 \qquad \text{Eq. 30}$$

For a plate thickness of ½ inch, this requirement is satisfied by $b/t \le 690$, for a plate thickness of 1 inch, Eq. 30 is satisfied by $b/t \le 490$, and for a plate thickness of 2 inches, Eq. 30 is satisfied by $b/t \le 340$. Therefore, one can conclude that none of the practical bottom flange plates under consideration are at a high risk for damage due to yielding during handling in the shop. Of course, the plate is more flexible in the longitudinal direction when $a > b$.

4.8 Requirements to Limit the Out-of-Plane Deflection of Longitudinal Stiffeners Under Self-Weight Plus a Concentrated Live Load

One can argue that it would be prudent to apply the service out-of-plane deflection criteria from ASCE 7-16, discussed in Section 4.5, also to any flange longitudinal stiffeners. This consideration may be assessed by considering an isolated longitudinal stiffener as shown in Fig. 18. The stiffener is subjected to its self-weight plus the self-weight of the tributary width of the stiffened plate. In addition, it is subjected to a 0.3 kip concentrated transverse load at its mid-length between the supporting diaphragms and/or transverse stiffeners. Any assistance from the stiffened plate in resisting the out-of-plane deflections is conservatively neglected, and the longitudinal stiffener is conservatively assumed to be simply supported between adjacent diaphragm and/or transverse stiffener locations. The combined deflections are limited to $b/300$, where b is the total width of the stiffened plate. Assuming a long plate, the limit $b/300$ is more restrictive than $a/300$.

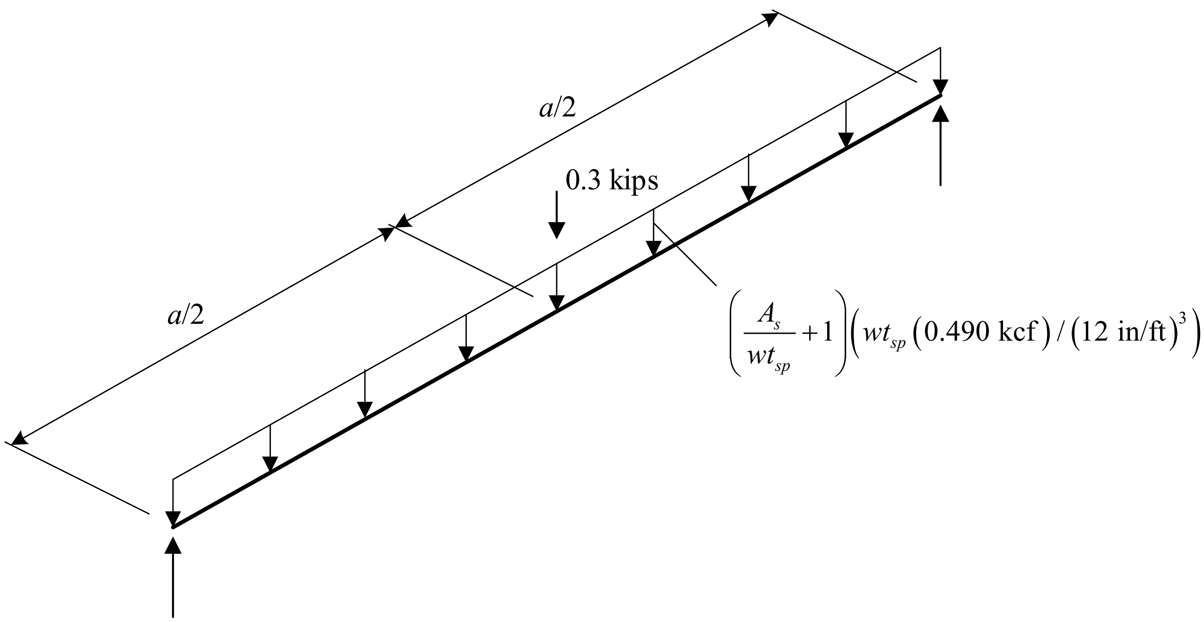

Figure 18. Model of a simply supported bottom flange plate longitudinal stiffener subjected to a 0.3 kip load at its mid-length between the diaphragms and/or transverse stiffeners, as well as its self-weight plus the self-weight of the stiffened plate tributary to the stiffener.

Since the prior normalized not-to-exceed limit for the longitudinal stiffeners recommended by Wolchuk and Mayrbaurl (1980) was expressed in terms of the slenderness a/r_s of the stiffeners, it is useful to consider quantifying the above deflection limit in this way if possible. The above service load-deflection criterion may be expressed as follows:

$$\delta = \frac{a^3}{EI_s}\left[\frac{0.3\text{ kip}}{48}+\frac{5}{384}\left(\frac{A_s}{wt_{sp}}+1\right)\left(awt_{sp}\left(0.490\text{ kcf}\right)/\left(12\text{ in/ft}\right)^3\right)\right]\le\frac{b}{300} \qquad \text{Eq. 31}$$

where I_s is the moment of inertia of the stiffener strut, composed of the longitudinal stiffener and the corresponding gross tributary width of the stiffened plate, A_s is the stiffener area, and t_{sp} denotes the thickness of the stiffened plate. Assuming equal stiffener spacing and a tributary width of the plate for a given stiffener equal to w, this equation may be solved for the required stiffener moment of inertia as

$$I_s \geq \frac{a^2}{E}\frac{a}{b}\left[1.875\ \text{kip} + \left(\frac{1}{900}\ \text{kip/in}^3\right)\left(\frac{A_s + wt_{sp}}{wt_{sp}}\right)awt_{sp}\right] \qquad \text{Eq. 32}$$

Next, taking the stiffener strut radius of gyration about an axis parallel to the plane of the stiffened plate as

$$r_s = \sqrt{\frac{I_s}{A_s + wt_{sp}}} \qquad \text{Eq. 33}$$

and substituting for the modulus of elasticity of steel, the corresponding requirement on r_s/a is

$$\frac{r_s}{a} \geq \sqrt{\frac{a}{b}}\sqrt{\frac{1}{15{,}470\left(A_s + wt_{sp}\right)} + \frac{a}{26{,}180{,}000}} \qquad \text{Eq. 34}$$

or

$$\frac{a}{r_s} \leq \frac{1}{\sqrt{\frac{a}{b}}\sqrt{\frac{1}{15{,}470\left(A_s + wt_{sp}\right)} + \frac{a}{26{,}180{,}000}}} \qquad \text{Eq. 35}$$

It is informative to note that the first term under the radical in the denominator of this expression comes from the concentrated applied load of 0.3 kips and the second term comes from the self-weight of the stiffener and the stiffened plate tributary to the stiffener. After multiplying the denominator and numerator by $\sqrt{wt_{sp}}$ and performing some additional algebraic manipulation, this equation may be expressed as

$$\frac{a}{r_s} \leq \frac{t_{sp}\sqrt{\frac{w}{t_{sp}}}}{\sqrt{\frac{a}{b}}\sqrt{\frac{1}{15{,}470}\frac{wt_{sp}}{\left(A_s + wt_{sp}\right)} + \frac{awt_{sp}}{26{,}180{,}000}}} \qquad \text{Eq. 36}$$

In addition, recognizing that for equal stiffener spacing $b = (n + 1)\,w$, the second term under the radial within the denominator can be written in a normalized form as shown in the following:

$$\frac{a}{r_s} \leq \frac{t_{sp}\sqrt{\frac{w}{t_{sp}}}}{\sqrt{\frac{a}{b}}\sqrt{\frac{1}{15{,}470}\frac{wt_{sp}}{\left(A_s + wt_{sp}\right)} + \frac{(n+1)}{26{,}180{,}000}\frac{a}{b}\left(\frac{w}{t_{sp}}\right)^2 t_{sp}^3}} \qquad \text{Eq. 37}$$

After factoring the first term outside of the radical in the denominator to this expression, one obtains

$$\frac{a}{r_s} \le \frac{124 t_{sp} \sqrt{\dfrac{w}{t_{sp}}}}{\sqrt{\dfrac{a}{b}} \sqrt{\dfrac{w t_{sp}}{\left(A_s + w t_{sp}\right)} + \dfrac{(n+1)}{1690} \dfrac{a}{b} \left(\dfrac{w}{t_{sp}}\right)^2 t_{sp}^3}} \qquad \text{Eq. 38}$$

The ratio in the first term under the radical in the denominator of the above expression may be taken approximately as 1/1.2. The largest value of $(A_s + wt_{sp})/wt_{sp}$ from representative designs is approximately 1.6, and for typical tub girders with wide longitudinally stiffened bottom flanges, typical maximum values are near 1.2. In the following assessments, $wt_{sp}/(A_s + wt_{sp})$ in Eq. 38 is taken as 1/1.2 to obtain a conservative (lower-bound) limit on a/r_s needed to restrict the out-of-plane deflections under self-weight plus a 0.3 kip concentrated load to $b/300$.

The results from Eq. 38 may be tabulated for a complete range of parameters $t_{sp} \ge 0.5$ inches, $w/t_{sp} \le 120$, $n \ge 1$, and a/b_{sp} from relatively small values to values corresponding to a practical maximum a dimension of 40 ft. Figure 19 shows the most restrictive a/r_s limits from Eq. 38 as a function of t_{sp} and w/t_{sp}. These results have the following characteristics:

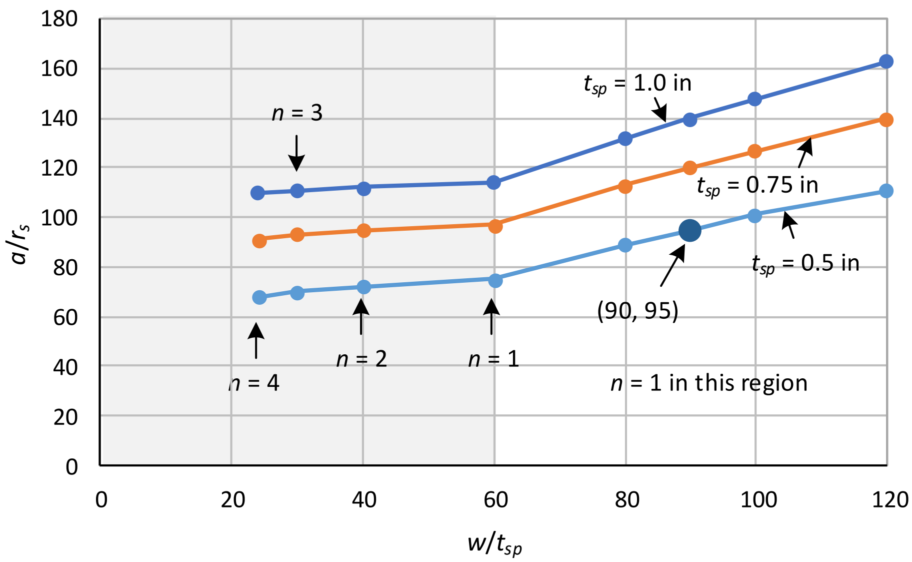

Figure 19. Plot of the most restrictive a/r_s limits from Eq. 38 as a function of t_{sp} and w/t_{sp}, based on a = 40 ft; the gray area corresponds to characteristics for which the plate satisfies the out-of-plane deflection limit of b/300 without the addition of a longitudinal stiffener.

1. For values of $w/t_{sp} \ge 60$, the most restrictive limits on a/r_s correspond to the use of a single longitudinal stiffener (n = 1) as well as the maximum practical longitudinal stiffener unsupported length of a = 40 ft.
2. For values of $w/t_{sp} \le 60$, the most restrictive limits on a/r_s correspond to w/t_{sp} values such that $b/t_{sp} = w\,(n + 1) / t_{sp} = 120$. The data points on the curves at w/t_{sp} = 60, 40, 30 and 24 correspond to n = 1, 2, 3 and 4 respectively. The reason for the correspondence with b/t_{sp} = 120 is that the plates generally are adequate by themselves to limit the out-of-plane deflection to less than $b/300$ under self-weight plus a concentrated load of 0.3 kips placed at the mid-length of the longitudinal stiffener, without longitudinal stiffening, when b/t_{sp} becomes less than or equal to this value. This is illustrated previously by Fig. 12. It should be recalled that Eq. 38 neglects the contribution from the plate to the transverse

stiffness; it focuses on the longitudinal stiffener alone as the source of stiffness to limit the above deflections. For the data points within the gray shaded area of the plot, the plates alone are adequate to limit the out-of-plane deflections to the target value under the specified loading.

3. For $t_{sp} \geq 1.0$ inches, Eq. 38 indicates that for essentially all cases, a limit of 120 on a/r_s is sufficient to restrict the transverse deflection under the above loads to a value less than $b/300$. The limit $a/r_s < 120$ was originally recommended by Wolchuk and Mayrbaurl (1980).
4. Generally, the most restrictive requirements on a/r_s occur for the smallest considered thickness, $t_{sp} = 0.5$ inches.

It should be noted that for a values smaller than 40 ft, and for n values larger than those indicated in the plot, the minimum required a/r_s from Eq. 38 is larger than the values shown in Fig. 19.

One can observe from the above results that for $t_{sp} < 1.0$ inches, Eq. 38 gives a required limit on a/r_s that is smaller than 120 in many situations. However, again, the contribution of the plate to the transverse stiffness is neglected in Eq. 38.

It would be much simpler to specify a basic maximum limit on a/r_s, for example $a/r_s \leq 120$ as recommended by Wolchuk and Mayrbaurl (1980), rather than requiring the use of a more complex equation such as Eq. 38 to ensure that the plate out-of-plane service deflections are within the targeted limits. The following specific geometries were evaluated to ascertain whether the simple limit of $a/r_s \leq 120$ would be accurate to conservative when the contribution of the plate to the out-of-plane stiffness is considered:

1. For $w/t_{sp} = 60$ with $a = 40$ ft, $t_{sp} = 0.5$ inches and $n = 1$, Eq. 38 gives a maximum limit on a/r_s of 75. However, as noted above, this plate is sufficient to restrict the out-of-plane deflections to less than the targeted values without any longitudinal stiffening.
2. For $w/t_{sp} = 90$ with $a = 40$ ft, $t_{sp} = 0.5$ inches and $n = 1$, Eq. 38 gives a maximum limit on a/r_s of 95. This data point is highlighted in Fig. 19. Separate refined analysis checks of this case indicate that the deflection of the stiffened plate is actually smaller than $b/300$ when a/r_s is limited to 120 instead of 95.
3. The case with $w/t_{sp} = 60$, $a = 40$ ft., $t_{sp} = 0.5$ inches and $n = 4$ (corresponding to $b/t_{sp} = 60(4 + 1) = 300$) was checked as a geometry where the contribution from the plate in resisting the out-of-plane deflections would be anticipated to be small. Equation 38 gives $a/r_s \leq 119$, essentially equal to 120, for this case. Since the plate out-of-plane deflections are limited to less than $b/300$ for this case, as well as for case 1 with $w/t_{sp} = 60$ with $a = 40$ ft, $t_{sp} = 0.5$ inches and $n = 1$, one can expect that the plate out-of-plane deflections will be adequate for $a = 40$ ft., $t_{sp} = 0.5$ inches, and $w/t_{sp} = 60$ combined with $n = 2$ and 3.

A number of other selected geometries were evaluated by the project team to check the above results.

Given these studies, it can be concluded that

$$a/r_s \leq 120 \qquad \text{Eq. 39}$$

is sufficient to ensure that the out-of-plane deflections due to self-weight plus a small concentrated load of 0.3 kips are less than or equal to $b/300$ on longitudinally stiffened plates in all situations, subject to $a \leq 40$ ft. and $t_{sp} \geq 0.5$ inches. It should be noted that this limit should not be applied to box-section member webs, since these webs are typically not subjected to the loadings considered above. In addition, in cases where longitudinal stiffeners are provided on flange plates in which $b/t \leq 120$, the flange plate stiffness is sufficient to restrict the out-of-plane deflections to $b/300$ without longitudinal stiffening, and therefore the above restriction on a/r_s is not needed. It is recommended that $b/t \leq 90$ be used instead as this limit, since 90 is the recommended maximum width-to-thickness ratio for longitudinally unstiffened flanges.

For cases with a single longitudinal stiffener, which have been identified in the above as producing larger demands on the longitudinal stiffener than cases with $n > 1$, it is possible to solve the above problem as a beam on an elastic foundation. This allows the consideration of the resistance to out-of-plane deformations from the "strips" across the width of the plate (still neglecting the ability of the plate itself to develop shear forces associated with bending in the longitudinal direction). Young and Budynas (2002, 213-220) provide tabulated equations and constants to facilitate this type of analysis. However, the beam on elastic foundation solution is significantly more complex than the above basic solution.

4.9 Requirements to Limit the Out-of-Plane Deflection of Transverse Stiffeners Under Self-Weight Plus a Concentrated Load

The load-deflection criterion proposed in Sections 4.5 and 4.8 also should be satisfied at transverse stiffeners in longitudinally stiffened plates. Figure 20 shows the model for this calculation.

