Designers' Guide to Cross-Frame Diaphragms

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INTRODUCTION

The Nature of Design

Design is a series of compromises. This generalization is especially true for the design of intermediate cross-frame diaphragms of tangent steel plate-girder bridges. The compromises are not compromises of safety or serviceability, but compromises of economy.

One potential designer's compromise is to design a simple cross-frame diaphragm which is easy and economical to fabricate but requires more effort, and thereby costs, during erection. The ultimate extension of this example is no permanent cross-frame diaphragm at all, infinitely cheaper to fabricate, but requiring temporary bracing during construction. Conversely, traditional regularly spaced cross-frame diaphragms can be relatively costly to fabricate, yet may be much more easily erected.

This is the overriding question for the designer: what compromise, or design solution, represents the least total cost?

Evaluating alternatives in terms of total cost can be very challenging for the designer. Costs are relative for each different contractor, fabricator or erector. Design/build projects represent the ideal environment for the designer to customize the bridge design to the particular contractors' economies. The traditional project-delivery methods, in which the bridge designer does not know the ultimate contractor, does not foster least cost solutions.

In some cases, costs are not necessarily actual costs as much as costs as perceived by the contractor. Further, new and different fabrication and erection techniques, and the unknown that they represent to the contractor, are perceived as more costly, at least initially.

Scope

This guide is limited to the discussion of intermediate cross-frame diaphragms for tangent steel plate-girder bridges with little or no skew (say, skew angles equal to or less than 20^{o}) and without longitudinal stiffeners. Intermediate diaphragms in curved or severely skewed girders can carry more significant loads than the cross-frame diaphragms discussed herein. Also, end diaphragms, those placed over abutments and piers, serve other functions than intermediate diaphragms. Finally, rolled-beam girders do not typically require cross-frame diaphragms with simpler solid, bent-plate diaphragms being an alternative.

FUNCTIONS OF INTERMEDIATE CROSS-FRAME DIAPHRAGMS

Intermediate cross-frame diaphragms of composite steel girder bridges can serve two distinct functions. They can:

1. brace the girders' compression flanges, and

2. distribute loads among the girders.

Bridge erectors also depend upon traditional, regularly spaced cross-frame diaphragms to simplify girder erection.

Load Distribution

DISTRIBUTION OF GRAVITY LOADS

The role of intermediate cross-frame diaphragms in live-load distribution has long been argued.

The recent National Cooperative Highway Research Program (NCHRP) Project 12-26, resulting in the *Guide Specifications for Distribution of Loads for Highway Bridges*, ignored any contribution of cross-frame diaphragms to live-load distribution. As such, the application of these distribution-factor equations assumes no regularly spaced cross-frame diaphragms are provided.

The *LRFD Specifications* include an adaptation of the *Guide Specifications*' distribution-factor equations with an additional provision included to account for the presence of cross-frame diaphragms. The participant's notebook for the Federal Highway Administration (FHWA) training course on LRFD for highway bridge superstructures presents a discussion on the apparent differences between the distribution-factor equations of the *Guide Specifications* and the *LRFD Specifications*. Article 4.6.2.2.2d of the *LRFD Specifications* includes the supplemental provision intended to account for the effect on exterior girders of significant, regularly spaced cross-frame diaphragms. Herein, the entire bridge cross section is assumed to deflect and rotate as a rigid body. The outlined procedure is analogous to the approximation conventionally applied to loads on pile groups. If significant, regularly spaced cross-frame diaphragms are not provided in the design, this supplemental provision should be ignored.

Thus, both the *Standard Specifications* via the referenced *Guide Specifications* and the *LRFD Specifications* provide load distribution-factor equations for the situation where regularly spaced cross-frame diaphragms are not provided.

DISTRIBUTION OF LATERAL LOADS

Intermediate cross-frame diaphragms are thought to resist lateral loads. These lateral loads include wind loads, construction loads and vehicular collision loads. Intermediate cross frames do not affect seismic response of steel bridges under lateral earthquake loads.

Wind Loads

Cross-frame diaphragms can help resist the lateral wind loads on steel girder bridges. Wind loads on steel girder bridges are typically assumed to be distributed as follows:

- wind on the upper half of the web of fascia girders is assumed to be resisted directly by the concrete deck, and

- wind on the lower half of these webs is assumed to be resisted by flexure of the bottom flange, either between brace points (cross-frame diaphragms or wind bracing) or bearings.

