APPENDIX H

DESIGN CRITERIA FOR NCHRP 12-79 PROJECT NEW BRIDGE DESIGNS

This appendix summarizes the criteria applied for the design of new hypothetical bridges considered in NCHRP 12-79's Task 7 parametric studies. The various considerations are presented in an overall outline form.

H.1 GENERAL

In general all requirements of the AASHTO *LRFD Bridge Design Specifications*, 4th edition, with 2008 interim revisions were followed. Where specific AASHTO guidelines were not available, or where the AASHTO specifications allowed for designer discretion, then the guidelines listed below governed.

In addition to the specific guidelines enumerated below, the parametric study bridge designs also followed generally accepted design and detailing practices. Typical reference documents used (and cited below) included, but were not limited to:

- Texas Steel Quality Council, Preferred Practices for Steel Bridge Design, Fabrication, and Erection, (TxDOT 2005),
- AASHTO/NSBA G12.1-2003, Design for Constructability,
- HDR, *Bridgeline* (Various editions)
- AASHTO, GHC-4 (2003), Guide Specifications for Horizontally Curved Steel Girder Highway Bridges with Design Examples for I-Girder and Box-Girder Bridges,
- NHI Course 130081 Design Manual (NHI 2007), and
- NSBA, Steel Bridge Design Handbook, Chapter 8 – Stringer Bridges, 2006.

H.2 MATERIALS

All structural steel was assumed to be ASTM A 709, Gr. 50 W except as follows:

- Any "existing bridges" included in the study were modeled using their specified materials.
- HPS 70W was not considered in the "non-existing" parametric study bridges for the following reasons:

  1. Hybrid girders are not widely used

  2. Hybrid girders introduce another level of complexity to the overall problem statement. It was decided that analysis trends would be easier to see without adding the question of "how does the use of hybrid girders affect the results?"

H.3 FATIGUE

The designers assumed good detailing practices were followed and designed girders assuming Category C' for transverse stiffener-to-flange and transverse stiffener-to-web weld conditions controlled (ref.: AASHTO LRFD Table 6.6.1.2.3-1)

H.4 LIVE LOAD DEFLECTIONS

In order to follow design practices which are still prevalent throughout the US, the parametric study bridges generally were designed to comply with the optional live load deflection control criteria of AASHTO LRFD § 2.5.2.6.2. The more stringent criteria for bridges subjected to pedestrian loads were not considered. These criteria were used as "guidelines" not as absolute limits; deflections as much as 10% beyond the AASHTO criteria were considered acceptable.

H.5 DECK DESIGN

Assumed a 9 1/2" thick concrete deck (including a ½" sacrificial wearing surface), f'_c = 4.0 ksi, nominal unit weight 0.150 kcf.

H.6 DECK LONGITUDINAL REINFORCING

The guidance provided by AASHTO LRFD § 6.10.1.7 was followed.

In the design of the girders in the negative moment regions (i.e., in the girder resistance checks, but not in the structural analysis as outlined below), the deck was considered ineffective. However, the longitudinal deck reinforcing was considered effective.

For all analyses, as suggested in AASHTO LRFD C4.5.2.2, uncracked section properties were assumed for the entire deck (in both positive and negative moment regions).

H.7 SHEAR CONNECTORS

The basic assumption was that shear connectors were provided throughout the length of all bridges, based on the recommendations in AASHTO LRFD § 6.10.10.1.

H.8 INTERMEDIATE DIAPHRAGMS / CROSS FRAMES FOR I-GIRDER BRIDGES

General Configurations:

- Girders with Web Depth $\leq$ 48": Solid diaphragms are common (ref.: Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, 2005, pg 2-25, AASHTO/NSBA G12.1-2003, pp 20-21, NCDOT *Structure Design Manual*, 2007, pg 6-36 and Fig. 6-98). For consistency in this study, assumed bent plate diaphragms, depth approximately 80-90% of web depth.
- Girders with Web Depth > 48" and spacing/web depth (s/d) ratio $\leq$ 1.5, assumed X-frame cross frames with top and bottom chords (ref.: HDR *Bridgeline*, Vol. 13, No. 1, pg. 3 and AASHTO/

NSBA G12.1-2003, pg. 20). Used single angle sections for chord and diagonals if possible; used WT sections for chords and diagonals if absolutely necessary by design.