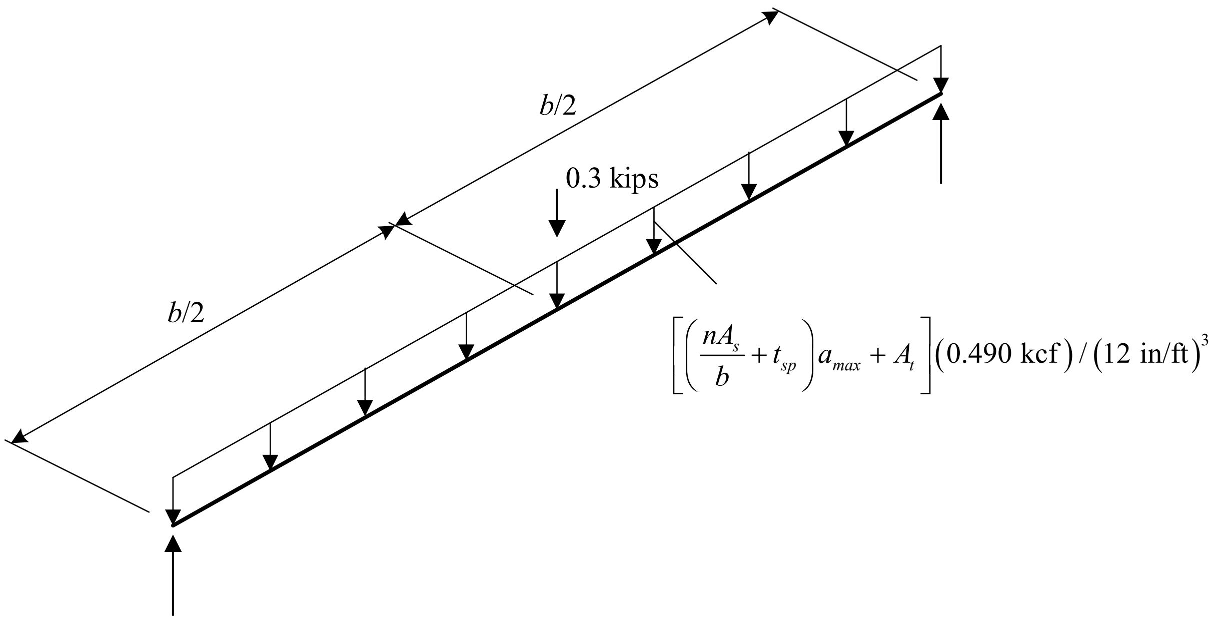

Figure 20. Model of a simply supported bottom flange transverse stiffener subjected to a 0.3 kip load at its mid-length, as well as its self-weight and the tributary self-weight of the longitudinal stiffeners and plate.

The deflection criterion is

$$\delta = \frac{b^3}{EI_t}\left[\frac{0.3\text{ kip}}{48} + \frac{5}{384}\left[\left(nA_s + bt_{sp}\right)a_{max} + bA_t\right](0.490\text{ kcf})/(12\text{ in/ft})^3\right] \le \frac{b}{300} \qquad \text{Eq. 40}$$

where I_t is the moment of inertial of the transverse stiffener, A_t is the area of the transverse stiffener, and a_{max} is taken conservatively as the larger of the longitudinal spacings to the next transverse stiffening element on each side of the transverse stiffener. The other variables in this equation have been defined previously. Eq. 40 can be solved to obtain the following equation for the required transverse stiffener moment of inertia:

$$I_t \ge \frac{b^2}{E}\left[1.875\text{ kip} + \left(\frac{1}{900}\text{kip/in}^3\right)\left[\left(nA_s + bt_{sp}\right)a_{max} + bA_t\right]\right] \qquad \text{Eq. 41}$$

It is informative to consider the ratio of this moment of inertia requirement to the required moment of inertia of the longitudinal stiffener struts:

$$\frac{I_{t.min}}{I_{s.min}} \geq \frac{b^2\left[1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left[\left(nA_s+bt_{sp}\right)a_{max}+bA_t\right]\right]}{a_{max}^2\left(\frac{a_{max}}{b}\right)\left[1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left(A_s+\frac{b}{n+1}t_{sp}\right)a_{max}\right]} \qquad \text{Eq. 42}$$

The subscripts "min" are added in the ratio of these variables to emphasize that this is a ratio of the minimum moment of inertia requirements, not a required ratio of the actual transverse stiffener moment of inertia to the longitudinal stiffener strut moment of inertia. Upon multiplying the numerator and denominator of this equation by (n + 1) and simplifying, one obtains

$$\frac{I_{t.min}}{I_{s.min}} \geq \left(\frac{b}{a_{max}}\right)^3 \frac{\left[1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left[\left(nA_s+bt_{sp}\right)a_{max}+bA_t\right]\right](n+1)}{\left[(n+1)1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left((n+1)A_s+bt_{sp}\right)a_{\max}\right]} \qquad \text{Eq. 43}$$

If the first occurrence of n in the numerator is modified conservatively to (n + 1), and if the first occurrence of (n + 1) in the denominator is modified conservatively to 1.0, one obtains

$$\frac{I_{t.min}}{I_{s.min}} \geq \left(\frac{b}{a_{max}}\right)^3 \frac{\left[1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left[\left((n+1)A_s+bt_{sp}\right)a_{max}+bA_t\right]\right](n+1)}{\left[1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left((n+1)A_s+bt_{sp}\right)a_{\max}\right]} \qquad \text{Eq. 44}$$

Finally, if the weight of the transverse stiffener itself is assumed to be smaller than the additional weight included in the numerator by changing the first occurrence of n to (n + 1), and therefore, the term bA_t is incorporated into the additional $A_s a_{max}$ associated with the above modification, this equation may be written as

$$\frac{I_{t.min}}{I_{s.min}} \geq \left(\frac{b}{a_{max}}\right)^3 \frac{\left[1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left[\left((n+1)A_s+bt_{sp}\right)a_{max}\right]\right](n+1)}{\left[1.875\text{ kip}+\left(\frac{1}{900}\text{kip/in}^3\right)\left((n+1)A_s+bt_{sp}\right)a_{\max}\right]} \qquad \text{Eq. 45}$$

Therefore, to ensure that the transverse stiffeners satisfy the recommended deflection criterion, the ratio of the minimum required transverse stiffener moment of inertia to the minimum required longitudinal stiffener moment of inertia may be taken as

$$\frac{I_{t.min}}{I_{s.min}} \geq \left(\frac{b}{a_{max}}\right)^3 (n+1) \qquad \text{Eq. 46}$$

For typical longitudinally stiffened plates, where b is significantly smaller than a_{max} and n is small, this criterion is less demanding than the criterion stated in Article 10.39.4.3.5 of AASHTO (2002, 272), listed at the end of Table 2, assuming that "equal size" is intended to mean equal moment of inertia and assuming that the longitudinal stiffeners are actually sized to the minimum requirements. However, for cases where b/a_{max} becomes close to 1.0, or for unusual cases where b/a_{max} is greater than 1.0, and if the longitudinal stiffeners are actually sized to the minimum requirement developed in Section 4.8, the criterion given by Eq. 46 may be more demanding than the AASHTO (2002, 272) rule. Fortunately, the a/r_s requirement given by Eq. 39 often satisfies the corresponding load-deflection criterion conservatively. Furthermore, $b/a_{max} \leq 1.0$ is a more specialized design case typically involving wide plates having a large number of longitudinal stiffeners. Therefore, in the judgment of the project team, it is sufficient to simply require, as a minimum, that the moment of inertia, I_t, of the transverse stiffeners should be greater than or equal to the moment of inertia, I_s, of the longitudinal stiffeners, within the Specifications. This simple requirement is implied by AASHTO (2002, 272). In addition, it is recommended that a statement be placed in the commentary of the corresponding AASHTO LRFD article to refer to the above discussion in this report for specialized design cases in which b/a_{max} is close to or greater than 1.0.

It should be pointed out that in the limit where both I_t and I_s are set equal to the above derived minimum values, the theoretical maximum out-of-plane deflection in the stiffened plate will be $2(b/300) = b/150$ relative to its longitudinal edges. However, the various conservative assumptions employed in the corresponding derivations and simplified recommendations are such that the actual deflections under the targeted service loadings will tend to be smaller than $b/300$.

4.10 Breathing of Bottom Flange Plates under Cyclic Tension

As noted by Respondent 5, extremely slender bottom flanges in box girders can be subject to plate breathing under cyclic tension, i.e., out-of-plane bowing in the flanges from initial fabrication imperfections and from self-weight deflection being cyclically straightened out and released, causing bending moments to be developed in the plate. Therefore, it is considered prudent to evaluate the extent of these actions for the ideal case of a simply supported plate subjected to its self-weight and cyclic uniaxial tension.

Young and Budynas (2002, 504) summarize a solution to this problem originally developed by Conway (1949). In the current study, this solution is implemented for a simply supported plate with $a/b = 4$, the largest plate aspect ratio presented by Young and Budynas. The equations summarized by Young and Budynas are configured in this study to compare the maximum out-of-plane deflection and the maximum plate bending stress in the width direction, at the mid-width of the plate for longitudinal axial tension stresses of 10, 20 and 30 ksi, to the corresponding deflections and plate bending stresses due to self-weight under zero axial tension.

Young and Budynas characterize the axial tension in the plate as a multiple of the compressive buckling stress given by Eq. 3 with $k = 4$, but with the load applied in tension, that is, they list solutions for the out-of-plane deflection and plate bending stress at tensile axial stress levels of γF_{cr} for $\gamma = 1, 2, 3, 4$ and 5. For the purpose of the present study, these axial tension values are set equal to applied axial tension stresses of $f_a = 10$, 20 and 30 ksi. As such, given the above equation, the b/t value corresponding to a given $\gamma F_{cr} = f_a$ is obtained as

$$\frac{b}{t} = \sqrt{\gamma \frac{E}{f_a} \frac{\pi^2}{3\left(1-\nu^2\right)}} \qquad \text{Eq. 47}$$

Given these b/t values, the normalized maximum out-of-plane deflection of the plate is calculated from the formula

$$\frac{\delta}{b} = \alpha \frac{q}{E}\left(\frac{b}{t}\right)^3 = \alpha \frac{\left[t\left(0.490\text{ kcf}\right)/\left(12\text{ in/ft}\right)^3\right]}{29{,}000\text{ ksi}}\left(\frac{b}{t}\right)^3 = 9.778\times10^{-9}\alpha t\left(\frac{b}{t}\right)^3 \qquad \text{Eq. 48}$$

and the maximum bending stress in the width direction of the plate is calculated as

$$\sigma_b = \beta q\left(\frac{b}{t}\right)^2 = \beta\left[t\left(0.490\text{ kcf}\right)/\left(12\text{ in/ft}\right)^3\right]\left(\frac{b}{t}\right)^2 = 2.836\times10^{-4}\beta\left(\frac{b}{t}\right)^2 \qquad \text{Eq. 49}$$

The solutions for α and β, from Young and Budynas (2002, 504) are listed in Table 4.

Table 4. Coefficients α and β for plate subjected to uniform out-of-plane distributed load and uniform axial tension (Young and Budynas 2002, 504).

Coeff. \ f_a / F_{cr}	0	1	2	3	4	5
α	0.140	0.118	0.102	0.089	0.080	0.072
β	0.741	0.624	0.540	0.480	0.420	0.372

Figure 21 plots the solutions for δ/b versus b/t for the minimum plate thickness of t = 0.5 inches recommended in the prior discussions. The dashed top curve shows the normalized deflections for zero axial tension, which is also given by Eq. 19. The second highest curve in the plot corresponds to f_a = 10 ksi. The lowest two plots correspond to f_a = 20 and 30 ksi. The right-most point on each of the curves associated with $f_a > 0$ (in tension) corresponds to $f_a = 5.0F_{cr}$ and the left-most point on each of these curves corresponds to $f_a = 1.0F_{cr}$. The largest difference between the f_a = 0 ksi and f_a = 10 ksi curves corresponds a cyclic movement between δ/b = 0.00821 and 0.00422 at b/t = 229. That is, this is a 229 x 0.5 = 114.5 inch wide, ½ inch thick plate that deflects under its self-weight by 0.00821 x 114.5 = 0.940 inches when the axial tension is zero, but in which the plate out-of-plane deflections are reduced to 0.00422 x 114.5 = 0.483 inches under an axial tension of 10 ksi. Clearly, the 0.940 – 0.483 = 0.457 inch change in the out-of-plane deflection will be visible under the cyclic application of this axial tension stress.

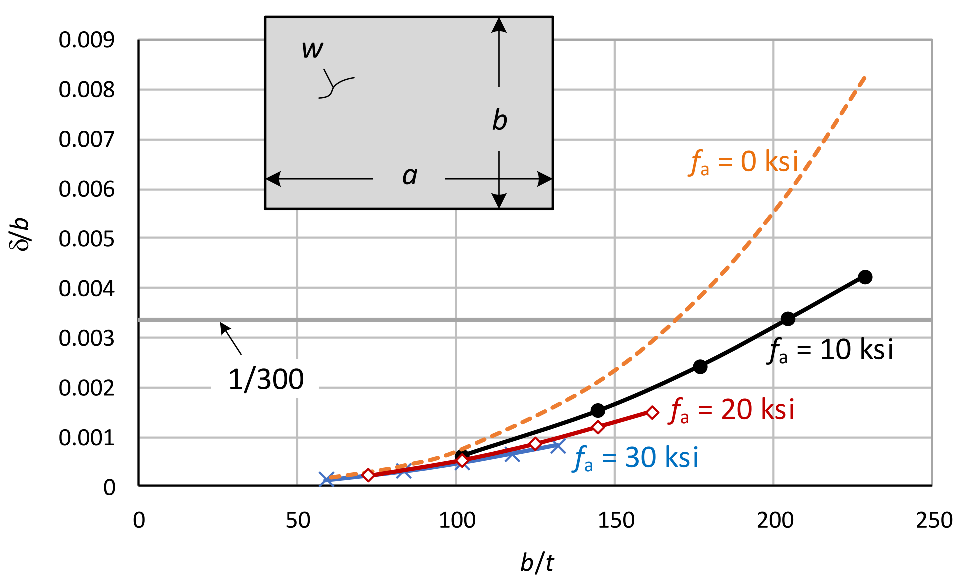

Figure 21. Maximum simply supported plate normalized out-of-plane deflection, δ/b, due to self-weight (1/2 inch thick plate) for several different values of longitudinal axial tension in the plate, plotted versus b/t, and compared to the limit 1/300.

It can be observed that these self-weight deflections are substantially reduced for a ½ inch thick plate with b/t = 120 (i.e., b = 60 inches). The solutions for f_a = 20 and 30 ksi exhibit a larger change in deflection relative to the dashed top curve for f_q = 0 ksi; however, the solutions listed by Young and Budynas (2002, 504) stop at an axial tension equal to five times the buckling load in compression. These curves can be visually extrapolated to the largest b/t value of 229 shown in the plot, suggesting displacement ranges of approximately 1.5 and 2.0 inches for cyclic tension values ranging from zero to 20 and 30 ksi respectively.

It should be noted that for a given b/t, the normalized out-of-plane deflections due to self-weight, δ/b, are larger for thicker plates (see Eq. 19). Therefore, the above normalized cyclic displacements will actually be larger when considering plates with the same b/t that have a plate thickness larger than ½ inches.

The corresponding plate bending stresses in the width direction of the plate are shown in Figure 22. One can observe that the 114.5 x ½ inch plate has a plate bending stress of 5.51 ksi when $f_a = 0$ ksi, and that this stress is reduced to 2.76 ksi for $f_a = 10$ ksi. Therefore, the cyclic change in stress is 5.51 – 2.76 = 2.75 ksi. By extrapolation of the solutions for $f_a = 20$ ksi and 30 ksi for the largest value of $b/t = 229$ shown in the plot, the cyclic changes are approximately 4 ksi and 5 ksi respectively.

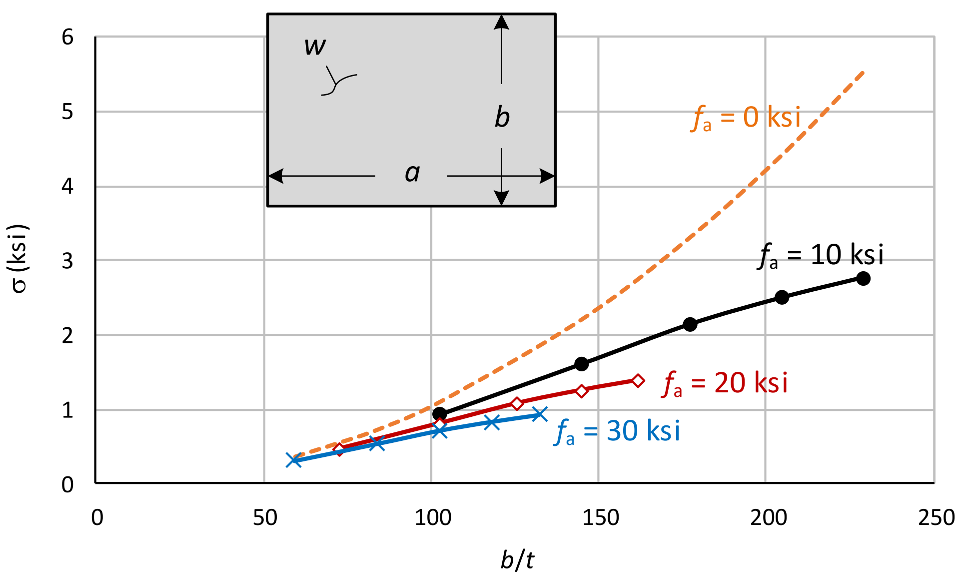

Figure 22. Maximum simply supported plate bending stress in the direction of the width, σ, due to self-weight (1/2 inch thick plate) for several different values of longitudinal axial tension in the plate, plotted versus b/t.

It should be noted that the above solutions are heavily idealized. For instance, the actual maximum plate bending stresses are apt to occur at the panel boundary with a transverse stiffener or diaphragm. However, these solutions show the tendency for the out-of-plane deflections under self-weight to be relatively large, and the changes in these deflections under cyclic tension to be noticeable, once the plate b/t becomes relatively extreme. Limiting the plate b/t to 120 or less prevents this issue.