Thus, while diaphragms can transfer load from one girder to another and into the concrete deck or bearings, other load paths of designed appropriately, are available in the absence of cross-frame diaphragms. The girder flange can be designed to resist the lateral wind load in cross bending between the bearings, or bracing can be provided specifically for wind.

Vehicular Collision Loads

The opinions of bridge engineers vary regarding the effects of intermediate cross-frames on steel-bridge response under lateral collision loads due to over-height vehicles or substandard vertical clearances. Some view regularly spaced cross frames as beneficial under such lateral load. Others view them as detrimental. No collected performance data exists to valid or quantify either viewpoint.

The *LRFD Specifications* are silent regarding vehicular collision loads due to over-height vehicles or substandard vertical clearances of steel bridges. The only vehicular collision load specified in the *LRFD Specifications* is for abutments and piers as specified in Article 3.6.5.2, entitled Vehicle and Railway Collision with Structures.

Regularly spaced cross frames can distribute lateral loads among all of the girders of the bridge cross section. Unfortunately, due to the rapid rate of loading of the collision load, namely an impact, the distribution is not very effective. If the collision impact occurs near a cross frame, the steel around the cross-frame connection has a tendency to tear before the load can be distributed to the other members. If the collision impact occurs near mid-span of the bottom flange between cross frames, the flange as a tendency to bend and plastically deform before the lateral load can be distributed to the other girders.

When vehicular impacts near a cross frame tear the steel near its connection to the girder, little alternative exists to cutting the torn material out and replacing the steel through a field-welded repair. Such a field-welded repair offers reduced fatigue resistance under normal service, and the potential for brittle fracture under a second lateral impact load.

An impact away from a cross frame which results in plastic deformations can be much more easily and reliably repaired through heat straightening.

Unfortunately, regularly spaced cross-frame diaphragms are relatively ineffective in distributing the lateral impact loads due to vehicular collisions with the bottom flange of steel girder bridges. More closely spaced cross-frame diaphragms should not be considered to resist accidental vehicular collision loads.

Bracing of Compression Flanges

The truly essential function of traditional cross-frame diaphragms is to stabilize the girders' compression flanges. As discussed above, the other functions attributed to cross frames can be, and perhaps should be, accomplished through other means. The question can even be asked if our traditional cross-frame diaphragms are the most appropriate form to accomplish the goal of bracing the compression flange.

DETERMINATION OF GIRDER BRACING LOCATIONS

The determination of girder bracing locations is another economical consideration. As girder compression-flange brace spacing increases, the required compression-flange size increases, however the compression-flange cost does not necessarily have to increase. Discussions with fabricators suggests that the "fineness" of the current design of girder flanges to produce a least-weight flange in fact increases the cost relative to a flange comprised of fewer thicknesses, but the more commonly rolled thicknesses which the fabricator could purchase more cost effectively. Standardization of a few common flange thicknesses could move the industry to the point where increased brace spacings may have no effect on flange cost. In any case, the additional cost due to a slightly increased compression flange is worth the cost of eliminating a relatively costly brace, perhaps a traditional cross-frame diaphragm.

Required Bracing Locations

Article 6.7.4 of the *LRFD Specifications*, entitled Diaphragms and Cross Frames, states, "The need for diaphragms or cross frames shall be investigated for all stages of assumed construction procedures and the final condition." It further states, "Diaphragms or cross frames required for conditions other than the final condition may be specified to be temporary bracing." These two provisions, taken together, raise the question, when and where are permanent bracings required in a continuous steel plate-girder bridge? The answers are:

- Permanent intermediate cross-frame diaphragms <u>are not required</u> in simple-span steel girder bridges, or in the positive-moment regions of continuous steel girder bridges.

- Temporary bracing <u>is required</u> on the compression flanges of simple span steel girder bridges (in other words, the top flange), and on the compressions flanges in the positive-moment regions of continuous steel girder bridges (again, the top flange).

- Permanent intermediate cross-frame diaphragms <u>are not required</u> in the negative-moment regions of continuous steel girder bridges.

- Permanent bracing <u>is required</u> on the compression flanges in the negative-moment regions of continuous steel girder bridges (in other words, the bottom flange).

The bracing requirements for the top flange are temporary since the discrete braces are only required until the cast-in-place concrete deck has hardened. Conversely, bracing requirements for the bottom flange are always permanent requirements.

If bracings of the top flange are in the form of temporary bracing, an future rehabilitator of the bridge must recognize that the temporary bracing must be replaced prior to concrete-deck replacement.

A further question is, for the particular bridge being designed, are permanent intermediate cross-frame diaphragms the most cost-effective form of permanent bracing for the compression flange.