- Girders with Web Depth > 48" and s/d ratio > 1.5, assumed inverted K-frame cross frames with top chords (ref.: HDR *Bridgeline*, Vol. 13, No. 1, pg 3 and AASHTO/NSBA G12.1-2003, pg 20). Used single angle sections for chord and diagonals if possible; used WT sections for chords and diagonals if absolutely necessary by design.

Addressing "Nuisance Stiffness" Issues in Skewed Bridges:

- Selectively omitted diaphragms / cross-frames near supports in order to reduce the effects of undesirable transverse stiffness ("nuisance stiffness"). Followed suggestions in the article "Nuisance Stiffness" (HDR *Bridgeline*, Vol. 4, No. 1, 1993, pp 1-3). For consistency among designs in the research project, use of "lean-on" bracing concepts was considered only on a limited basis (this approach is not yet fully implemented nationally).

Spacing:

- Cross frame / diaphragm spacing in horizontally curved bridges was selected so as to limit lateral flange bending stresses. The spacing was initially determined using Eq. C9-1 in AASHTO GHC-4 (2003), with the target bending stress ratio $r_\sigma = | f_\ell / f_b |$ set at 0.30. The spacing was generally limited to a maximum value of 25' in order to result in a reasonably "typical" framing plan.
- Cross frame / diaphragm spacing in tangent, skewed bridges was selected to maximize the spacing. The general target was approximately 25' spacing.

Design:

- Generally followed design procedures as presented in HDR cross frame design spreadsheets developed for recent design projects, but consideration of connection details and their design were omitted. The focus was only on chord and diagonal member design in order to establish reasonable member sizes for use in the analysis models.

Connections:

- It was assumed that all connections were fully effective for analysis modeling purposes.

Cross Frame / Diaphragm Modeling in 2D and 3D Analysis Models:

- In 2D Grid analysis models, truss-type cross frames must be modeled using an equivalent single line element. The cross sectional properties for the equivalent line element were determined using one of the two methods outlined in *Analysis of Steel Girder Bridges – New Challenges* (Coletti and Yadlosky 2007). The specific choice of the "shear stiffness" approach was made early on in Task 7 following a brief study.
- In 2D Grid analysis models, plate-type diaphragms were modeled using an equivalent single line element. The determination of the cross sectional properties for that single line element was a

relatively straight-forward direct application of the cross sectional properties of the actual diaphragm.

- In 3D FEM models, the cross frame or diaphragms were modeled in detail, with each of the chords and diagonals of truss-type cross frames modeled directly, and with the full web depth of plate-type diaphragms modeled directly, with flanges modeled using line elements on the top and bottom of the web plate element.

H.9 END DIAPHRAGMS / CROSS-FRAMES FOR I-GIRDER BRIDGES

As a general rule, the same guidelines were followed as for intermediate diaphragms / cross-frames for I-girder bridges, except that the top chord was assumed to be a channel section extending up to support the edge of the deck (ref.: AASHTO/NSBA G12.1-2003, pg 22).

H.10 PIER DIAPHRAGMS / CROSS-FRAMES FOR I-GIRDER BRIDGES

As a general rule, followed the same guidelines as for intermediate diaphragms / cross-frames for I-girder bridges.

H.11 HORIZONTAL LATERAL BRACING FOR I-GIRDER BRIDGES

Horizontal lateral bracing was provided when necessary; the initial guideline assumption was that it would be considered primarily for the 350' span range I-girder bridges. As a design goal, lateral bracing requirements were met whenever possible by providing only top flange lateral bracing; this is consistent with current common design practice and is done to avoid the situation of bottom flange lateral bracing which is subject to significant live load effects after deck placement. The extent of lateral bracing was limited to a minimum number of bays and panels required to achieve stability and control stresses induced by wind loading. In general, the guidance in the NSBA Steel Bridge Design Handbook Example 1, pp. 50-52 was followed. As a design target, lateral bracing was designed to limit lateral deflections caused by wind loading to a value of approximately L/300 (ref.: 2006 draft of NHI Course 130081 Design Manual, Vol. 1, pg. 2.69).

H.12 INTERNAL INTERMEDIATE DIAPHRAGMS FOR TUB-GIRDER BRIDGES

In general, the suggestions and references presented by Coletti, et al. (2006), Practical Steel Tub Girder Design, regarding spacing and sizing of internal intermediate diaphragms were followed. In particular, a likely spacing configuration was to set the top flange lateral bracing bay spacing approximately equal to the tub girder internal top flange center to center web spacing, and to provide internal intermediate diaphragms at a spacing double that of the top flange lateral bracing bay spacing. The internal intermediate diaphragms consisted of inverted K-frames, with the top chord forming part of the top flange lateral bracing system. Typically the top chord was a WT section, while the diagonals were angle sections.