CHAPTER 5

Synthesis of Practice and Issues, and Recommendations for AASHTO Specifications and Guidelines

5.1 Synthesis of Existing Practice and Issues, and Overview of Corresponding Recommendations

Based on the survey of owners and subject matter experts, literature review and analytical studies, the following conclusions can be drawn regarding not-to-exceed or not-to-be-lesser-than dimensional limits for bottom flanges of steel box girders:

Regarding the minimum flange plate thickness:

- $t \geq$ ½ inch is a rational absolute minimum requirement, unless otherwise approved by an Owner. Steel box girder flanges with t less than this value are apt to have increasing sensitivity to welding distortions and deflections under self-weight and small concentrated applied loads. Smaller thicknesses are common, however, for box-section members employed in long-span bridge construction, where the expense associated with handling and control of distortion in thin stiffened plates is justified by the savings in weight.
- Regarding thickness requirements to facilitate fabrication and erection, it is clear from the surveys and from the literature that generally thicker is better, particularly when considering potential welding distortion. However, specific quantification of the effects of a thickness limit on ease of fabrication and/or erection is somewhat elusive due to the many factors involved. Key considerations from Chapters 2, 3 and 4 may be summarized as follows:
  - One of the survey respondents cited a limit of $t > 5/8$ inch to accommodate threaded holes for drain pipes, and to avoid problems with "manutention" (i.e., handling) before assembly.
  - AASHTO (2017) LRFD Article 6.7.3 specifies a thickness limit of 5/8 inches for orthotropic decks. However, the bottom flange of a box section is not subjected to the same types of demands that a deck plate is subjected to.
  - Multiple survey respondents recommended $t >$ ¾ inch to facilitate the control of welding distortion, and generally to facilitate fabrication and erection. However, it is clear from the surveys that numerous bridges exist in service that have smaller thicknesses, and therefore it does not appear that violating a thickness limit of even ½ inches will in itself lead to performance problems during the life of a box-girder bridge.
  - Thicknesses as small as 9 mm = 0.35 inches combined with $w/t < 90$ are common and are not a problem for longitudinally stiffened flanges in box-section members employed in long-span bridge construction, where the expense associated with handling and control of distortion in thin stiffened plates is justified by the savings in weight. Plates this thin require attention to established

procedures necessary to control welding distortion, but with $w/t < 90$, buckling distortion (i.e., oil canning and waviness of the plates) due to welding is expected to be very controllable.

  - It should be noted that if the flange plate is longitudinally stiffened, particularly if two or more longitudinal stiffeners are employed, the overall b/t of the plate will tend to be significantly larger than 90. Therefore, the plate will tend to be sensitive to buckling distortion due to welding residual stresses, and it is likely that clamping and restraining of the plate and other control measures will need to be applied to limit welding distortions.

- It is advisable for the compression and tension flange thicknesses corresponding to the box section principal axis direction with the largest bending moment never to be smaller than the thickness of the member webs. Girders with $t_f < t_w$ will tend to have plate bending stresses due to the distortion of the box-section under torsional loads that are larger in the flanges than in the webs.

Regarding the maximum flange plate slenderness:

- A width-to-thickness ratio limit of $b/t \leq 90$ is advisable for longitudinally unstiffened flanges. A limit of $w/t \leq 90$ is advisable for longitudinally stiffened flanges. Flanges with larger b/t (unstiffened) or w/t (stiffened) values are apt to exhibit noticeable oil canning or waviness due to welding residual stresses with or without a small applied axial compression.

The following behavioral considerations are correlated approximately with the following b/t or w/t limits:

- For welded box sections with a b/t (without longitudinal stiffening) or w/t (with longitudinal stiffening) of the flange plates greater than 100, the fabricator may need to be particularly cautious not to over-weld (i.e., to avoid providing welds that are larger than required for strength and/or minimum size requirements) and to appropriately restrain the plate during welding. Some noticeable distortion of the flange plate may occur due to placement of typical minimal welding of the flange to the webs and/or the welding of any stiffeners to the flange plate if these limits are exceeded.
- Box flanges with b/t or w/t values larger than about 130 will have difficulty maintaining less than 1/300 out-of-plane deflection under self-weight, or under self-weight with a small concentrated transverse load; therefore, plate out-of-plane deflections due to these nominal loads may be noticeable. Therefore, box flanges subject to tension with b/t values exceeding 130 are required to have longitudinal stiffeners.
- Dynamic excitation of a box-section member flange in plate bending can be a potential issue in boxes with b/t or w/t greater than about 210.
- Box section flanges with b/t greater than about 210 may be susceptible to fatigue damage due to plate breathing under cyclic tension, with the out-of-plane bow in the flanges from initial fabrication imperfections and from self-weight deflection being cyclically straightened out and released, causing bending stresses to be developed in the plate.
- It should be noted that the strength of box section flanges subjected to compression becomes smaller relative to F_y as b/t or w/t becomes relatively large. For welded longitudinally unstiffened box-section flanges, the ultimate strength of the plate is approximately $0.8F_y$ at $b/t = 40$, $0.6F_y$ at $b/t = 60$, and $0.4F_y$ at $b/t = 90$. These considerations are addressed specifically in broader recommended provisions for design of box-section members (White, et al. 2019), which have been balloted and approved for the 9th Edition of the AASHTO LRFD Specifications at the time of the writing of this final report (July 2019).

Regarding flange extensions in box-section members:

- A minimum extension of 1 inch is commonly recommended, but with a fabricator option to increase for welding access (AASHTO/NSBA 2016, 42).
- Wolchuk and Mayrbaurl (1980) recommended a maximum width-to-thickness of 20 for the tension flange extensions beyond the web as a reasonable practical maximum limit. However, this limit appears to be relatively arbitrary, and engineers are not likely to use values this large. The project team recommends that the b/t of the tension flange extensions be limited to 12.0, which is the current practical upper limit on the projecting flange width for I-girder flanges specified in AASHTO Article 6.10.2.2 and for the top flanges of composite tub-girders in AASHTO Article 6.11.2.2.
- A smaller limit equal to the compactness requirement for a flange plate supported only on one longitudinal edge is recommended for flange extensions on compression flanges in box-section flexural members. This ensures that F_y can be developed on the full width of the flange extension in compressive strength calculations. If the flange extension is larger than this value, the compressive strength calculations may be performed neglecting the width of the plate wider than the compact limit. This is a simple rule that avoids the need for complex consideration of effective widths due to local buckling, etc. on flange extensions.

Regarding the overall slenderness of longitudinal stiffeners placed in box-section member tension flanges:

- It is recommended that flange longitudinal stiffeners should satisfy $a/r_s \leq 120$ when the overall b/t of the flange is greater than 90. This limit ensures against excessive out-of-plane deflection of a longitudinally stiffened plate under the self-weight of the plate plus a small transverse concentrated load.

Finally, for transverse stiffeners:

- A simple minimum requirement (where other Specification requirements do not govern), is that the transverse stiffeners should have a moment of inertia, I_t, greater than or equal to the moment of inertia, I_s, of the longitudinal stiffeners. This is judged sufficient to ensure against excessive out-of-plane deflection of a stiffened plate under its self-weight plus a small transverse concentrated load for most designs. Commentary language is recommended, referencing this report for further discussion, for specialized cases where the flange width is close to or greater than the maximum longitudinal spacing to the adjacent transverse stiffening elements at a given transverse stiffener.

The following are two additional box girder bottom flange limits gleaned from the literature review conducted in this research:

- From FDOT (2018, 5-7), "Design each box girder with minimum 2-inch diameter ventilation or drain holes located in the bottom flange on both sides of the box spaced at approximately 50 feet or as needed to provide proper drainage. Place drains at all low points against internal barriers."
- From TSQC (2015, 2-17), "If using longitudinal stiffeners, try to maintain a clear distance between longitudinal stiffeners of no less than 24 inches (more is better) to accommodate automated welding equipment. Therefore, the minimum flange width, between webs, is 48 inches when using one stiffener and 72 inches for two stiffeners."

The first of these recommendations is a useful requirement for boxes that are not sealed; however, this requirement would of course not apply to sealed boxes and therefore is not recommended for inclusion in the AASHTO LRFD Specifications or Commentary. The second recommendation is also an important consideration that can facilitate fabrication of longitudinally stiffened plates. However, it is viewed to be more of design guidelines nature and therefore is not recommended for inclusion in the AASHTO LRFD Specifications and Commentary.

Based on the survey responses, the literature review, and the design experience within the NCHRP project team, it is considered preferable in some situations not to stop bottom flange longitudinal stiffeners within the tension-only regions of a box girder. The following behavioral aspects should be considered in this regard:

1. Tension flanges may be subjected to at least some minor compression at some stage during the life of the bridge; e.g., due to stress reversal near points of dead-load contraflexure in continuous-span flexural members. In such cases, the longitudinally unstiffened flange must be checked to ensure it has sufficient compressive resistance.
2. The termination of longitudinal stiffeners can be problematic in terms of the fatigue design, as well as with respect to inspection of the bridge during its service life (e.g., it may be difficult to determine whether apparent cracks observed at such a location cracks in the steel, or cracks in the paint).
3. During construction, when cantilevering the girder out or when lifting a girder, the zone of the bottom flange near the mid-span of the girder in its final constructed condition potentially may be subjected to significant compression due to self-weight.
4. On a wide, thin bottom flange plate, the longitudinal stiffeners may be important to avoid unsightly sagging, or drooping, of the bottom flange under its self-weight.

5.2 Recommendations for AASHTO LRFD Specifications

Given the above findings, it is recommended that Article 6.12.2.2.2b of the 9th Edition AASHTO LRFD provisions, which were balloted and approved by the AASHTO Committee on Bridges and Structures in June 2017, should be adopted (where no changes are required) and updated as follows. Only the portions of this article directly related to the above findings are shown below. The references to White et al., 2019b are intended as references to this final report. The Articles 6.12.1.2.4, 6.12.2.2.2f, and E6 referenced in these provisions are the article numbers of additional recommended 9th Edition AASHTO LRFD provisions discussed by White et al. (2019). At the time of the writing of this final report (July 2019), these additional 9th Edition provisions have been balloted and approved by the AASHTO Committee on Bridges and Structures, including the recommendations shown below.

6.12.2.2.2b Cross-Section Proportion Limits … Longitudinally unstiffened compression and tension flanges <u>should</u> be proportioned such that: $b_{fi}/t_f \leq 90$ (6.12.2.2.2b-3) where: b_{fi} = inside width of the box section flanges; for welded box sections, the clear width of the flange under consideration between the webs. For HSS, the provisions of Article 6.12.1.2.4 shall apply (in.)	*C6.12.2.2.2b* … Article 6.12.2.2.2b specifies a number of broad not-to-exceed or not-to-be-smaller-than limits on the proportions of box-section members. Various specific design criteria may require dimensions that are more restrictive than these maximum or minimum limits. Plates subjected to significant uniform compression stresses at the fatigue and/or service limit states, or during construction, will tend to be limited to $b_{fi}/t_{f\cancel{c}}$ significantly less than 90 or w/t_f less than 90, as applicable, and D/t_w less than 150 or 300, as applicable, by the provisions of Article 6.12.2.2.2f.

t_f = for welded box sections, the thickness of the flange under consideration. For HSS, the provisions of Article 6.12.1.2.4 shall apply (in.)

The thickness of the compression and tension flanges corresponding to the box section principal axis subjected to the larger bending moment should not be less than the thickness of the webs. The thickness of compression and tension flanges shall not be less than 0.5 in., unless otherwise specified by the Owner.

Compression flanges exceeding the limit given by Eq. 6.12.2.2.2b-3 shall include longitudinal stiffeners. Tension flanges with b_{fi}/t_f exceeding 130 shall include longitudinal stiffeners. Longitudinally stiffened flanges should be proportioned such that:

$$w/t_f \leq 90 \qquad (6.12.2.2.2b\text{-}4)$$

where:

w = widths of the flange plate between the centerlines of the individual longitudinal stiffeners and/or between the centerline of a longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of a longitudinally stiffened plate element (in.)

…

Flange longitudinal stiffeners shall satisfy the requirements of Article E6.1.3. Transverse stiffeners, when utilized to strengthen or stiffen a longitudinally stiffened flange shall satisfy the requirements of Article E6.1.4.

…

Flange extensions on compression flanges of_box sections shall be proportioned such that:

The limits of $b_{fi}/t_f \leq 90$, $w/t_f \leq 90$ and $t_f \geq 0.5$ inches are recommended to limit potential local deformation or distortion of box section flanges during fabrication, transportation, erection, and service conditions. Flanges violating these limits may have out-of-plane plate deflections approaching a significant fraction of $b_{fi}/300$ or $w/300$ under their self-weight plus a transverse concentrated load of 0.3 kips, which is considered by ASCE 7-16 (ASCE, 2016) as a visible deflection limit. These limits also help alleviate significant buckling distortion due to welding residual stresses resulting in oil canning and waviness of the flange, and the amplification of these distortions by small unintended axial compressive and/or shear stresses under service conditions, such as shifting of true inflection point locations from nominal positions calculated in the design (White et al., 2019b).

For box sections with b_{fi}/t_f or w/t_f of the flange plates greater than 100, some noticeable buckling distortion of the flange plate may occur during fabrication due to placement of typical minimal welding of the flange plate to the webs and/or welding of any stiffeners to the flange plate. In addition, the nominal resistance of compression flanges is relatively small as b_{fi}/t_f or w/t_f exceeds 100. Box flanges with b_{fi}/t_f or w/t_f values larger than about 130 will have difficulty maintaining the $b_{fi}/300$ or $w/300$ out-of-plane deflection under self-weight, or under self-weight with a small concentrated transverse load; therefore, plate out-of-plane sagging due to these nominal loads may be noticeable.

The flange thicknesses corresponding to the box section principal axis direction with the largest bending moment should not be smaller than the corresponding web thicknesses. As such, the plate bending stresses due to distortion of the box section under torsional loads will tend to be larger in the webs than in the flanges.

For welded box sections, a minimum thickness of ¾ in. is recommended for component plates subjected to significant stress due to bending about a cross-section axis parallel to the plates. This suggested minimum thickness is intended to ensure robustness and resiliency of the member response, to facilitate handling, and to minimize distortion and possible cupping of the plates during welding. An absolute minimum thickness of 0.5 in. is specified, unless otherwise permitted by the Owner, to avoid increased sensitivity to welding distortions and deflections under self-weight and small concentrated applied loads. Smaller thicknesses are common, however, for box-section members employed in long-span bridge construction, where the expense associated with handling and control of distortion in thin stiffened plates is justified by the savings in weight.

…

$b/t_f \le 0.38\sqrt{\dfrac{E}{F_y}}$ (6.12.2.2.2b-6) where: b = clear projecting width of the compression flange under consideration measured from the outside surface of the web (in.)	Eq. 6.12.2.2.2b-6 limits the width-to-thickness ratio of compression-flange extensions in box sections such that these components are not subject to any strength reduction associated with local buckling under flexural and/or axial compression. As such, the full gross width of the extensions, measured from the outside surface of the box section webs, may be employed in the calculation of the effective and gross box-section properties.

Regarding the recommended minimum requirements for longitudinal stiffeners, it is recommended that a new Appendix E6.1.3 of the 9th Edition AASHTO LRFD provisions include the following requirements:

E6.1.3—Longitudinal Stiffeners … Longitudinal stiffeners in flanges with $b_{sp}/t_{sp} > 90$ generally shall satisfy: $a/r_s \le 120$ (E6.1.3-3) in which: r_s = radius of gyration of the stiffener strut about an axis parallel to the plane of the stiffened plate (in.) = $\sqrt{I_s/A_{gs}}$ (E6.1.3-4) where: a = longitudinal spacing between locations of transverse stiffeners or diaphragms that provide transverse lateral restraint to the longitudinally stiffened plate under consideration (in.) b_{sp} = total width of the longitudinally stiffened plate, taken as the inside distance between the plates providing lateral restraint to its longitudinal edges (in.) A_{gs} = gross area of an individual stiffener strut as defined in Article E6.1.2 (in.^{2}) I_s = moment of inertia of an individual stiffener strut as defined in Article E6.1.2 (in.^{4}) t_{sp} = thickness of the longitudinally stiffened plate under consideration (in.)	**CE6.1.3** … The limit $a/r_s \le 120$ ensures against excessive out-of-plane deflection of a stiffened plate under the self-weight of the plate plus a 0.3 kip concentrated load (White et al. 2019b). This limit is applied regardless of whether or not the member is in a horizontal configuration in the final constructed geometry, to limit such deflections with the member oriented horizontally during construction operations. This limit need not be satisfied in flanges with $b_{sp}/t_{sp} \le 90$ since the out-of-plane deformations due to the above load effects tend to be small in these cases without longitudinal stiffening.

Regarding the recommended minimum requirements for transverse stiffeners, it is recommended that a new Appendix E6.1.4 of the 9th Edition AASHTO LRFD provisions include the following requirements:

E6.1.4—Transverse Stiffeners	
E6.1.4.1--General	**CE6.1.4.1**
…	…
Transverse stiffeners generally should also have a moment of inertia, I_t, defined in Article E6.1.5.2 greater than or equal to the moment of inertia of the longitudinal stiffeners, I_s, defined in Article E6.1.3. Transverse stiffeners used to increase the compressive resistance of a longitudinally stiffened plate also shall satisfy the moment of inertia requirements specified in Article E6.1.5.2.	The minimum requirement that the transverse stiffener moment of inertia, I_t, be greater than or equal to the moment of inertia, I_s, of the longitudinal stiffeners ensures against excessive out-of-plane deflection of a stiffened plate under its self-weight plus a small transverse concentrated load in cases where the calculated axial compression in the plate is small, and therefore the requirements from Eqs. E6.1.4.2-1 and E6.1.4.2-2 are small (White et al. 2019b). One example of this situation is a case where transverse stiffeners are installed to serve only as points of termination of longitudinal stiffeners in a tension zone. In designs where b_{sp}/a_{max} is close to or greater than 1.0, where b_{sp} is the total width of the stiffened plate as defined in Article E6.1.4 and a_{max} is the largest of the longitudinal spacings to the adjacent transverse stiffeners or diaphragms providing lateral restraint to the plate, I_t may need to be larger than I_s to satisfy out-of-plane deflection criteria under service loads (White et al., 2019b)

In addition to the above recommendations, it is suggested that updates similar to those proposed for Article 6.12.2.2.2b should be implemented in Article 6.11.2.2, Flange Proportions, pertaining to composite box girder bottom flanges. As noted in Section 1.1 of this report, the present AASHTO LRFD Specifications do not specify any limits on the thickness or slenderness of these structural components. The thinking behind this recommendation is that AASHTO Committee and users of the Specifications would prefer repetition of the pertinent requirements separately in Article 6.11, as opposed to having Article 6.11 refer back to Article 6.12.2.2.2b for these requirements.