Procedures for Determining Bracing Locations

POSITIVE-MOMENT REGIONS

The bracing locations for positive-moment regions of composite steel I-section girders are determined through the constructability criteria of the specifications as the braces are not required in the final condition. Cost-effective girders are braced at intervals based upon the lateral-torsional buckling compression-flange bracing requirements of LRFD Article 6.10.4.2.6a for non-composite girders. The girder, which is composite in its final condition, is non-composite prior to the hardening of the concrete deck. Trying to meet the compact or non-compact section requirements will result in brace point spacings even less than the traditional values of 25 feet.

The steps in determining the positive-moment region bracing locations are:

Step 1: Select a convenient trial brace spacing,

Step 2: Determine the web-slenderness ratio, $2D_c/t_w$ for the non-composite section under the constructability load condition, i.e., all non-composite loads.

Step 3: Either,

a. If

$$\frac{2D_C}{t_w} \le \lambda_b \sqrt{\frac{E}{F_{yc}}}$$

then, use

$$M_n = 3.14EC_bR_n\left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\left(\frac{J}{I_{yc}}\right)+9.87\left(\frac{d}{L_b}\right)^2}$$

(LRFD Equation 6.10.4.2.6a-1)

to determine if the non-composite section's resistance is sufficient for the constructability condition, or

b. if

$$\frac{2D_C}{t_w} > \lambda_b \sqrt{\frac{E}{F_{yc}}}$$

and

$$L_b \le L_r = 4.44\sqrt{\frac{I_{yc}d}{S_{xc}}\frac{E}{F_{yc}}}$$

then use

$$M_n = C_b R_b R_h M_y \left[1 - 0.5\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \quad \text{(LRFD Equation 6.10.4.2.6a-2)}$$

to determine if the non-composite section's resistance is sufficient for the constructability condition, or

c. if

$$\frac{2D_C}{t_w} > \lambda_b \sqrt{\frac{E}{F_{yc}}}$$

and

$$L_b > L_r = 4.44\sqrt{\frac{I_{yc} d}{S_{xc}} \frac{E}{F_{yc}}}$$

then use

$$M_n = C_b R_b R_h \frac{M_y}{2}\left(\frac{L_r}{L_b}\right)^2 \quad \text{(LRFD Equation 6.10.4.2.6a-3)}$$

to determine if the non-composite section's resistance is sufficient for the constructability condition.

Step4: If the non-composite section's resistance is less than the required resistance, the brace spacing must be decreased. If the non-composite section's resistance is much greater than the required resistance, the brace spacing should be increased. If the non-composite section's resistance is slightly greater than the required resistance, the iterative process is complete.

The design for the constructability condition determines both the top flange area and the brace point spacing. The design as noted in the Introduction is a trade-off. Greater brace spacings will result in larger top flange areas. The sensitivity of this trade-off should be investigated for each design.

NEGATIVE-MOMENT REGIONS

Most cost-effective continuous steel girders will have negative-moment sections which, at best, will qualify as non-compact sections. Thick, overly costly webs and small brace-point spacings are required for most negative-moment sections to qualify as compact sections.

The bracing locations off the pier in the negative-moment region for non-compact sections are determined using:

- The non-compact section compression-flange bracing provisions of Article 6.10.4.1.9, or

- An adaptation of the composite section compression-flange lateral-torsional buckling flexural resistance provisions of Article 6.10.4.2.5a, for the governing Strength load combination for the girder in its final condition.

The steps in determining the negative-moment region bracing locations for the typical non-compact pier sections are:

Step 1: Select a convenient trial brace spacing,

Step 2: Determine the allowable maximum unbraced length through a simplified approach specified in the LRFD Specifications using:

$$L_b \le L_p = 1.76 r_t \sqrt{\frac{E}{F_{yc}}}$$ (LRFD Equation 6.10.4.1-1)

or a more refined approach as suggested in the Commentary using:

$$L_b \le \left(1.33 - \frac{1}{C_b}\right)\left(\frac{r_t}{0.187}\right)\sqrt{\frac{E}{F_{yc}}}$$ (adapted from LRFD Equation 6.10.4.2.5a-1)

where:

r_t = the radius of gyration of a notional section, comprised of the compression flange plus one-third of the depth of the web in compression, taken about the vertical axis (if this notional section varies between brace points due to girder cross-section changes, the minimum r_t of the girder between brace points can be conservatively used.),

C_b = the moment-gradient correction factor given in LRFD Equation 6.10.4.2.5a-4, using the compression-flange forces at the brace points due to the governing Strength load combination (the flange force can be convenient calculated as the moment at the brace point divided by the vertical distance between the centroids of the flanges.).