Member loads in internal intermediate diaphragms were calculated as follows:

- For Approximate Method (M/R) and 2D Grid Analysis models: Followed the guidance offered by Fan and Helwig (2002), "Distortional Loads and Brace Forces in Steel Box Girders."

- For 3D FEM analysis models: Direct force results were obtained from the model.

Chord and diagonal members were designed, but consideration of connection details and their design were omitted.

H.13 EXTERNAL INTERMEDIATE DIAPHRAGMS FOR TUB-GIRDER BIRDGES

In general, the suggestions and references presented by Coletti, et al. (2006), Practical Steel Tub Girder Design, regarding spacing and sizing of external intermediate diaphragms were followed. In addition, recent work by Helwig, et al . (2007), in *Design Guidelines for Steel Trapezoidal Box Girder Systems*, was followed to perform preliminary calculations related to the need and suggested spacing of external intermediate diaphragms. In general, full depth truss-type diaphragms (inverted K-frame with top chords) were used and were assumed to remain in place after deck placement. Partial depth plate-type diaphragms are becoming more popular recently, but to date full depth truss-type diaphragms have been more widely used. WT and angle sections were considered for the chords and diagonals based on loading and detailing requirements.

Member loads in internal intermediate diaphragms were calculated as follows:

- For Approximate Method (M/R) models: N/A. The M/R Method was typically limited to use on single tub-girders.
- 2D Grid Analysis models: Used the procedures recommended by and associated with the MDX program for converting internal forces in the equivalent line elements used for diaphragm modeling to member forces in the actual truss-type external diaphragms.
- For 3D FEM analysis models: Direct force results were obtained from the model.

Chord and diagonal members were designed, but consideration of connection details and their design were omitted.

H.14 INTERNAL AND EXTERNAL DIAPHRAGMS AT SUPPORTS

In general, the suggestions and references presented by Coletti, et al. (2006), Practical Steel Tub Girder Design, regarding sizing of internal and external diaphragms at supports were followed. Full depth plate diaphragms were used for both internal and external diaphragms at supports. Top and bottom flanges for these diaphragms were discontinuous across the entire width of the girder system, following recent research and recommendations by Helwig, et al. (2007), *Design Guidelines for Steel Trapezoidal Box Girder Systems*.

Member loads in internal and external diaphragms at supports were calculated as follows:

- For Approximate Method (M/R) models and 2D Grid Analysis models: Followed the guidance presented by Coletti, et al. (2006), Practical Steel Tub Girder Design, for evaluation of simple free body diagrams.
- For 3D FEM analysis models: Direct force results were obtained from the model.

Overall flange sizes and web thicknesses were designed, but consideration of connection details and their design as well as consideration of access openings (manholes) was omitted.

H.15 TOP FLANGE LATERAL BRACING

In general, the suggestions and references presented by Coletti, et al. (2006), Practical Steel Tub Girder Design, regarding spacing and sizing of top flange lateral bracing were followed. In particular, a likely spacing configuration was set the top flange lateral bracing bay spacing approximately equal to the tub girder internal top flange center to center web spacing, and to provide internal intermediate diaphragms at a spacing double that of the top flange lateral bracing bay spacing. In general, WT and angle sections were used to form the top flange lateral bracing system. The top flange lateral bracing system conformed to a Warren Truss arrangement. Pratt Truss configurations were not used unless modeling an "existing bridge" which used such a configuration.

Member loads in the top flange lateral bracing system were calculated as follows:

- For Approximate Method (M/R) models and 2D Grid Analysis models: Followed the guidance presented by Fan and Helwig (1999), "Behavior of Steel Box Girders with Top Flange Bracing."
- For 3D FEM analysis models: Direct force results were obtained from the model.

Top flange lateral bracing members were designed, but consideration of connection details and their design as well as consideration of access openings (manholes) was omitted.

H.16 GIRDER DESIGN PERFORMANCE RATIOS

In general, all performance ratios (demand/capacity) were kept at or below a maximum value of 1.0.