Regarding the requirements pertaining to longitudinal and transverse stiffeners recommended for inclusion in Articles E6.1.3.1 and E6.1.4.1, it is anticipated that AASHTO Article 6.11 will be updated at a future time to incorporate the advancements in design of longitudinally, or longitudinally and transversely stiffened plates adopted for the 9th Edition of the AASHTO LRFD Specifications. The recommendations of minimum longitudinal and transverse stiffener design limits for bottom flanges in composite box girders can be incorporated most easily into the Specifications when these updates to Article 6.11 are developed.

5.3 Recommendations Pertaining to Other AASHTO Design Guidelines

It is also recommended that various aspects pertaining to the findings of NCHRP 20-07/415 should be considered in future updates to the AASHTO/NSBA G12.1 document, *Guidelines to Design for Constructability*. Specifically, this document may wish to highlight a number of the findings from the NCHRP 20-07/415 research in a way that complements and elaborates upon the recommended Specification requirements, and the streamlined Commentary discussion of these requirements. For instance, the enumeration of the different values of b/t summarized above in Section 5.1 may be of particular value for designers. In addition, the guidance from FDOT (2018) and from TSQC (2015) summarized at the end of Section 5.1 may be useful for inclusion in the G12.1 document.

References

AASHTO. 1993. *Guide Specifications for Horizontally Curved Highway Bridges*, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Ed., American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2003. *AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges (LFD)*, American Association of State Highway Transportation Officials, Washington, DC.

AASHTO. 2017. AASHTO LRFD Bridge Design Specifications, 8th Ed., American Association of State Highway Transportation Officials, Washington, DC.

AASHTO/NSBA. 2016. *Guidelines to Design for Constructability*, G12.1-2016. American Association of State Highway Transportation Officials, Washington, DC, National Steel Bridge Alliance, Chicago, IL, and AASHTO/NSBA Steel Bridge Collaboration.

AISC. 2016. *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, American Institute of Steel Construction, Chicago, IL.

Air Products. 2019. Air Products offers advice about how to prevent and control welding distortion – a common problem affecting weld quality, http://www.airproducts.com/~/media/Files/PDF/industries/metals/metals-fabrication/metals-fabrication-how-to-prevent-control-welding-distortion.pdf/ (As of January 5, 2019).

ANSYS. 2019. *ANSYS nCode DesignLife, Predict Your Product's Fatigue Life Before You Build It*, https://www.ansys.com/products/structures/ansys-ncode-designlife, ANSYS Software System (As of January 5, 2019).

Asadnia, F. 2018. *Out-of-Flatness Plate Tolerance for Steel I-Shaped and Tub Highway Bridge Plate Girders*, Doctoral Dissertation, The School of Engineering and Applied Science, The George Washington University, Washington, DC.

ASCE. 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

BSI. 1975. B/116/3/3, Part 3: The Design of Steel Bridges, British Standards Institution, Document 75/11722, July (Draft Proposal, private circulation).

BSI. 2000. *Steel, Concrete and Composite Bridges – Part 3: Code of Practice for Design of Steel Bridges*, BS 5400-3:2000, British Standards Institution.

Canadian Metalworking. 2014. Clamp Down – Welding to Avoid Distortion, https://www.canadianmetalworking.com/article/welding/clamp-down-welding-to-avoid-distortion (As of January 5, 2019).

CAMM Metals. 2017. 14 Tips to Minimize Welding Distortion, http://inbound.cammmetals.com/blog/14-tips-to-minimize-welding-distortion, (As of January 5, 2019).

Coletti, D., Z. Fan, W. Gatti, J. Holt, and J. Vogel. 2005. *Practical Steel Tub Girder Design*, National Steel Bridge Alliance, Chicago, IL.

Conway, H.D. 1949. "Bending of Rectangular Plates Subjected to a Uniformly Distributed Lateral Load and to Tensile or Compressive Forces in the Plane of the Plate," *Journal of Applied Mechanics*, American Society of Mechanical Engineers, Vol. 16, No. 3.

Crandall, S.H. and W.D. Mark, W.D. 1963. *Random Vibration in Mechanical Systems*, 2nd Ed. Academic Press.

Dabrowski, R. 1968. *Curved Thin-Walled Girders, Theory and Analysis*, Cement and Concrete Association, London.

Fan, Z. and T.A. Helwig. 1999. "Behavior of Steel Box Girders with Top Flange Bracing," *Journal of Structural Engineering,* American Society of Civil Engineers, Vol. 125, No. 8, pp. 829-837.

Fan, Z. and T.A. Helwig. 2002. "Distortional Loads and Brace Forces in Steel Box Girders," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 128, No. 6, pp. 710-718.

FDOT. 2018. *Structures Design Guidelines, Structures Manual*, Volume 1, Florida Department of Transportation, Tallahassee, FL.

Feng, Z. 2005. *Processes and Mechanisms of Welding Residual Stress and Distortion*, Woodhead Publishing.

Gray, T. C. Camilleri, and N. McPherson. 2014. Control of Welding Distortion in Thin-Plate Fabrication, Design Support Exploiting Computational Simulation, Woodhead Publishing.

Grubb, M.A., D.H. Hall, J.M. Yadlosky, K.E. Wilson, and L.E. Volle. 2010. *Analysis and Design of Skewed and Curved Steel Bridges with LRFD, Reference Manual*, FHWA NHI 10-087, Federal Highway Administration, National Highway Institute, Arlington, VA.

Goldberg, J.E. and H.L. Leve. 1957. "Theory of Prismatic Folded Plate Structures," *IABSE*, 16, International Association for Bridge and Structural Engineers, Zurich, Switzerland.

Haaijer, G. 1981. "Simple Modeling Technique for Distortion Analysis of Steel Box Girders." *Proceedings of the MSC/NASTRAN Conference on Finite Element Methods and Technology*, MacNeal/Schwendler Corporation, Los Angeles, CA, March 19-20.

Heins, C.P. and D.H. Hall. 1981. *Designers Guide to Box Girder Bridges*, Bethlehem Steel Corporation, Bethlehem, PA, 32 pp.

Heins, C.P. 1978. "Box Girder Bridge Design – State of the Art," *Engineering Journal*, AISC, 4th Quarter, pp. 126-142.

Helwig, T.A., J. Yura, R. Herman, E. Williamson, and D. Li. 2007. *Design Guidelines for Steel Trapezoidal Box Girder Systems*, Report No. FHWA/TX-07/0-4307-1, Center for Transportation Research, University of Texas, Austin, TX.

ICC. 2017. 2018 International Building Code, International Code Council, Country Club Hills, IL.

KOBELCO. 2019. The ABC's of Arc Welding, http://www.kobelco-welding.jp/education-center/abc/ABC_2006-03.html, (As of January 5, 2019).

Korol, R.M. and E.G.Thimmhardy. 1984. "Field Investigation of Out-of-plane Deviations for Steel Box Girder Bridges," *Canadian Journal of Civil Engineering*, Vol. 11, pp. 377-386

Lalanne, C. 2014. *Fatigue Damage, Mechanical Vibration and Shock Analysis*, Volume 4, Mechanical Engineering and Solid Mechanics Series, Wiley.

Lincoln Electric. 2019. Weld Distortion, file:///C:/Users/dw128/Downloads/Weld%20Distortion.html (As of January 5, 2019).

Lokhande, A. and D.W. White. 2018. *Improved Characterization of the Flexural and Axial Compressive Resistance of Welded Steel Box-Section Members*, Research Report, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA.

Mandal, N. 2004. *Welding and Distortion Control*, Alpha Science Intl. Ltd.

Mandal, N. 2009. *Welding Techniques, Distortion Control and Line Heating*, Narosa Book Distributors Pvt Ltd.

Mattock, A.H. 1967. "Commentary on Criteria for Design of Steel-Concrete Composite Box Girder Highway Bridges," (unpublished).

Metal Construction Association. 2013. *Oil Canning in Metal Roof and Wall Systems*, Version 1, Chicago, IL.

Michaleris, P. 2011. Minimization of Welding Distortion and Buckling, Modeling and Implementation, 1st Ed., Woodhead Publishing.

Murray, T.M., D.E. Allen, E.E. Ungar, and D.B. Davis. 2016. *Vibrations of Steel-Framed Structural Systems due to Human Activity*, Steel Design Guide 11, American Institute of Steel Construction, Chicago, IL.

Radaj, D. 2012. *Heat Effects of Welding, Temperature Field Residual Stress Distortion*, Springer-Verlag.

Roberts, T.M. and A.W. Davies. 2002. "Fatigue Induced by Plate Breathing," *Journal of Constructional Steel Research*, Vol. 58, pp. 1405-1508.

Scholler, S. and T. Ringer. 2018a. NSRP Standardized Welding Curriculum and Testing for Shipyards, Ingalls Shipbuilding, National Shipbuilding Research Program.

Scholler, S. and T. Ringer. 2018b. *Welding Distortion Workbook*, National Shipbuilding Research Program.

Sen, R. and S. Stroh. 2010. *Design and Evaluation of Steel Bridges with Double Composite Action.* Contract BD544, RPWO #18, Univ. of South Florida.

Simulia. 2019. FE-SAFE, Durability Analysis Software for Finite Element Models, https://www.3ds.com/products-services/simulia/products/fe-safe/latest-release/, Dessault Systemes, Simulia, (As of January 5, 2019).

Skaloud, M. and M. Zörnerovà. 2005. "The Fatigue Behaviour of the Breathing Webs of Steel Bridge Girders," *Journal of Civil Engineering and Management*, Vol. XI, No. 4, pp. 323-336.

Texas Steel Quality Council. 2015. *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, Texas Department of Transportation.

Thimmhardy, E.G. and R.M. Korol. 1987a. "Geometric Imperfections and Tolerances for Steel Box Girder Bridges," *Canadian Journal of Civil Engineering*, Vol. 15, pp. 437-442.

Thimmhardy, E.G. and R.M. Korol. 1987b. "An experimental investigation of the effects of imperfections on the strength of steel box girders," *Canadian Journal of Civil Engineering*, Vol. 15, pp. 443-449.

Timoshenko, S. and J.M. Gere. 1961. *Theory of Elastic Stability*. McGraw-Hill, New York, NY.

Timoshenko, S. and S. Woinowsky-Krieger. 1959. *Theory of Plates and Shells*, McGraw-Hill, New York, NY.

TWI. 2019. Distortion Control – Prevention by Fabrication Techniques, https://www.twi-global.com/technical-knowledge/job-knowledge/distortion-control-prevention-by-fabrication-techniques-036 (As of January 5, 2019)

Welding Answers. 2019. 7 Ways to Control Distortion in Welding, http://weldinganswers.com/7-ways-to-control-distortion-in-welding/ (As of January 5, 2019).

White, D.W., A. Lokhande, A. Ream, C. King, and M. Grubb. 2019. *Proposed LRFD Specifications for Noncomposite Steel Box-Section Members*, IDIQ Task Order 5011, Final Report submitted to the FHWA Office of Bridges and Structures, Washington, DC, May.

Wirsching, P.H., T.L. Paez, and K. Ortiz. 1995. *Random Vibrations, Theory and Practice*, Dover.

Wolchuk, R. and R.M. Mayrbaurl. 1980. *Proposed Design Specifications for Steel Box Girder Bridges*, Final Report, FHWA-TS-80-205, Washington, DC.

Wright, R. N., and S.R. Abdel-Samad. 1968. "BEF Analogy for Analysis of Box Girders," *Journal of the Structural Division*, American Society of Civil Engineers, New York, NY, Vol. 94, No. ST7, pp. 1719-739.

Yen, B.T., T. Huang, D. Wang and C.K. Chuang. 1986. *Composite Compression Bottom Flange for Steel Box Girders,"* Final Report to Federal Highway Administration, Fritz Laboratory, Lehigh Univ., Bethlehem, PA.

Yoo, C.H., J. Kang, J. and K. Kim. 2015. "Stresses due to Distortion on Horizontally Curved Tub Girders," *Engineering Structures*, Vol. 87, pp. 70-85.

Young, W.C. and R.G. Budynas. 2002. *Roark's Formulas for Stress and Strain*, 7th Ed., McGraw-Hill, New York, NY.

Zhang, Y. 2007. *Strength-Based Plate Tolerances for Steel Bridge Girders,* Doctoral Dissertation, Department of Civil, Architectural and Environmental Engineering, University of Houston, Houston, TX.

Ziemian, R.D. (ed.) 2010. *Guide to Stability Design Criteria for Metal Structures*, 6th Ed., Wiley, Hoboken, NJ.

Appendix A – List of Individuals Surveyed

The following individuals were contacted by Professor White with letters of introduction and inquiry, or responded as AASHTO/NSBA Task Group Members. The responses received are greatly appreciated.

AASHTO T-14, Structural Steel Design

1. Tom Macioce (T-14 Chair), Pennsylvania DOT
2. Richard Marchione (T-14 Vice Chair), New York State DOT
3. Elmer Marx, Alaska DOT
4. Konjit Eskender, District of Columbia DOT
5. Brain Kozy, FHWA
6. Justin Ocel, FHWA (also member of AASHTO T-17)
7. Sam Fallaha, Florida DOT
8. Matt Farrar, Idaho DOT
9. Carl Puzey, Illinois DOT
10. John Jones, Kansas DOT
11. Brian Hanks, North Carolina DOT
12. John Hastings, NSBA, formerly with the Tennessee DOT
13. Jamie Farris, Texas DOT (also a member of AASHTO T-17)
14. Michael Menghini, Wyoming DOT

AASHTO T-4 Construction

15. Carmen Swanwick, Utah DOT (T-4 Chair)
16. Ahmad Abu-Hawash, Iowa DOT (T-4 Vice Chair)

AASHTO T-17 Metals Fabrication

17. Alexander Bardow, Massachusetts DOT (T-17 Chair)
18. Justin Walker, Mississippi DOT (T-17 Vice Chair)

Other Bridge Owners and State DOT Experts

19. Scott Arnold, Florida DOT
20. Prince Baah, Indiana DOT
21. Scot Becker, Wisconsin DOT

22. Timothy Carre, Pennsylvania DOT
23. Hannah Cheng, New Jersey DOT (also a member of AASHTO T-17)
24. Lian Duan, CalTrans
25. Tom Fan, TxDOT
26. Ben Goldsberry, Florida DOT
27. Scott Lagace, NYSDOT
28. Phillipe Thomas, MARTA
29. Andy Zickler, Virginia DOT

Fabricators

30. Ronnie Medlock, High Steel
31. Dennis Noernberg, W&W | AFCO Steel
32. Heather Gilmer, HRV Conformance Verification Associates, Inc., also NSBA Fabrication Committee Chair)
33. Gary Wisch, Delong Steel
34. Eric Levesque, Canam Steel – Bridges

Erectors

35. Bob Cisneros, High Steel
36. Douglas Haven, Tampa Steel Erecting

Inspection

37. Chaz Kussoy, CalTrans
38. Yongquian Lin, TxDOT

Design Experts

39. Allan Berry, RS&H (Chair of NSBA TG 12, Design for Constructability)
40. Brandon Chavel, HDR (Chair of NSBA TG11, Design)
41. Domenic Coletti, HDR
42. Ron Crockett, Consultant, formerly with American Bridge
43. Dennis Golabek, WSP, formerly with FDOT
44. David Goodyear, T.Y. Lin International
45. Chris Hendy, Atkins (Chair of SCI Steel Bridge Group, Member of subcommittee ECCS/TC8)
46. John Holt, Modjeski and Masters, formerly with TxDOT
47. Richard Hornby, Arup

48. Finn Hubbard, Fickett Structural Solutions
49. Amir Jamshidi, Supreme Group
50. Tom Murphy, Modjeski and Masters
51. Tony Ream, HDR
52. Hormoz Seradj, Consultant, formerly with the Oregon DOT
53. Tony Shkurti, HNTB
54. Jason Stith, Michael Baker (Chair of NSBA TG 10, Erection)
55. John Vogel, Sam Houston Tollway, Ship Channel Bridge, formerly with TxDOT
56. Chris Walker, COWI
57. Dayi Wang, FHWA
58. Ed Wasserman, Modjeski and Masters (Former AASHTO T-14 Chair)

Academic and Professional Experts

59. Karl Frank, Consultant, NSBA (formerly with Hirschfeld Steel, and with the University of Texas, Austin)
60. Chris Garrell, NSBA/AISC
61. Ulrike Kuhlmann, University of Stuttgart (Chair of Eurocode Committee for Steel Structures, Chair of the subcommittee of ECCS/TC8, Plated Structures)
62. Joshua Steelman, University of Nebraska

Appendix B – Sample Letter of Inquiry

NCHRP Project 20-07/Task 415
Proposed AASHTO Guidelines for Bottom Flange Limits of Steel Box Girders

Professor Donald W. White
School of Civil and Environmental Engineering
Georgia Institute of Technology
Atlanta, GA 30332-0355
dwhite@ce.gatech.edu
678-895-5451

October 8-10, 2018

Dear NSBA Collaboration Group Member:

We would like to request your input toward an important NCHRP Project 20-07 investigation. The project is NCHRP 20-07/415. Its goal is to provide guidance regarding bottom flange thickness and/or *b*/*t* limits in steel bridge box girders. The impetus for this project has been concerns raised regarding bottom flange small thickness and large *b*/*t* values in some box-girder cases, typically in locations where the bottom flange is never subjected to any calculated flexural compression, and the potential implications of these small thickness and large *b*/*t* values.

Attached is a two-page summary of the background, objective, research approach and areas of inquiry and investigation for this project. We would like to call your attention in particular to the list of 10 specific areas of investigation/query within this summary.

Succinctly put, could you help us with the following questions:

1) Do you have specific or general guidelines employed in your practice pertaining to any of the 10 areas of investigation/query within the summary?

For any of these limits or guidelines, could you please indicate the background to or basis for the limit or guidance, wherever that information may exist, or identify when particular limits are historical, based on judgment, etc. *We are required to tabulate any thickness or slenderness limits including this information*.

2) Have you encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

If so, could you provide any information to us on these cases?

3) Could you please rank the 10 items of investigation/query in terms of their importance from your perspective, using the attached *ranking sheet*?

Comments regarding any particular items are welcome.

4) Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

If you would like to request anonymity regarding any information provided, we will honor your request.

Again, thank you for time in collecting any information you may be able to provide, responding to the above questions, and assisting us as you see fit. If possible, could you please respond within the next week, by Monday, October 15? We will compile the results from our queries, and will be happy to provide you with these results.

Best regards.

Sincerely,

Donald W. White
Professor, Project PI

Charles King
Ph.D., C.Eng., F.I.Struct.E
Senior Steel Specialist
COWI Bridge

Michael A. Grubb, P.E.
M.A. Grubb & Associates, LLC

NCHRP 20-07/Task 415 Project
Proposed AASHTO Guidelines for Bottom Flange Limits of Steel Box Girders

Background

Steel box girders are highly efficient in their structural function, particularly in applications that benefit from their large torsional stiffness. They have an aesthetically pleasing uncluttered appearance that reduces the exposed surface needing maintenance. Due to their substantial structural efficiency, designers can be tempted to specify very thin and slender bottom flanges, particularly in positive moment regions where the bottom flange is in tension. However, specifying too thin or too slender of a bottom flange can result in potential problems during fabrication, transportation, erection, or future inspection and maintenance. In addition, excessively thin flanges can lead to concerns regarding strength and fatigue, especially in regions of stress reversal.

Various references pertaining to box girder design in the US and internationally provide recommended minimum thickness and maximum slenderness limits for steel box girder bottom flanges. Industry recommendations on minimum bottom flange thickness range from ¾ to 1 inch. Recommendations for bottom flange width-to-thickness (b/t) ratios range from 80 to 140. A prominent design example in the classical reference *Designers Guide to Box Girder Bridges* (Heins and Hall, 1981), pertaining to the estimation of distortional warping and distortional transverse plate bending stresses in box girders, has a bottom flange b/t ratio of 142 without any longitudinal stiffening. Recent flange maximum not-to-exceed limits balloted and approved by AASHTO Committee on Bridges and Structures for design of nonlongitudinally stiffened noncomposite box-section members are $b/t \leq 90$. These recommendations have been expanded to address general noncomposite box-section members with longitudinally stiffened plate elements in draft AASHTO LRFD Specification provisions under consideration by the AASHTO Technical Committee T-14 on Structural Steel Design. In these provisions, a more general not-to-exceed limit of $w/t \leq 90$ has been specified, where w is the width of the plate between the centerlines of individual longitudinal stiffeners and/or between the centerline of a longitudinal stiffener and the inside of the laterally-restrained edge of a longitudinally stiffened plate. The present AASHTO LRFD Specifications (2017) do not specify any limits on the thickness or slenderness (b/t) of composite box girder bottom flanges.

Specific documentation of quantitative and experiential backing for the above limits generally appears to be lacking.

Objective

The objective of this project is to synthesize information from all sources, and additionally to supplement this information with further limited quantitative assessments, to help color the ramifications of different

1) minimum thickness limits and
2) maximum slenderness limits

for steel box girder bottom flanges. The ultimate objective of the project is recommended AASHTO guidelines for minimum thickness and maximum slenderness limits for these structural elements. Broad criteria to be considered include fabrication, handling and erection, as well as the overall long-term performance of box girders in their final constructed configurations. Recommended guidelines/specifications for incorporation into the AASHTO LRFD Specifications and/or other pertinent industry documents will be developed as project deliverables. These recommended guidelines/specifications will be submitted to the AASHTO Committee on Bridges and Structures Technical Committee T-14 Structural Steel Design for their evaluation and potential adoption.

Research Approach

To accomplish the above objective, the NCHRP 20-07/Task 415 project is seeking detailed information on current bottom flange thickness and slenderness limits from bridge owners and stakeholders, as well as experiences pertaining to these limits. The research team is seeking both national and international information. Limited analytical studies will be conducted to provide further understanding of the behavioral ramifications of different limits. The collected information and the fundamental studies will be analyzed to provide the technical basis for the recommended specifications/guidelines to ensure adequate performance during fabrication, transportation and construction, as well as the service, fatigue and strength performance of box girders in the final constructed condition. The research will address both stiffened and non-stiffened bottom flanges in both composite and non-composite box sections. Potential issues in box girders due to the lack of requirements for minimum thickness and maximum slenderness will be identified.

Areas of Inquiry and Investigation

The NCHRP 20-07/Task 415 project is seeking information pertaining to the following broad considerations:

- Fabrication and handling (e.g., transportation)
- Erection / Construction
- In-service performance, including inspection and maintenance.

Specific areas of investigation/query pertaining to bottom flange thickness and *b*/*t* limits include:

1) Welding distortion.
2) Vibration of excessively thin plates during transportation.
3) Localized plate bending stresses induced by handling and erection operations.
4) Vibration of excessively thin plates due to in-service loadings on the completed bridge.
5) Localized plate bending stresses due to box-section distortion (i.e., distortion transverse bending stresses) as well as secondary stresses due to out-of-plane deformation of the box-section bottom flange.
6) "Bending reluctance" of excessively thin flanges, that is, the tendency for thin flanges to not bend fully along with the overall curvature of the box-section member.
7) Localized out-of-plane deflection due to a concentrated transverse load applied to the bottom flange, or to a plate panel within the bottom flange.
8) Buckling of thin bottom flange plates intended to serve predominantly in tension, due to unanticipated or accidental axial compression in the plates during handling and transportation, erection, and in the final service condition, particularly in the vicinity of inflection points in continuous-span girders or near locations where longitudinal stiffeners are terminated.

9) Perception of vertical vibrations, or general “sponginess” in bottom flanges of box girders, by bridge inspection personnel. These vibrations may be due to transient live loads on the bridge or due to the individual’s movements when walking the inside of the box girder.
10) Potential “oil canning” of thin plates in bottom flanges, i.e., snapping in or out of the plates between edge supporting elements when pushed on by a light force.

These considerations will be prioritized based on a review of the literature and on input from surveys of a range of experts in steel bridge design, fabrication, erection and maintenance.

NCHRP 20-07/Task 415
Proposed AASHTO Guidelines for Bottom Flange Limits of Steel Box Girders

Ranking of considerations pertaining to bottom flange thickness or slenderness limits

- Welding distortion.
- Vibration of excessively thin plates during transportation.
- Localized plate bending stresses induced by handling and erection operations.
- Vibration of excessively thin plates due to in-service loadings on the completed bridge.
- Localized plate bending stresses due to box-section distortion (i.e., distortion transverse bending stresses) as well as secondary stresses due to out-of-plane deformation of the box-section bottom flange.
- "Bending reluctance" of excessively thin flanges, that is, the tendency for thin flanges to not bend fully along with the overall curvature of the box-section member.
- Localized out-of-plane deflection due to a concentrated transverse load applied to the bottom flange, or to a plate panel within the bottom flange.
- Buckling of thin bottom flange plates intended to serve predominantly in tension, due to unanticipated or accidental axial compression in the plates during handling and transportation, erection, and in the final service condition, particularly in the vicinity of inflection points in continuous-span girders or near locations where longitudinal stiffeners are terminated.
- Perception of vertical vibrations, or general "sponginess" in bottom flanges of box girders, by bridge inspection personnel. These vibrations may be due to transient live loads on the bridge or due to the individual's movements when walking the inside of the box girder.
- Potential "oil canning" of thin plates in bottom flanges, i.e., snapping in or out of the plates between edge supporting elements when pushed on by a light force.

Appendix C – Written Responses and Follow-Up Discussions from Task 2 Survey

The individuals providing written responses to the letters of introduction and inquiry are listed as Respondents 1 through 36 in the headings of the sections below, followed by pertinent excerpts of their written responses. The various responses to the survey are greatly appreciated. Greetings and other written responses not pertinent to the survey results are not included; however all other written responses are included. Specific bridge names and references to individuals, organizations and states associated with a particular bridge are redacted. References to organizations and individuals where published reference information exists are shown. Where needed to provide context, the specific survey question being answered is marked with the sub-headings ***Survey Question #*** in bold italics. Summaries of data provided with the written responses are marked with the sub-headings ***Summary of Provided Data*** in bold italics. Follow-ups to the written responses from Professor White are marked by the sub-headings ***Follow-up***. Written comments on the specific list of considerations provided with the surveys are marked by the sub-headings: ***Regarding Item # of the considerations listed in the survey, …*** with a short statement of the consideration in the sub-heading. In a few cases, the respondents commented on particular statements within the project summary provided with the survey. These comments are denoted by the sub-headings: ***Statement within the project summary transmitted with the survey***, followed by the statement.

Respondent 1

I don't have any experience with the in-service or transportation concerns listed but have seen issues with welding distortion, particularly at end bearing diaphragm assemblies (the ones in the middle for continuous spans tend to be at thicker flanges but the ends can get very thin). It's been a long time since I've been around a Texas-style dapped-end girder but I don't remember the thin plates being harder to fit to the cut curves in the webs than the thicker plates.

Respondent 2

… during my career at [redacted], we only designed and constructed around 9 composite tub girders, all but one were curved. … Spans reached up to around 300-ft as I recall.

As I recall, the designs met the limited guidance existing in the Standard Specifications at the time, influenced by Dr. Conrad Hine's work at the University of Maryland and the consortium that developed the curved girder provisions at the time. The designs were also influenced by a US Steel publication developed by Richard Fountain.

The bridges were designed using the M/R method on a TI-59 calculator!

I can arrange for you to download the plans of several of these bridges if you desire. The bridges were twin tubs supporting a 42-ft roadway and had no external cross-frames, "ALA FOUNTAIN" and had limited top flange lateral bracing. While there were no known difficulties during construction, issues dealing with the quality of the partial depth precast pre-stressed bridge deck panels led us to core several decks post-construction. Cores amazingly revealed the tubs had rotated torsionally and deck thicknesses in some of the positive moment areas of the long spans exceeded 12-15 inches as I recall whereas the nominal thickness was to be 8-inches. The only problems that the bridges have experienced have been related to fatigue cracking at the termination of the cross-frame connection welds which were not connected to the top flanges in tension nor the compression flanges, an acceptable practice at the time. At the tension flanges the stiffeners were cut short and at the compression flanges the stiffeners were a tight fit. Torsion is suspected as the culprit and repairs to all connections have been retro-fitted.

Summary of Provided Data: The plans provided indicate tub girder bottom flange widths ranging from 68 to 121.5 inches, and unstiffened bottom flanges with thicknesses ranging from 5/8 to 1.25 inches and b/t values ranging from 68 to 177. Four of six bridges provided had unstiffened bottom flanges with b/t of 120 and greater. All six bridges had bottom flanges with a single longitudinal stiffener at some location along the length, with thicknesses in these regions ranging from 11/16 to 1.625 inches and w/t ranging from 10.5 to 58.9.

Respondent 3

Unfortunately I have little to offer for the scope of your studies. The issues raised and the specific study are largely outside my experience as a constructor, and I don't think I can contribute much beyond the brief statements below.

All of my experience in this area involves stiffened bottom plates encountered in medium and long span bridges that must accommodate compression for various permanent loading conditions (orthotropic box girders, composite steel tub girders, arch boxes and the like).

I cannot think of additional factors to add to your list – it seems comprehensive to me.

From my experience I regard Items 1 and 5 [items numbered in the order of the list of considerations provided in the survey], weld distortion combined with required flatness and straightness tolerances to minimize secondary bending forces, as the most significant issues to address because they can lead to fatigue issues (and buckling under transient compression loading). Flange and web flatness and straightness tolerances that are required to be maintained during fabrication are often difficult to achieve without extraordinary and expensive measures, especially as plates become thinner, remain unstiffened and are more prone to weld distortion. Tolerances must be calibrated to consider fatigue, and fabrication practices must be adjusted to meet tolerances.

Based on my experience with stiffened bottom flanges, I am unaware of particular erection issues that cannot be solved with appropriate erection analysis, erection procedures and lifting and handling devices. Nevertheless, this would be a factor if unstiffened bottom flanges must accommodate compression during erection but were not designed to accommodate such loading (see Items 3 and 8).

In my opinion achieving tolerances that will not cause excessive stress reversals due to a variety of loading conditions must always be considered, as implied among the 10 factors. The importance will vary according to design, transportation, handling and erection procedures, which are always bridge and site specific. Referring to Items 2, 6, 7 and 8, fatigue life needs to be considered when significant stress reversals and cycles can occur due to transport methods and positioning of transport supports and bracing.

Respondent 4

The [redacted] has no formal policy on minimum bottom flange thickness for trapezoidal steel box girders but a gross review of several bridges indicates that bottom flanges typically have b/t < 84 for those section in full tension (only positive moment) sections. The average b/t ~ 60. Lower values for curved, higher values for straight girders. Thinnest bottom flange is 3/4" (b/t ~ 70).

We have no records of fabrication or in-service problems with bottom flange thickness - but this does not mean that none have occurred.

Respondent 5

I have only come across this problem once and, coincidentally, we are still involved with undertaking strengthening to the bridge. The bridge is [redacted]. It is a steel-concrete composite bridge with multiple steel boxes across the width of the bridge supporting the deck slab. The bottom flange is typically 2.1 m – 2.3 m wide with cross frames at 3 m centres. The bottom flanges generally longitudinally stiffened but near midspan, for panels which remain in tension under all load combinations, all the longitudinal stiffeners are curtailed leaving a wide unstiffened panel with a b/t ratio of between 260 and 280. The bottom flanges were observed to vibrate in and out of plane when viewed from below the bridge and this led to structural assessment and site monitoring being initiated about 5 years ago. The bridge has been in service since the late 1980's.

No specific guidelines exist in Eurocodes for shape limits of internal tension flanges. I raised this issue a few years back at one of our working group meetings of the committee that maintains EN1993-1-5 (the Eurocode dealing with stiffened plates) and none of the other European delegates had encountered the issue or had any national rules covering the subject or any national guidance. There was no appetite from anyone to investigate the problem further and to try to develop any rules for EN 1993-1-5 because of the lack of existing examples, the feeling that such a design was bad practice and because the Eurocodes were considered to provide sufficient high level principles (in theory) for such flanges to be designed if someone wanted to (e.g. through dynamic analysis and non-linear material and geometrical analysis).

The vibration on [redacted] prompted concerns about fatigue at the welded connections with the panel boundaries and particularly at the connection between the unstiffened panel and the transverse stiffeners where the longitudinal stiffeners started, which provides a stiff rotational restraint to the panel boundary. Strain gauges were used to measure insitu stress ranges (and digital image correlation was also used as verification) – these confirmed that the peak bending stresses occurred at the panel boundary with the transverse stiffener. Non-linear geometric modelling was also undertaken. The two approaches gave very similar stress ranges under typical traffic loading and were sufficiently high to confirm concerns over fatigue – indeed we have found a few fatigue cracks but nothing like the number predicted from assessment calculations. But the high stress range did not arise from vibration but from breathing of the flange panel under cyclic tension – the out of plane bow in the flanges from initial fabrication imperfections and from self-weight deflection were being cyclically straightened out and released, causing bending moments to be developed in the plate. So this breathing was the critical issue, although aspects of other items in your list were present.

Follow-up: …Indeed, this case does seem to have characteristics of the type that were the impetus for the NCHRP 20-07/415 project. You point out an additional response not included specifically in our list, i.e., the breathing of the plate associated with cyclic straightening out and release of the initial fabrication imperfections and self-weight deflection. I suppose our category "oil canning" is the closest, but clearly the breathing of the plate under cyclic tension is different than oil canning. We would be very interested in the assessment reports on this bridge, if they are available without too much trouble. We would be pleased to cite this case as an example while removing all specific information identifying the bridge.

I will ask and come back to you.

Follow-up: …We will need to perform our synthesis of the inputs received on this topic at the end of October. We would be most appreciative of any additional details you can provide regarding the [redacted].

I am chasing up. The problem is that there are three stakeholder organizations, all of whom have to give permission for release of the two reports I had in mind.

Respondent 6

We do not have any specific limits on flange thickness (particularly in tension zones). Offhand I do not remember ever using a bottom tension zone plate below 1" thick.

We have not had any issues with our steel box girders to date. We have not used bottom flange thickness below ¾ inches either so this may be part of why we have not had any issues. I heard that a major steel box girder in [redacted] has experienced all kinds of fatigue issue over the years. Seems it was built 20 to 25 years ago with very thin plates as I recall.

We have had a few maintenance issues with water (salt water) getting into our boxes and causing paint issue. Drainage and drainage hole size are the issues as the holes can clog.

Respondent 7

Follow-up: Identification and discussion of major steel box girder bridge cited in the above response

There are very few box girder bridges in [redacted] and only one in [redacted]. The one in [redacted] is the [redacted] and there haven't been any significant issues with the box portion of that bridge. There are three box girder bridges in [redacted], and again no issues there.

There are two box girder bridges in [redacted], [redacted] left and right in [redacted] that has had fatigue issues. There are a few others that have had issues to a smaller extent but none in [redacted] that I know of.

I finally have a chance to look more carefully into the box girder issues and still haven't come up with any steel box girder bridges in [redacted]. We have had fatigue cracking issues with some steel box girder bridges but these are all located in [redacted]. The bridges in our inventory are on State routes and local roads and omit any railroad owned bridges.

We have the most issues with [redacted] (left and right) and it's possible that this is the pair of bridges that you are inquiring about. This bridge was built in the 1970s with 3/8" bottom flange and fatigue crack issues dating back to construction.

I've attached the 2015 peer review which gives some history and details of on-going fatigue cracking issues with these bridges.

Summary of Data Provided: The provided information shows issues that point clearly to web gap fatigue problems in the box girder webs of this bridge. There does not appear to be any clear correlation between the fatigue cracking and the slenderness of the bottom flanges. The bottom flanges in the box girders of this bridge are 3/8 inch thick in a number of segments along the bridge length. The box girders have three webs, one at the mid-width of the box, and have three longitudinal stiffeners between each of the webs. The w/t of the flange plates ranges from 51 to 177, with the w/t in 18 of the segments being larger than 120.

My expertise is on the inspection end of things, not design so I will need to leave the design parts of the questions to others who are better qualified to answer. Also, when it comes to steel box girder bridges, we just don't have all that many in our state, roughly 15 or so of this type.