H.17 INELASTIC DESIGN / MOMENT REDISTRIBUTION

Inelastic design and moment redistribution (as provided for in Appendix B6 of AASHTO LRFD) was generally not considered. All of the parametric study bridges were designed to meet all requirements while remaining fully elastic. If an existing bridge was designed using inelastic design or moment redistribution provisions, that design however, was not changed (none of the bridges considered were designed in this way).

H.18 GIRDER SPACING FOR I-GIRDER BRIDGES

Two main deck widths were proposed for the study bridges: 30' and 80'.

- 30' Deck Width Bridge: Used 3'-6" overhangs with 3 girders spaced at 11'-6".
- 80' Deck Width Bridge: Used 3'-6" overhangs with 7 girders spaced at 12'-2".

(ref.: AASHTO/NSBA G12.1-2003, pg 1)

H.19 GIRDER DEPTH FOR I-GIRDER BRIDGES

- Variable web depth girders were not considered.
- Target span/depth ratio: AASHTO LRFD Table 2.5.2.6.3-1 suggests minimum ratios for simple spans and continuous spans for both the noncomposite steel section depth and the overall composite section depth. However, a more practical target is to use the recommendation for minimum total composite section depth as the recommended web depth (ref NSBA SBDH Example 1, pg 8). Therefore, for simple spans targeted a web depth of 0.040L and for continuous spans target a web depth of 0.032L, where L is the total span length.

H.20 WEB SIZING FOR I-GIRDER BRIDGES

In general, web thickness was selected to result in an unstiffened or "partially stiffened" web design (ref.: Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, 2005, pg 2-10; *NSBA Steel Bridge Design Handbook, Chapter 8 – Stringer Bridges*, pg. 8-11, NHI Course 130081 Design Manual, Volume 2, pg 2.124).

H.21 FLANGE SIZING FOR I-GIRDER BRIDGES

Flange widths were generally set to be roughly 20% to 30% of the web depth (the Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, 2005, pg 2-7 recommends 30% or greater, but many designers contacted feel this is too wide). Curved girders may tend to the wider end of the above range, while straight girders may tend toward the narrower end of this range. This is wider than the AASHTO LRFD 6.10.2.2 specified minimum flange width, $b_f \geq D/6$. These were not considered as absolute limits, and engineering judgment was exercised to develop designs which satisfied current norms for constructible, economical designs. Top flanges were generally different widths than bottom flanges. In general, bottom flange widths were held constant over the entire length of a bridge, while top flange widths were allowed to change at field splices if warranted. In addition, the ratio of field section length, L, to flange width, b, was not allowed to exceed 85, i.e., $L/b \leq 85$ (ref.: AASHTO LRFD Eqn. C6.10.3.4-1; Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, 2005, pg 2-7; other references).

The absolute minimum flange width was 12" (ref.: NHI Course 130081 Design Manual, Volume 2, pg 2.113). The typical minimum flange width was 14".

Typically, as is relatively commonly accepted for composite construction, the bottom flange was typically wider than the top flange (ref.: NHI Course 130081 Design Manual, Volume 2, pg 2.118).

Minimum flange thickness was ¾" (ref.: NHI Course 130081 Design Manual, Volume 2, pg 2.116; AASHTO/NSBA G12.1-2003, pg 2).

Flange transitions were addressed on a case by case basis typically following engineering and fabrication suggested guidelines such as those found in AASHTO/NSBA G12.1-2003, pp 5-6 and *NSBA Steel Bridge Design Handbook, Chapter 8 – Stringer Bridges*, pg. 8-8.

H.22 GIRDER SPACING FOR TUB-GIRDER BRIDGES

Two main deck widths were proposed for the study bridges: 30' and 80'.

- 30' Deck Width Bridge: Used 3'-6" overhangs with 2 girders. The girders had a C-C web spacing of 7'-6" to 8'-6", depending on the web depth which affects bottom flange width. An absolute minimum bottom flange C-C web spacing of 4'-0" was observed (ref.: Coletti, et al. (2006), Practical Steel Tub Girder Design) with a minimum bottom flange C-C web spacing of 4'-6" desired.
- 80' Deck Width Bridge: Used 3'-6" overhangs with 4 or 5 girders. Girders typically had a C-C web spacing of 8'-0" to 10'-6", depending on the web depth which affects bottom flange width. Shorter span bridges were assumed to likely have 5 narrower girders, while longer span bridges would likely have 4 wider girders.