Regarding question 1, I will leave that to more qualified design engineers to answer.

#2. Yes. We have two bridges, [redacted] (left and right), that has a history of problems dating back to construction that has a thin bottom flange. Details of the problems associated with this bridge were sent in the previous separate email.

#3. I've attached the sheet separately. I have not encountered all of the issues mentioned and only attached a number to the items the inspection team has encountered.

#4. Yes, corrosion from the inside as well as the outside. Boxes are often not properly sealed for water intrusion, bird and human populations. The result is ponding water that doesn't drain as well as collections of bird droppings that sit for years without being cleaned off has a tendency to create section loss. When the corrosion process is working from the interior and exterior of a thin bottom flange the resulting section loss can become accelerated.

A related issue is homeless people taking up residence inside or under the steel boxes. We have cases of homeless individuals storing flammable liquids along with flammable materials. There have been cases of small fires causing significant bridge damage.

The interiors of steel boxes are out of sight and don't always get the same maintenance. It becomes complicated to move out people and possessions to perform the needed maintenance.

Respondent 8

[redacted] does not have any specific requirements beyond AASHTO to steel box girders.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

I'm not aware of any.

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

Ability to make heat straightening or other repairs in the event of collision damage. We do a fair amount of heat straightening of I-shaped steel girder and the ability to repair these is a concern.

Respondent 9

Our top ranking items are distortion issues related to welding/construction /localized deflection/buckling.

One issue that is not listed is the aesthetics and the visibility of the distortion /deflection.

One question: Has vibration due thin bottom flange been an issue?

Follow-up: …Regarding the vibration of a thin bottom flange, the following excerpt is from our literature review:

"Wolchuk and Mayrbaurl (1980) specifically state that "slender and flexible tension flanges which may be subject to dynamic excitation shall possess sufficient rigidity, or be suitably damped, to withstand excitation." They also state that "Tension flanges of multi-box composite girders designed under the provisions of Art. 1.7.203 [i.e., their recommended provisions for bridges of moderate length supported by two or more single-cell composite box girders] shall be deemed to satisfy the dynamic stability requirements." However, there do not appear to be any restrictions on tension flange thickness or b/t values in this article. Elsewhere, Wolchuk and Mayrbaurl (1980) do specify a maximum value of 120 for the b/t of longitudinally unstiffened and the w/t of longitudinally stiffened tension flanges, as well as a slenderness ratio L/r limit of longitudinal stiffeners on tension flanges. They indicate that these limits are arbitrary, and that the value of 120 was a proposed limit in a 1975 draft British standard (BSI 1975). Furthermore, they state that this maximum b/t is intended to limit the dynamic excitability of the flange."

From responses thus far, we have a few mentions of fatigue issues in a number of boxes with excessively thin flanges (don't have specifics on the actual b/t's with this yet). We have one reference to a steel-concrete composite bridge outside of the US with multiple steel boxes across the width of the bridge supporting the deck slab. The bottom flange is typically 2.1 m – 2.3 m wide with cross frames at 3 m centres. The bottom flanges generally longitudinally stiffened but near mid-span, for panels which remain in tension under all load combinations, all the longitudinal stiffeners are curtailed leaving a wide unstiffened panel with a b/t ratio of between 260 and 280. The bottom flanges were observed to vibrate in and out of plane when viewed from below the bridge and this led to structural assessment and site monitoring being initiated about 5 years ago. This bridge has been in service since the 1980's. Some fatigue cracking has been observed in these flanges, "but nothing like the number predicted from assessment calculations."

Clearly b/t = 260 and 280 can be considered as an outlier.

Respondent 10

I have provided some questions/comments with some of the items. Not knowing more background, this seems like a rather ambitious scope.

Regarding Item 4 of the considerations listed in the survey, vibration of excessively thin plates due to in-service loadings on the completed bridge:

Is this a common occurrence? And what is resulting damage?

Follow-up: [The above reply was also transmitted to this responder.]

Regarding Item 1 of the considerations listed in the survey, welding distortion:

I'm surprised that plate thickness limitations based on weld distortion are not already well established. Weld distortion would have to meet AWS tolerances. So is this considering potential implications of distortions that are within the AWS tolerances?

Follow-up: Unfortunately, AWS D1.5 does not provide any guidance regarding thickness or b/t limits that may be tied to welding distortion.

Regarding Items 10 and 6 of the considerations listed in the survey, potential "oil canning" and "bending reluctance" of excessively thin flanges:

... sound very similar

Regarding Items 10 of the considerations listed in the survey, "bending reluctance"

Fabrication issue? Curved flange plates are cut to shape, so I don't understand the concern or problem.

I now understand "bending reluctance" is a fabrication issue and not a constructability or performance issue

Follow-up: The project team did not intend to define "bending reluctance" as a fabrication issue; however, it is clear that this term is somewhat ambiguous and confusing. Bending reluctance is possibly one of the more obscure aspects of the behavior of box-section members. When a box section is subjected to bending, a certain amount of curvature is induced at the webs of the section, and at the edges of the flanges. Based on beam theory, it can be said that the shear flow in the webs and in the flanges induces the flexural compression and tension stresses within the cross-section. The flexural compression stresses induce shortening on one side of the neutral axis and elongation on the other side. This in turn causes the overall curvature of the box-section member. If we focus upon the compression flange, the shortening of this element due to compressive stress-strain does not directly induce curvature. The bending curvature in this flange is induced by its attachment to the webs, and by the curvature in the webs associated with the differential in the normal strains through the web depth. If one takes a plate and induces a curvature by deforming only its edges (via the curvature of the webs in the box-section member), the interior portions of the plate do not generally follow the same curvature as the plate edges. Twisting moments, M_{xy}, and transverse bending moments, M_{yy}, are also induced within the plate, in addition to the plate bending moments M_{xx} and the membrane forces P_x (with the x direction being the longitudinal direction of the member).

Your interpretation of "bending reluctance" is certainly a logical one. Our thinking is that this interpretation would suggest that thicker plates are more difficult to bend to say fit the cut vertical curve of the webs. Thinner bottom flanges would be easier to bend, but also potentially more difficult to handle in terms of maintaining a required "flatness" or "uniform curvature" in fitting to the webs.

Respondent 11

FDOT Guidelines:

SDG 5.5 MINIMUM STEEL DIMENSIONS [6.7.3]

A. The following minimum dimensions have been selected to reduce distortion caused by welding and to improve girder stiffness for shipping and handling.

…

3. The minimum box girder bottom flange thickness is 1/2-inch.

4. The minimum stiffener thickness is 1/2-inch

"Engineering Practice": Deflection of box flange due to self-weight and a 500 lb. concentrated live load shall not exceed L/360.

I have heard of a couple of cases where the box flange was so flexible during fabrication that the fabricator had to add transverse stiffeners to the box flange to stiffen it. No details though.

Comments on Specific Areas of investigation /query:

1. Add area regarding to allowable out-of-flatness fabrication tolerances with respect to ASTM A6 Table 14, and associated Tables 14, 1 & 2. Also consider lack of criteria in D1.5 for a fabricated welded tub girder.

2. Add area regarding buckling of box flange in compression w.r.t acceptable out-of-flatness due to fabrication tolerance and loads applied directly to box flange (e.g. self-weight and 500Ib concentrated live load). A related reference is Asadnia, M, "Out-of-Flatness Plate Tolerance for Steel I-shaped and Tub Highway Bridge Plate Girders," Ph.D. Dissertation, Department of Civil and Environmental Engineering, the George Washington University, Washington D.C.

Follow-up: …I have the Asadnia dissertation, as well as the earlier dissertation by Zhang at U. Houston and other collections regarding imperfection tolerances. I will give the connection between out-of-flatness tolerances and plate slenderness some further thought.

Respondent 12

The fabricators involved in the Texas Steel Quality Council at the time the Council discussed this problem are either out of the bridge fabrication business ([redacted]) or have been bought-out and shuttered ([redacted]). This all occurred over two decades ago.

Texas Steel Quality Council (TSQC) recommendations came from discussions with bridge designers and fabricators after TxDOT had built several steel tub girders with thin flanges (3/4" and less); plate distortion/oil canning was observed and workers walking inside the girder caused local deflection. These flanges met the FHWA upper b/t limit of 140 but their performance was unsatisfactory to TxDOT engineers. Conversations among TxDOT engineers led to the recommendations of 1" as a preferred minimum thickness for bottom flanges, with designers asked to verify if practical stiffness needs would be met with thinner flanges. TSQC makes provision for an absolute minimum of ¾" and further stipulates an upper b/t of 80, 2/3 of the value in the FHWA document. I have not heard of a problem with local deflection since. There was not a strong consensus on this issue, hence the recommended limit (1") and absolute limit (3/4"). TSCQ Preferred Practices are not TxDOT design policy; they are an aid to steer designers toward fabricator- and erector-friendly bridges. It is good this issue is being investigated in-depth.

Inspection access holes are in bottom tension flanges and they take out a large portion of the width; additional flange thickness would mitigate the section loss and stress ranges.

Todd Helwig is researching the viability of flatter web slopes, which would decrease bottom flange width for a given vertical web depth. This could help limit flange material added by designers just to satisfy b/t limits.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

Yes, per the discussion noted above on observed distortion and undesirable local deflections. I do not recall the exact bridge names; this occurred over twenty years ago.

Service life issue/consideration: I did a condition assessment of the interior of a 40 yr-old non-composite box girder; rain water was getting inside it, but couldn't drain out and was ponding inside the girder to a depth of a few inches. The girder is fabricated from weathering steel and the interior is uncoated. The bottom flange and bottom portion of webs had lost section from corrosion. The thinnest bottom flange was 1" thick. If they were thinner, the section loss would have been a bigger concern.

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

Resilience is an area of concern. Collision with over-height trucks has damaged flanges and webs. Fires damage bridges, too. AASHTO does not require design for either of these extreme events, but every state DOT has had to deal with over-height collisions on their bridges and I suspect most, if not all, have had to address fire-damaged bridges. Having a thicker flange beyond meeting minimum requirements can offer owners low-cost resilience.

I'm aware of [redacted] having to repair a tub girder that had been hit by an over-height truck. It would be worthwhile to know the specifics of the girder design for that bridge.

Several tub girder bridges had been built in [redacted] before we generated the Preferred Practices limit and they had observed what they thought was undesirable behavior in thin bottom flanges. We also had design policy from Colorado DOT that limited b/t for tub girder tension flanges to no greater than 120 (and this was copied from an FHWA Report dated 1980).

The [redacted] experience was that b/t of 120 was too thin. This was when [redacted] and [redacted] were fabricating most [redacted] tubs. Empirical evidence was a bottom flange would deflect with a worker or inspector inside. We also had concerns of bottom flange oil-canning. Based on this experience, the Preferred Practices pointed designers to 1" min flanges, but allowed ¾" as an absolute minimum and the flange slenderness b/t should be limited to 80 or less. There was even some hesitance to allow plates thinner than 1", but we did.

[redacted], former [redacted] engineer, was instrumental in the Texas Steel Quality Council limits on b/t ratios for bottom flanges in tub girders. I talked with him today and he's open to you reaching out to him for his thoughts. He recalled specific cases of "too thin" bottom flanges. [redacted] was one of the co-authors of Practical Steel Tub Girder Design.

Respondent 13

.I served as a bridge design team leader from 1991 through 2011. During and after that time the [redacted] Bridge Design Section designed dozens of steel trapezoidal bridges, mainly for direct connectors but also for long span overpasses. My team did all the analysis and code checking. I was only the manager. Steel Trapezoidal Bridges are no longer in favor here due to the bridge inspection burden. My memory of bottom flange stability is more than 20 years old. At that time we were building several Single Tub HOV direct connectors designed by a consultant for [redacted]. I believe the bottom flanges were ½ inch thick. The fabricator, [redacted] complained about oil canning and buckling of the bottom flange during handling and storage in the shop. The issue was discussed very early after the creation of the Texas Steel Quality Council, which I was a member of. Obviously we needed a rule related to flange width. My memory is that [redacted] who were members, came up with the w/t criteria. I know of no issues with two girder bridges with narrower bottom flanges. One of my primary duties was to resolve fabrication and construction issues for [redacted]. Currently I am serving on the construction side of the profession. I would rank stiffness/buckling as the most important issue. Serviceability still often controls in the load and resistance factor design world. Nobody likes wobbly girders even if it is a temporary condition. I have never heard of a complaint about vibration or excessive deflection in a completed bridge. I have not heard of issues with welding distortion for the plate thicknesses used in my time because we tend to thicken plates rather than stiffen them. Localized stresses have not been an issue; however, 3D FEA is not used in Tub Girder design so thank God for ductility. I have no knowledge of "bending reluctance." I know if no issues related to localized out-of-plane deflection due to a concentrated transverse load.

Follow-up: …Would it be possible to identify and obtain plans from [redacted] for a number of the bridges that exhibited oil canning and buckling of their bottom flange during handling and storage in the shop?

So the structures I recall are the [redacted]. [redacted] is the District Bridge Engineer. [redacted] He should be able to help out on this. … I believe these bridges had ½ inch bottom flanges.

Follow-up:

Dear [redacted]

I am the PI on an NCHRP 20-07 study targeted understanding and recommending potential not-to-exceed slenderness limits and/or minimum thickness limits for bottom flanges in steel box girders. I'm attaching information on this project distributed to a number of NSBA Steel Bridge Collaboration Group members at their meeting last week in Austin.

[redacted] ***suggested that you may be able to assist us with access to plans for a group of bridges in which the tub girders exhibited oil canning and buckling of their bottom flange during handling and storage in the shop. John indicates that the bridges*** [redacted]***. My understanding is that the fabricator was*** [redacted]***. Also, I understand that these bridges were one consideration in the setting of recommendations by the Texas Steel Quality Council (2015) Preferred Practices for Steel Bridge Design, Fabrication and Erection of "Bottom tension flanges should never be less than ¾ inch thick" and "bottom tension flanges should have a w/t ratio of 80 of less."***

Would you be able to assist us with access to these plans, such that we can ascertain the bottom flange slenderness and thickness values in these bridges? We are not seeking to identify any specific bridges, but we are hoping to gain specific knowledge of thickness and b/t values and the corresponding behavior.

Respondent 14

Please see the attached comments from [redacted], and the Tub Girder details for the [redacted].

Summary of Data Provided: The provided bridge plans show bottom flange thicknesses ranging from ½ inch to 1.75 inches in the dead-load positive moment regions. Maximum b/t values of 222, corresponding to a thickness of ½ inches, were employed near the abutments and dead load inflection point locations, ranging down to 63 in the maximum positive moment locations, corresponding to a thickness of 1.75 inches.

Respondent 15

The [redacted] has largely followed the [redacted] Steel Quality Council's Preferred Practice and limited the plate thickness (t) to be min 3/4" (mostly 1") and the width/thickness (b/t) ratio to be max 80 for tub girder bottom flanges.

Some of the 10 listed problems were experienced in the early projects in [redacted] in the 1990's and early 2000's, when thinner and more slender plates were used in tub girder bottom flanges. The lessons learned in those projects are part of the bases for the current design recommendations in the Preferred Practice. With the adoption of these policies, there have been few reported problems for bottom flanges during erection and construction in recent years.

This doesn't mean that there is no problem during fabrication and shipping. However, those processes are usually monitored by Construction Division, which is a good source of information for these problems.

Comments on item 5) or e) about Distortion: By using the currently recommended spacing of internal cross-frames as well as the limits on min t and max b/t, distortion has not been a problem. There has been no serious distortional related out-of-plane bending observed.

Comments on item 8) or h) about Stress Reversal: Even if some of the bottom flanges experience the stress reversal, these temporary stresses of opposite direction are usually small in magnitude for typical span layouts and construction methods. The limits on min t and max b/t seem to be able to provide enough capacity to handle those temporary reversed stresses.

A note on Longitudinal Stiffeners: Longitudinal stiffeners were used in earlier steel tub constructions in [redacted], but were rarely, if any, used in the last 20 years of tub girder bridges. It was found that increasing the thickness of the bottom flange when needed in lieu of using longitudinal stiffeners usually results in simpler details and more economic design. This practice will most likely continue for the typical direct connectors in light of the current trends of using more and narrower tubs to evade the fracture critical status.

The above comments are made on the typical [redacted] highway bridges using steel tub girders. They are not applicable to special bridges with very wide tubs or boxes.

Respondent 16

Follow-up: I was looking back at "Practical Steel Tub Girder Design," and I noticed on page 16 that you had quoted from the Texas Steel Quality Council (2000), "In no case should bottom tension flanges be less than ½ inch thick." However, in the current 2015 book the statement is "… ¾ inch thick." Would it be quick for you to check whether the 2000 reference stated "1/2"? This would indicate a change to"3/4 inch" in subsequent versions of the document?

I'll be darned if I can find a copy of the 2000 version of the TSQC Preferred Practices guide. I have 2005 and 2009 and 2015. All three say 3/4".

There must have been a 2000 version… the Practical Steel Tub Girder Design guide was published in 2004, written before then, so we were clearly referencing a TSQC Preferred Practices earlier than 2005.

I've CC'ed [redacted] (one of the coauthors of the Practical Steel Tub Girder Design guide, and a key player in the TSQC going way back).

[redacted] can you provide earlier copies of the TSQC Preferred Practices guides? I only have 2005, 2009, and 2015… I must have lost my earlier copies.

Respondent 17

I cannot find a copy of the original Preferences (the word "original" demands some elasticity in how it applies to the preferences). But I did find some proposed modifications (dated 11/2000) to the original Preferences with the ½" min bottom flange thickness recommendation unchanged—see first attachment, Section 9.1 The second attachment is a ballot version (dated 8/2001), with this recommendation changed to ¾". The push for a thicker plate recommendation would have come from the [redacted] office.

Respondent 18

We did a query and we only have 28 steel box girder bridges in our state bridge inventory (if all are coded properly) and the last one that was constructed was in 1991. Most of them were constructed in the 1970's. So from a design/fabrication perspective, I don't think we can consider ourselves as experts. I am not aware of any performance issues with these bridges, with the exception of one with fatigue cracking issues at the cross frame connection plate to web welds.

Follow-up: …Would you be able to glean out the bottom flange b/t values for the bridges in your inventory, and the w/t values (w = stiffener spacing) if the flanges have longitudinal stiffening? That would be helpful in quantifying the aspect that [redacted] hasn't encountered any problems with your bridges.

Don, please see the attached file. I hope this is the information you are looking for.

Summary of Data Provided: The provided compilation indicates tub girder bottom flange widths ranging from 60 to 81 inches, unstiffened bottom flanges with thicknesses ranging from ½ to 2.25 inches and b/t values ranging from 27 to 162, and with 17 bridges having unstiffened bottom flanges with b/t of 120 and greater. The compilation indicates 24 bridges having bottom flanges with a single longitudinal stiffener, thicknesses ranging from ½ to 2 inches, and w/t ranging from 17.1 to 60.

Respondent 19

Web and flange panning. It is an issue with the thin plates due to shrinkage.

Respondent 20

Survey Question 1: Do you have specific or general guidelines employed in your practice, or within your organization, pertaining to any of the 10 areas of investigation/query within the summary?

A general [redacted] rule for tub girders is b/t ≤ 60 for compression and b/t ≤ 80 for tension. However, these are not hard limits. Near a flange transition where a positive moment flange is in compression at a transition, 60 will be exceeded if the compression stress is low (i.e. <10 ksi). Similarly, for tension flanges, a b/t of 83 may be OK if it means upping the flange 1/8" to meet 80. I would generally use a minimum of ¾"

During a large DB project, the contractor was questioning our limits and requested b/t < 135 for tension and as necessary for compression stresses based on input from an independent design firm (more known for concrete). They also requested a minimum plate thickness of 5/8" based on input from [redacted]. The design team pushed back on the b/t (accepted the minimum 5/8) and provided the attached references [redacted] from various sources including the AASHTO 93 curved spec, AASHTO LFD specifications and TXDOT detailing manual, which back up 60 and 80 numbers. TxDOT specifies a minimum ¾" flange. We ended up going a little larger than 60 or 80 where possible but generally, the design stresses (or distortion stresses) limited the flange thickness to our 'normal' ranges, especially without longitudinal stiffening, which we didn't feel was economical based on the box widths of the project.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

For a while we were doing independent analyses of tub girder designs for [redacted] projects. On one project the designer had an 11' wide flange at a simple support with a dapped girder. Because the end was dapped, the bottom flange was elevated and wide. Because it was an end support, they had a ½" flange detailed, which we questioned based on all the non-strength concerns detailed in your check list. We looked at point loads and lateral compression due to shear forces in the inclined webs and concluded those stresses were close to the very low compression capacities based on using Timoshenko without consideration of distortion or secondary stresses. I'm not aware of what the final resolution was.

Survey Question 3: Could you please rank the 10 items of investigation/query in terms of their importance from your perspective, using the attached ranking sheet? Comments regarding any particular items are welcome.

Importance for me is strength (including lateral compression capacity of plate), effect of long term consistent vibrations and distortion on fatigue, and finally appearance inspectors, owner and traveling public.

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

This is probably related to one of the items, but I would add the lateral compression capacity of the plate in relation to the compression loads due to the shear of inclined webs in tub girders (horizontal component applied to bottom flange).

Respondent 21

Survey Question 1: Do you have specific or general guidelines employed in your practice, or within your organization pertaining to any of the 10 areas of investigation/query within the summary?

We follow the specified codes and specs in order to get the limits and guidelines + States of the art in order to control the Welding distortion.

We do not have a record of any thickness or slenderness limits including this information.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

No record

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

The use of longitudinal and/or transverse stiffeners should be minimized. The increase of material thickness associated to the reduction of complexity in fabrication usually lead to savings for the owner. Saving on short terms and saving on long term having better fatigue performance (if applicable). Anyway, less details will also lead to less corrosion problems on long term and easiest/safest access for the inspectors.

Drainage - waves could cause in time local corrosion problems.

Regarding Item 2 of the considerations listed in the survey, vibration of excessively thin plates during transportation:

We do not have data - depending of the blocking, sometimes has to be considered for transportation

Regarding Item 3 of the considerations listed in the survey, localized plate bending stresses induced by handling and erection operations.

Erection procedures have to be prepared with consideration of the localized plate bending stresses. However, most of the time, critical section is the extremity of the cantilever and bottom flange is in Compression.

Regarding Item 8 of the considerations listed in the survey, buckling of thin bottom flange plates intended to serve predominantly in tension, due to unanticipated or accidental axial compression in the plates during handling and transportation, erection, and in the final service condition

We do not have data - handling, transportation, erection follow the engineered procedures

Some extra comments:

From memory, the problems with the thin bottom flange were at the support where we had difficulties with the flatness of the bottom flange for the bearing assemblies (contact). Our solutions are to mill the diaphragms and to shop weld the sole plates to the bottom flange before manufacturing (welding of the elements together) the box.

In this case, they mention the boxes with bottom flange in tension, which should therefore exclude the boxes above the supports. In the "background", they mention that the industry recommends 3/4 "- 1", which is a good start. In addition, when the bottom flange has welded longitudinal stiffeners, depending on the width of the bottom flange, it should not go less than these values because of the deformations generated by the welding of these stiffeners, which can cause problems of flatness at mechanical joints.

In fabrication, tables from A6 have to be respected (Flatness & Waviness) related to Length and width.

Also some other items to be considered in fabrication:

- Drain pipes - if threaded holes are used need a min thickness of about 5/8 in
- Manutention before assembly - plates less than 5/8 in may be a problem with local distortions (at lifting points)
- CJP for thin plates less than 1/2 in may be more risk for local deformations
- A limit of b/t = 90 seems reasonable. However, as we have for Orthotropic Steel Deck, a thickness of 5/8 in min should also be a good reference

Follow-up: Are there any particular nonproprietary references you would recommend that we should consult regarding state-of-the-art to control welding distortion as it relates to plate thickness and/or plate slenderness (b/t)?

The answer is not easy... I do not remember seeing anything (reference as a public document) related to the present R&D subject. It is really unique to each company, each method of manufacturing and the tools that are used (SAW = more heat input than FCAW). Many parameters are involved ... We often do a mock-up (even when not contractually required) to validate our assumptions for distortion control. This is often the case for movable bridges.

Respondent 22

Regarding Item 1 of the considerations listed in the survey, welding distortion

[redacted] follows AWS 1.5.

Regarding Item 4 of the considerations listed in the survey, vibration of excessively thin plates due to in-service loadings on the completed bridge.

[redacted] follows AASHTO.

Regarding Item 3 of the considerations listed in the survey, localized plate bending stresses due to box-section distortion

[redacted] follows AASHTO.

Regarding Item 6 of the considerations listed in the survey, "bending reluctance" of excessively thin flanges

[redacted] follows AASHTO.

Regarding Item 7 of the considerations listed in the survey, localized out-of-plane to a concentrated transverse load applied to the bottom flange

[redacted] follows AASHTO.

Regarding Item 9 of the considerations listed in the survey, perception of vertical vibrations, or general "sponginess" in bottom flanges of box girders, by bridge inspection personnel.

No observed problems recorded

Regarding Item 10 of the considerations listed in the survey, potential "oil canning" of thin plates in bottom flanges

Problem not noted

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

None

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

Special cases of a box designed as 2 I girders but then the bottom flange is a single plate for the appearance of a box. The flange thickness requirements penalize the bending capacity even though the plate area between the I girders is not needed structurally.

Respondent 23

1. The topic and outcome: This is a valid and needed topic, to add a slenderness limit to steel box girder bottom flanges under tension. The outcome is expected to be in a very simple format – t-min, b/t-max, whichever controls. Do not mix this with compression. Do not cover beyond highway bridges (like "particular industry groups"). I see no need to have an analysis-based limit, hopefully not even any calculation.

2. Answers to the four questions:

   1) In the real world, this issue is usually taken care of by experienced engineers. Very thin/slender plate is a poor design, even though it has not being clearly prohibited by the code. Lacking general guidelines is the reason that we need this research.

   2) It is a common problem for box girder thin plates during fabrication/construction, when the box is handled as open end without adequate bracings. However, more often the more effective remedy is to properly brace/support, not the plate thickness. There is no "good answer" and there should not be an attempt to set a general one, as this a matter of contractors' method and means. A very thick plate may still not be enough for open end erection, and a quite thin plate could be well handled with proper clamping/bracing. This is totally different from final structure performance criteria.

   3) The listed 10 items are valid reasons that we need a limit. Some more could be added, like corrosion, temperature effect. I would rank these items into two groups: (a) under final finished structure, like items 4, 5 (partial), 6 & 7 (design mistakes), 9 (also design mistake); and (b) under temporary condition, like items 1, 2, 3, 5(partial), 8. Group (b) cannot be quantified, until the means and methods are decided – a set limit like $b/t<90$ may not be enough for unbraced open-end construction but may be overly conservative if proper bracing/support is provided during fabrication/erection.

Follow-up: The project team would agree that temporary conditions should not necessarily drive the bottom flange thickness or b/t limits.

Respondent 24

Survey Question 1: Do you have specific or general guidelines employed in your practice, or within your organization, pertaining to any of the 10 areas of investigation/query within the summary?

[redacted] does not have a written guidelines for minimum thickness or maximum slenderness limits.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

No

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

Initial imperfection of thin plate is sensitive to local buckling.

Respondent 25

Survey Question 1: Do you have specific or general guidelines you would recommend from your practice pertaining to any of the 10 areas of investigation/query within the summary? We are particularly interested in gathering any input that may be available regarding the background to the w/t = 80 and t = ¾ inch limits recommended in the Texas Steel Quality Council document on Preferred Practices for Steel Bridge Design, Fabrication and Erection.

The ¾ in. flange thickness limit is based upon cupping of the flanges from the web to flange welds. This is problem which will also occur in boxes. The ¾ in. flange thickness limitation should be imposed for both stiffened and unstiffened flanges bottom box girder flanges.

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

Need consider how cambered boxes are assembled. The camber is cut in the web and lower flange is jacked up against the web to produce the desired girder camber. The jacking force is a concentrated force and will cause local dishing of the flange at the jacking location if the flange is too thin. The ¾ in. limit takes care of the issue.

Limit fillet weld size on thin flanges. Large weld yield larger distortion. Avoid at all cost full penetration welds.

Regarding Item 7 of the considerations listed in the survey, localized out-of-plane deflection due to a concentrated transverse load applied to the bottom flange, or to a plate panel within the bottom flange.

This is very likely to occur in the fabrication of the positive moment section.

Regarding Item 8 of the considerations listed in the survey, buckling of thin bottom flange plates intended to serve predominantly in tension, due to unanticipated or accidental axial compression in the plates during handling and transportation, erection, and in the final service condition, particularly in the vicinity of inflection points in continuous-span girders or near locations where longitudinal stiffeners are terminated.

Handling is the big issue both in the shop and field. Center span field pieces can end up with bottom flange in compression due to erection shoring.

Respondent 26

Survey Question 1: Do you have specific or general guidelines employed in your practice, or within your organization, pertaining to any of the 10 areas of investigation/query within the summary?

Generally follow $b/t < 60$ for compression, and $b/t < 80$ for tension. These limits are noted in the NSBA "Practical Steel Tub Girder Design" for compression and tension, and in the TxDOT Preferred Practices document for tension. The TxDOT document (Section 2.4.1) also notes to not use less than ¾" for the thickness. The compression $b/t < 60$ comes from the AASHTO Standard Spec, 10.39.4.2.4.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

I personally have not encountered any issues, but any designs I have done, checked, or reviewed typically follows the guidance I mentioned in response to Question 1.

Survey Question 3: Could you please rank the 10 items of investigation/query in terms of their importance from your perspective, using the attached ranking sheet? Comments regarding any particular items are welcome.

For me, strength and constructability and ability to fabricate are the most important issues.

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

A few thoughts….. 1) Make sure the b/t limits for stiffened flanges are addressed as well. 2) Are there limits for forces imparted by the inclined webs? 3) Are there limits for torsional shear based on how thin the flange is?

Respondent 27

The last box girder project I worked on was a very large \$432M interchange in [redacted] back in 2009 [redacted]. There were 4 alternative designs that were signed & sealed for the interchange and, unfortunately, a segmental alternative won the bid and was subsequently constructed. However, for the steel alternatives, I was the EOR for two steel box girder flyover bridges that were practically a mile long each. One bridge had 9 continuous units and the other had 7 units. See the attached framing plans, girder elevations, and typical sections for the 16 units. When the project started, the AASHTO LRFD code had not yet incorporated curved steel girders, so we were designing using the 2003 AASHTO Guide Specifications for Horizontally Curved Highway Bridges (LFD). When the LRFD code incorporating curved steel design finally came out, [redacted] did not want to have their first project to be as large as ours was, so we continued with the 2003 guide spec until the end. As for the bottom flange thicknesses and b/t ratios, you can see in the attached plans that our minimum bottom flange thickness was 5/8" and our maximum b/t ratio was 120. I do not recall whether the 2003 guide spec had a limit on b/t for bottom flanges in tension. It might have and, if so, was probably 120 (need to check that). I do not have that old AASHTO guide spec to check. Also, I designed those bridges when I was at one of my previous employers. I can tell you that we designed the bottom flange according to whatever provisions were in that 2003 code for stress, thickness, slenderness, etc. Since these steel bridges were never constructed, I cannot comment on all the fabrication issues in your questionnaire.

Respondent 28

Survey Question 1: Do you have specific or general guidelines employed in your practice, or within your organization, pertaining to any of the 10 areas of investigation/query within the summary?

Generally, there is no engineering analysis performed to determine the expected stability of box girder sections during fabrication or transportation. One exception would be when there is a specific contract requirement for a "transportation plan to be submitted for review", which is rare. It is typically left to the experience of those in the operations area to evaluate how a piece will be handled during fabrication.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

Following are specific cases where relatively thin bottom flanges affected fabrication:

[redacted] Trapezoidal box girders - see attached partial design plans and picture during fabrication. In this case, the 3/4" thick bottom flange "sagged" at the point where the end diaphragm was to be fit & welded. The solution was to clamp a "restraint" while welding. After welding, applied heat was necessary to bring the bottom flange to within flatness tolerances in the area of the bearing device as a result of welding distortion. Something on this girder that is unrelated to the bottom flange is in regard to the 1" thick top flanges. They tended to bend under the load of the girder when handling with "tongs" during moves through the shop and while loading for shipment.

Summary of Data Provided: The bottom flange was 84 inches wide in the plans provided, giving a b/t* of *112.

Test girder for [redacted] research project – see attached shop detail drawings. Note that the thin bottom flange had the same tendency to "sag" and require correction of weld distortion as did the previous example. While this problem is not too difficult to overcome in the shop, the same phenomenon occurs to some degree throughout the length of the girder.

Summary of Data Provided: The bottom flange in these test girders was 37.25" x 7/16" throughout the girder length, giving a b/t* of *85.

Respondent 29

[redacted] has not constructed a steel box (other than straddle bents, which generally have very thick flanges) in more than 10 years.

To my knowledge, there have not been any complaints related to the in-service issues of thin flanges you mention.

We have had some issues with thin webs come to light, but not flanges that I am aware of.

Welding distortion is a common issue we deal with through our Materials QA section.

Statement within the project summary transmitted with the survey: Steel box girders are highly efficient in their structural function, particularly in applications that benefit from their large torsional stiffness:

This typically is used to justify use as a curved structure.

Statement within the project summary transmitted with the survey: However, specifying too thin or too slender of a bottom flange can result in potential problems during fabrication, transportation, erection, or future inspection and maintenance. In addition, excessively thin flanges can lead to concerns regarding strength and fatigue, especially in regions of stress reversal.

How does the thin flange impact torsional resistance? Do we see flange distortion as torsion increases?

Follow-up: Related to your question about the impact of thin flanges on the torsional resistance and the flange distortion as torsion increases, we are aiming to provide some quantification of that. In basic terms, the thin flange influences the girder St. Venant torsional constant via the calculation

$$J = \frac{4A_o^2}{\sum \frac{b}{t}}$$

where A_o is the cross-sectional area enclosed by the mid-thickness of the walls of the box-section member. The St. Venant torsional stress in the box-section flange is then calculated theoretically as

$$\tau = \frac{T}{2tA_o}$$

The thin flange also reduces the overall cross-section resistance to distortion, and if the flange is sufficiently thin, distortional plate bending stresses in the flange can be larger than those in the webs of the cross-section.

Statement within the project summary transmitted with the survey: Specific documentation of quantitative and experiential backing for the above limits generally appears to be lacking.

Boxes are often haunched, the choices are either constant web slope or constant flange width; how do these choices fit in?

Follow-up: The sloped webs reduce the width and the overall b/t of the bottom flange. The sloped webs also can introduce some transverse forces into the bottom flange.

Regarding Item 1 of the considerations listed in the survey, welding distortion:

Minimum thickness of 3/4", not specific to boxes.

Regarding Item 2 of the considerations listed in the survey, vibration of excessively thin plates during transportation:

Not addressed.

Regarding Item 9 of the considerations listed in the survey, perception of vertical vibrations:

Nothing reported that I know of

Regarding Item 10 of the considerations listed in the survey, potential "oil canning"

Some webs reported, no flanges that I am aware of.

Respondent 30

Here are some responses for you from our different areas. Also, I am attaching a spreadsheet for your use.

Summary of Data Provided: The spreadsheet shows bottom flange widths within the positive moment regions of 31 tub-girder bridge designs ranging from 42 to 112 inches, with thicknesses ranging from 5/8 to 1.75 inches and b/t values ranging from 36 to 152. Nine of these cases have b/t values larger than 120. The negative moment sections in the bridges provided have bottom flange widths ranging from 42 to 91.5 inches, with thicknesses ranging from 5/8 to 2 inches and b/t or w/t values ranging from 33 to 105.

From our Fabrication unit:

We haven't experienced bottom flange distortion/warping of steel box girders.

If you are interested - For plate girders, due to some previous issues with warpage on thinner flanges, we have increased our minimum flange thicknesses shown in the [redacted] Standard Drawings to reflect a ¾"minimum.