H.23 GIRDER DEPTH FOR TUB-GIRDER BRIDGES

- Variable web depth girders were not considered.
- The guidance provided in Coletti, et al. (2006), Practical Steel Tub Girder Design, was followed; minimum web depth was 5'-0". Target steel girder section depth ranged between roughly L/25 and L/35, where L is the span length for simple spans, and 0.80 times the span length for continuous spans. The shallower end of this range was the preferred design target in general.

H.24 WEB SIZING FOR TUB-GIRDER BRIDGES

In general, web thickness was selected to result in an unstiffened or "partially stiffened" web design (ref.: Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, 2005, pp 2-10 & 2-17; *NSBA Steel Bridge Design Handbook, Chapter 8 – Stringer Bridges*, pg. 8-11, NHI Course 130081 Design Manual, Volume 2, pg 2.124).

H.25 FLANGE SIZING FOR TUB-GIRDER BRIDGES

Top flange widths were set following guidance similar to that followed for I-girder flanges.

The minimum flange width for 150' span bridges was 14". For the 250' and 350' span bridges, the minimum top flange width was 20". At 14", gusset plates would be required for connection of the top flange lateral bracing. At 20", the top flange lateral bracing members can be bolted directly to the girder top flange, which is preferred.

Bottom flange widths were set 4" wider than the C-C web spacing at the bottom flange.

Minimum flange thickness was ¾" (ref.: NHI Course 130081 Design Manual, Volume 2, pg 2.116; AASHTO/NSBA G12.1-2003, pg 2; Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, 2005, pg 2-16).

The bottom flange b/t ratio was limited to 80 in positive moment regions, a limit which has been cited by fabricators as helpful in avoiding problems with distortion of the bottom flange during welding (Texas Steel Quality Council, *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, 2005, pp 2-16). Older AASHTO proposed guide specifications have suggested a maximum b/t limit of 120, but 80 is considered more representative of current practice.

Flange transitions were addressed on a case by case basis typically following engineering and fabrication suggested guidelines such as those found in AASHTO/NSBA G12.1-2003, pp 5-6 and *NSBA Steel Bridge Design Handbook, Chapter 8 – Stringer Bridges*, pg. 8-8.

H.26 STIFFENERS

Stiffeners and related details (transverse intermediate stiffeners, bearing stiffeners, cross frame connection plates, etc.) were designed using the current AASHTO LRFD criteria. Stiffener spacing, as it affects the shear capacity of the girder web, were determined as part of the design.

The use of longitudinal web stiffeners was avoided for the 150' and 250' span length parametric bridges. Longitudinal stiffeners were considered as appropriate for the 350' span I-girder bridges.

The use of longitudinal bottom flange stiffeners was avoided for the tub girder bridges, since the bottom flange b/t ratios were anticipated to be well below the range where longitudinal flange stiffeners offer benefits. Most of the tub girders studied had relatively narrow bottom flanges.

H.27 BEARINGS

It was assumed that all bridges were supported on steel-laminated elastomeric (neoprene) bearings, with one bearing per girder for both I-girders and tub-girders.

Bearing restraints were determined on a case by case basis, but generally followed common design practices such as:

- Longitudinal direction: One set of "fixed" bearings was provided at one support; bearings at all other supports were "free" or "guided."
- Transverse direction: Generally accepted good practices were followed in deciding bearing fixity details. In most cases, one bearing per support was "fixed"; all other bearings were "free" or "guided."
- Curvature: The direction of longitudinal movement for "guided" bearings was oriented along the direction of anticipated thermal movement (ref.: Coletti and Yadlosky (2007), "Analysis of Steel Girder Bridges – New Challenges." Consideration was given to the use of circular bearings in cases of severe curvature or skew.

H.28 DESIGN FOR STEEL ERECTION

In general, the girders were sized and the erection schemes were determined such that the erection conditions would not control the girder sizing. However, as erection stresses were determined through the study, if it became necessary to resize girders in order to satisfactorily address an erection situation, the typical decision was to resize the girder rather than to reconfigure the erection scheme. All models of a given bridge were adjusted as necessary in that scenario. This is consistent with current AASHTO design guidance placing responsibility on the Engineer of Record to determine at least one constructible erection scheme as part of the design process (reference AASHTO LRFD §2.5.3). In some extreme cases where constructability was clearly anticipated to be a controlling factor in the design, the erection analysis was conducted before, or in parallel with, the design modeling.