Note: The standard drawings and Specifications from this owner agency define a plate girder as and I-section member. Therefore, the above requirement is not necessarily applicable to box girders.

From our steel and standards area:

We don't set restrictions, other than AASHTO limits, for b/t ratios. Our in-service bridges, with the b/t ratios on the attached spreadsheet, seem to be performing well.

We have walked through a number of these over the years and have not noticed a "sponginess".

Respondent 31

The [redacted] bridge bottom flange plates are composed of two thicknesses. Bottom flange extending to support and experiencing flexural stresses and significant torsional stress has slenderness of 95. The girders are performing well and none of those concern stated on bulleted items affected the performance of tub-girders.

I am a conservative engineer and believe having a little extra material does not hurt when is justified.

Regarding Item 1 of the considerations listed in the survey, welding distortion:

Distortion is not a huge concern and I have not seen distortion in tub-girders. This is not limited to inclined or plumb webs of tub girders. It should be noted that fabricators are well aware of distortion resulting from weld procedure and adjust welding procedure accordingly. It should be noted that commonly web plate thicknesses are less than flange plate thickness.

Regarding Item 2 of the considerations listed in the survey, vibration of excessively thin plates during transportation.

With having minimum plate thickness requirement in BDS I am assuming the author is referring to slenderness of bottom flange in tub-girders. There are bridge owners requiring contractor provide transportation plan, with supported calculation and specification. I would be more concern about the torsion effect on box girder or tub-girders (specially) during transportation when it was not secured properly.

Regarding Item 3 of the considerations listed in the survey, localized plate bending stresses induced by handling and erection operations.

The contractor spliced tub-girders to the full length prior to erection as permitted in the contract documents. During lifting process one of tub girders dropped from supporting rigs, resulting in web plate bending. We fixed the web with heat straightening procedure, after erection of tub-girder, and checked web plate hardness for serviceability. I am assuming this is not a design issue but it may become analysis requirement.

Regarding Item 4 of the considerations listed in the survey, vibration of excessively thin plates due to in-service loadings on the completed bridge.

I am assuming slenderness of bottom flange plate (in tub-girders) might be a concern. Or boxes in some structures such as tied arch, trusses…. I have not seen any report on in service vibration of plate elements of Tub-girder or box girders (webs or flange/s) even though some had slenderness of more than 100.

Regarding Item 5 of the considerations listed in the survey, localized plate bending stresses due to box-section distortion (i.e., distortion transverse bending stresses) as well as secondary stresses due to out-of-plane deformation of the box-section bottom flange.

I was concerned about the transverse bending stress, torsion of tub-girder during concrete deck placement. Therefore, I find it necessary checking for such stresses that are more concerning in curved and/or skewed bridges. Out of plane deformation of non-composite girders can be a concern and differential deflection in composite girders may results in deck cracking. And result in some torsional stress which might be negligible for non-skewed and curved girder bridges.

Regarding Item 6 of the considerations listed in the survey, "bending reluctance" of excessively thin flanges, that is, the tendency for thin flanges to not bend fully along with the overall curvature of the box-section member.

I am wondering if the effect of interior diaphragm has been considered for such cases. My question might be why interior diagrams with proper spacing are not helpful.

Follow-up: Interior diaphragms are helpful with respect to this consideration.

Regarding Item 7 of the considerations listed in the survey, localized out-of-plane deflection due to a concentrated transverse load applied to the bottom flange, or to a plate panel within the bottom flange.

Negligible out-of-plane deflection may not be a big concern, otherwise question might be fatigue and web plate stresses in such case.

Regarding Item 8 of the considerations listed in the survey, buckling of thin bottom flange plates intended to serve predominantly in tension, due to unanticipated or accidental axial compression in the plates during handling and transportation, erection, and in the final service condition, particularly in the vicinity of inflection points in continuous-span girders or near locations where longitudinal stiffeners are terminated.

I have provided an example of such case resulting web distortion. I understand significance of bottom flange in stress reversal zone however I am wondering why heat straightening should not work. In such case there is need for sufficient guidance.

Regarding Item 9 of the considerations listed in the survey, perception of vertical vibrations, or general "sponginess" in bottom flanges of box girders, by bridge inspection personnel. These vibrations may be due to transient live loads on the bridge or due to the individual's movements when walking the inside of the box girder.

Vibration of superstructures under heavy load is common and can be felt when standing on bridge deck. Composite tub-girder or box girder bridges are experiencing less vibration as whole superstructure. I have not noticed bottom or flange vibration between supporting elements in our bridges under traffic loading.

Regarding Item 10 of the considerations listed in the survey, potential "oil canning" of thin plates in bottom flanges, i.e., snapping in or out of the plates between edge supporting elements when pushed on by a light force.

I have a hard time understanding how an engineer might ok having such design.

Respondent 32

A general answer from the Manager and a couple employees is that they are not aware of any issues with the performance of steel box girder bottom flange.

On the Design side, we follow AASHTO LRFD and have no special requirements for bottom flange b/t ratio in our Design Manual.

Respondent 33

Survey Question 1: Do you have specific or general guidelines you have employed in your practice, or within your organization, pertaining to any of the 10 areas of investigation/query within the summary?

[redacted] has only designed and constructed a few Steel Box girder bridges. [redacted] does not currently have any details or requirements specific to Steel Box Girders. [redacted] does have a minimum ¾" flange thickness and ½" web thickness requirement for structural steel bridges in general.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

I do not believe that [redacted] has had any issues with thin bottom flanges during fabrication, construction or service.

Respondent 34

I have attached the two papers that I can find in my archives regarding plate breathing and fatigue crack. I should be able to send you the [redacted] standard for the box girders by next week.

Summary of Data Provided:

The provided papers are:

Roberts, T.M. and Davies, A.W. (2002). Fatigue Induced by Plate Breathing, Journal of Constructional Steel Research, 58, 1495-1508.

Skaloud, M. and Zörnerovà, M. (2005). The Fatigue Behaviour of the Breathing Webs as Steel Bridge Girders, Journal of Civil Engineering and Management, XI(4), 323-336.

Follow-up: In the second paper, the authors state, in regard to the fatigue behavior of web panels,

"…a stronger limitation of web slenderness (i.e., to slenderness values lower than 175) will be necessary if it is desired to disregard the effects of web breathing in design.

This conclusion is not surprising because web breathing is nothing else than many times repeated web buckling; consequently, all webs prone to buckle are also prone to breath under repeated loads. Of course, in the case of webs with low depth-to-thickness ratios, the effect of buckling is less significant than with slender webs and, hence, also the impact of web breathing must in this case be less pronounced; and for low enough depth-to-thickness ratios it will be entirely negligible…

Among numerous fatigue tests conducted to date in Prague, there were also some for which the upper value of the cycling load was inferior to the critical buckling load; nevertheless, the effect of breathing was still pronounced (even though less so than for higher loading) and therefore not negligible. Even this observation is not surprising, since for a web with initial imperfections the critical load, resulting from the linear buckling theory and related to an "ideal" web without imperfections, does not mean much and, therefore, such webs buckle (and under repeated loads, breathe) even when subjected to loading inferior to the critical load."

The authors proceed to recommend S-N curves directly based on their experimental results.

In the first of these papers, the authors state, "The introduction of ultimate limit states design methods, in particular in relation to the slender webs of plate and box girders, enabled the restrictions on plate slenderness to be relaxed and potentially permitted service loads greater than the buckling loads. This change of design philosophy led to the initiation of research into the influence of plate breathing on the fatigue life of slender plated structures…

In principle it is now possible to perform rigorous assessments of fatigue induced by plate breathing, based on established S-N relationships and theoretical predictions of geometric stress ranges. However, geometric stress ranges depend upon the form and magnitude of initial imperfections, which in general are not known. Also, the theoretical predictions of geometric stresses are considered too complex for routine design. Attempts have been made therefore to simplify the assessment of fatigue associated with plate breathing…

An interim design rule, to limit the occurrence of fatigue damage due to plate breathing, was recently incorporated in Eurocode 3: Part 2. It is recommended that the combinations of membrane compressive and shear stresses in a web panel, $\sigma_{x.Ed.ser}$ and $\tau_{Ed.ser}$, corresponding to the frequent service load combination, are limited by the equation

$$\left[\left(\frac{\sigma_{x.Ed.ser}}{k_\sigma \sigma_e}\right)^2 + \left(\frac{\tau_{Ed.ser}}{k_\tau \sigma_e}\right)^2\right]^{1/2} \leq 1.1$$

where $k_\sigma \sigma_e$ and $k_\tau \sigma_e$ are the corresponding elastic buckling stresses of an assumed simply supported panel."

The authors proceed to recommend an update to the above Eurocode 3: Part 2 formula based on extensive research by multiple investigators, including those of the first second reference above. The above formula is still the formula published in BS EN (2006). AASHTO Section 6 adopts a related approach of:

- *Disallowing theoretical elastic bend buckling of longitudinally unstiffened or longitudinally stiffened webs under longitudinal stresses due to flexure associated with factored construction loading conditions.*
- *Separately disallowing theoretical elastic or inelastic web shear buckling or shear yielding under factored construction loading conditions.*
- *Separately disallowing theoretical elastic bend buckling of longitudinally unstiffened or longitudinally stiffened webs under longitudinal stresses due to flexure under Service II loading conditions.*
- *Separately disallowing theoretical elastic or inelastic web shear buckling or shear yielding in interior panels of webs with transverse stiffeners under the unfactored permanent load plus a factored fatigue live load.*
- *Placing a limit of D/t_w = 150 on webs without longitudinal stiffeners.*
- *Placing a limit of D/t_w = 300 on webs with longitudinal stiffeners.*

The AASHTO provisions do not address interaction between web shear and normal stresses with respect to buckling limit states in any of the above requirements.

Eurocode 3, Part 2 does not specify any restrictions on theoretical buckling of box section member flanges under any loading conditions. The Eurocode provisions consider the flange postbuckling resistance.

The AASHTO (2017) LRFD Specifications restrict the compression flange of composite box girders to a buckling or yielding resistance, including the interaction of longitudinal stresses due to flexure and shear stresses due to torsion under Strength loading conditions.

The provisions for square and rectangular HSS in the AASHTO (2017) LRFD Specifications consider the flange postbuckling resistance in flexure, but do not account for any interaction with shear within the flanges.

Proposed unified AASHTO LRFD provisions for all types of rectangular box-section members, including HSS, limit the flange and web longitudinal stresses under general axial compression and biaxial flexure to the theoretical compression buckling resistance under (1) the factored construction loadings, and (2) Service II loadings. These provisions also separately disallow theoretical elastic or inelastic shear buckling or shear yielding in flanges under (1) the factored construction loadings and, (2) the permanent dead load plus a factored fatigue live load. Similar to the AASHTO (2017) provisions for I- and box-girder webs, the philosophy of these provisions is that these separate theoretical plate compression buckling and shear buckling limit limits are sufficient to guard against any adverse effects of plate local buckling, without considering any interaction between the theoretical compression buckling and shear buckling resistances.

Follow-up: We're wrapping-up on synthesis of input we've received regarding limits and behavior of steel box-girder bottom flanges. If you are able to send us some additional information, that would be excellent.

I had to pull some of the original calculations for the [redacted] box girders. As discussed in our last meeting [redacted] has two types of steel box girders; they are referred as standard and non-standard girders. The basis of the standard girders was a finite element conducted by Professors J.G Bouwkamp and E.L Wilson at the University of California, Berkeley; the non-standard girders were designed by different consultants as the need arises.

A review of the calculations for the standard and several non-standard does not show any guidelines nor limits on the b/t ratio for the bottom flanges; the minimum thickness for the bottom flange is ¾" and the width is 84." I have attached the [redacted] standard drawings for the standard girder and a drawing for a non-standard girder; the main difference between the standard and the non-standard is the length/depth and the end condition of the girder. Regarding the ten areas of study, I have to say that we have not experienced any problems related to those areas. Our longest box girder (140') has a set of damping layers attached to the bottom flanges and the webs to control the girder vibration (IMG 0033). We have experienced web gap cracking at several of the non-standard (see top flange and bottom flange photo) girders.

I have scanned sections of the original calculations and reports for the standard box and one standard box and I will put them in a dropbox folder for your use.

Please let me know if you need more information and I hope that above was helpful.

It appears that the Adobe cloud, shared file, did not work; I have attached a scanned copy of the calculations. I did not include the sections on the shear connectors and the miscellaneous.

Summary of Data Provided: The standard girder drawings indicate unstiffened bottom flange thicknesses ranging from ¾ to 2 inches, b = 84 inches and with b/t values ranging from 42 to 112. The nonstandard girder drawings indicate 84 inch wide unstiffened bottom flange thicknesses ranging from 1.75 to 2 inches, and b/t values ranging from 42 to 46.

The photos and reports provided indicate web gap fatigue cracking in the bottom and top of the box girder webs of the non-standard design. There does not appear to be any clear correlation between the fatigue cracking and the slenderness of the bottom flanges.

Respondent 35

I have to confess that I do not have much guidance to offer on this subject. The thin plates that I have encountered have been in stiffened panels for bottom flanges of aerofoil box sections for suspension bridges and also for footway deck plates. The minimum thickness has been 9mm typically (Humber, Storebaelt, Canakkale) with a maximum panel size of 650mm (b/t max = 72) and typically in the 600mm range. I am not aware of any significant issues that have been encountered with panel slenderness of this magnitude and would note that these panels will be experiencing both tensile and compressive loads as unstressed under permanent loading. It has been girder webs that I understand have suffered more form excessive slenderness when 'web breathing' has occurred under load. The same could manifest itself for bottom flanges also.

Can you advise what has prompted the call for this guidance? Have there been instances where boxes have not performed adequately in service due to excessively thin flange plates?

Follow-up: Thank you for this information. It is good to understand the values of the related design parameters for these signature bridges.

NCHRP 20-07/415 is a small study in response to concerns/questions raised as to whether a simple not-to-exceed limit was prudent for bottom flanges subjected predominantly to tensile stresses. The context has been chiefly tub-girders in positive bending, where in many cases in US practice, the bottom flanges do not have any longitudinal stiffening. Guidance such as in the attached document (p. 2-17) limit the bottom flange thickness to ¾ inch (19 mm) and the bottom flange width to thickness ratio to 80. Thicknesses as low as ½ inch (13 mm) and width-to-thickness values of 120 to 140 are not uncommon in US practice with these components in these types of bridges. Our charge is to collect input, perform a few basic analytical studies to complement this input, and provide a recommendation.

Respondent 36

Thanks for the opportunity to provide input. I'm sorry that I didn't reply last fall.

…I didn't rank your 10 values because I haven't encountered issues at all with most of them, making them hard to rank. However, if there is still time, I can reach out to our transportation folks and see if they have encountered any of the transportation issues that you list.

Follow-up: That would be great to hear from your transportation folks about any experiences they've had with the issues listed. (No response received to the follow-up at the time of preparation of the final report.)

Regarding your specification limit [pertaining to the industry recommendations on minimum bottom flange thickness], I support ¾" thick as a minimum but suggest stronger language supporting 1". Maybe we should use ¾" min, 1" preferred… ¾" is generally workable, with some remediation needed, but 1" is better.

Survey Question 1: Do you have specific or general guidelines employed in your practice, or within your organization, pertaining to any of the 10 areas of investigation/query within the summary?

In box fabrication, as with other types of bridge fabrication, plate distortion from welding must be managed to achieve the final intended geometry and condition of the box, particularly regarding plate flatness. Plate flatness is not only a concern of final condition, but also lack of plate flatness can also increase the effort needed to assemble components.

From shop experience, we see that the propensity for plates to distort from welding is dependent upon

- plate thickness: the thinner the plate, the more likely it is to distort and the greater the distortion;
- weld size: the larger the weld, the more heat and initial plate expansion from welding; and
- stiffener use: stiffeners help control distortion.

Designs sometimes call for longitudinal stiffeners when thin flanges are used. From a shop constructability (fabrication ease and cost-effectiveness), it is better to increase flange thickness such that stiffeners can be avoided than use thin flanges and stiffen them. The combined weight and stiffeners of a stiffened flange may be less than the weight of an unstiffened flange, which would save material cost, but unless the flange must be very thick to avoid use of the stiffeners, the trade-off does not result in cost savings due to the labor needed to attach the stiffeners.

To control distortions from welding a minimum tub girder bottom flange thickness of one inch is preferred, but ¾" is generally workable. At ¾" some distortion will result, and some corrections may be needed to satisfy flatness requirements, particularly at bearings. This effort goes up considerably when thinner flanges are used.

Stiffener thickness can also cause issues for welding if they are too thin. When stiffeners are thinner than ½" thick, the welds that attach the stiffeners can "bridge" (touch, so to speak) beneath the stiffener and cause lack of fusion. This is particularly known to be a problem when Dart welding stiffeners to girder webs, particularly because in Dart weld it is common for the welding arcs to be directly opposite each other during weld. Dart welding uses submerged arc welding

(SAW), and Dart welds are usually 5/16” fillets. Dart welding is not used for attaching longitudinal stiffeners, and therefore welding arcs can be staggered, which helps avoid bridging, but Dart welding would certainly be preferred for transverse stiffeners.

Survey Question 2: Have you or your organization encountered any design projects, or existing bridges, where thin bottom flanges in composite or noncomposite box girders have led to problems during fabrication, construction or service of the bridge?

As described in the question 1 response, thinner flanges create challenges dealing with distortion. On recent and current tub girder projects, bottom flange thickness have been at least 1” thick, and these are not a problem.

Also as described in the question 1 response, use of longitudinal stiffeners on thin flanges creates extra work and therefore are not preferred. However, installing these is not a “problem”- just extra effort and cost.

Survey Question 3: Could you please rank the 10 items of investigation/query in terms of their importance from your perspective, using the attached ranking sheet? Comments regarding any particular items are welcome.

I have only encountered two of these issues – welding distortion and, on very thin flanges, oil canning. However, if more time is available, I can check with our transportation folks and see if they have encountered any of the issues associated with transportation.

Survey Question 4: Are there any areas of concern other than the 10 listed in the summary that you would suggest that we consider in this project?

No.