Step 3: Iterate until the trial brace spacing is less than or equal to the calculated maximum unbraced length.

The simplified approach conservatively assumes that the unbraced length is subject to a constant moment. In this approach, the brace spacing is calculated directly from the properties of the girder cross section alone. Unfortunately, the assumption of a constant moment results in small, costly brace spacings.

Under the more realistic moment gradient, which is used in the refined approach through the moment gradient correction factor, C_b, larger allowable unbraced lengths result. Unfortunately, in addition to the girder cross-sectional properties, the moments at the brace points must be used in the calculation to arrive at C_b.

If larger brace spacings are desired than that yielded through the above procedure, the composite section compression-flange lateral-torsional buckling provisions of Article 6.10.4.2.5a must be applied directly, similar to the procedure outlined above for the positive-moment region.

Illustrative Example

The following example has been adapted from Example 3 of the AISI/HDR/NSBA Four LRFD Design Examples of Steel Highway Bridges, U.S. units.

POSITIVE-MOMENT REGION

Step 1

Select a brace spacing of 28 feet or 336 inches.

$$L_b = 336''$$

Step 2

$$\frac{2D_c}{t_w} = \frac{2(41.91)}{0.4375} = 191.6$$

Step 3

Since,

$$\frac{2D_c}{t_w} = 191.6 > 111.7 = 4.64\sqrt{\frac{29{,}000}{50}} = \lambda_b\sqrt{\frac{E}{F_{yc}}}$$

and $\quad L_r = 4.44\sqrt{\frac{I_{yc}d}{S_{xc}}\frac{E}{F_{yc}}} = 4.44\sqrt{\frac{298.7(70.75)}{1.325}\frac{29{,}000}{50}} = 427''$

and therefore,

$$L_b = 336'' < 427'' = L_r$$

use Step 3b

$$M_n = C_b R_b R_h M_y \left[1 - 0.5 \left(\frac{L_b - L_p}{L_r - L_p} \right) \right]$$

therefore,

$$M_n = (1.14)(0.93)(1.0)(66{,}250) \left[1 - 0.5 \left(\frac{336 - 196}{427 - 196} \right) \right]$$

Step 4

$$M_r = \phi M_n = 49{,}000 > 46{,}440 = M_u$$

Therefore, the non-composite section is sufficient.

NEGATIVE-MOMENT REGION

Step 1

Select a brace spacing of 28 ½ feet or 342 inches.

$$L_b = 342''$$

Step 2

Use the refined approach to arrive at a cost-effective spacing,

$$L_b \le \left(1.33 - \frac{1}{C_b} \right) \left(\frac{r_t}{0.187} \right) \sqrt{\frac{E}{F_{yc}}} = \left(1.33 - \frac{1}{1.42} \right) \left(\frac{4.6}{0.187} \right) \sqrt{\frac{29{,}000}{50}}$$

$$L_b \le 371$$

Step 3

The brace spacing is acceptable as

$$L_b = 342'' \le 371$$

DETAILING OF INTERMEDIATE CROSS-FRAME DIAPHRAGMS

Cross-frame Diaphragm Configurations

Many different configurations of cross-frame diaphragms have been proposed, and employed, in the construction of steel plate-girder bridges. For the class of bridges discussed herein, straight non-skewed (i.e., skew angles equal to or less than about 20°) steel plate-girder bridges, cross-frame diaphragms of the "X" configuration, as opposed to the "K" configuration, are sufficient and more economical. Herein, the type referred to as the "X" configuration includes a simple "X," an "X" with a bottom strut, and an "X" with both top and bottom struts. Each "X"-configuration sub-type is adequate so long as the designer can be certain that a sufficient load path exists to carry the required loads.

Article 6.7.4 of the LRFD Specifications, entitled Diaphragms and Cross Frames, specifies that cross-frame diaphragms "shall be investigated for all stages of assumed construction procedures and the final condition." The more crucial stage for cross-frame diaphragms in the service life of the bridge is during construction.

SIMPLE "X" CONFIGURATION

The simple "X" configuration, as recommended in the AISI's Short-Span Bridge program is the most economical to fabricate. A properly designed and constructed plate-girder bridge with simple "X" cross frames may not however be the most cost-effective bridge in terms of total cost.

Let us examine the construction stages of such a bridge. During construction prior to the placement of the reinforced concrete deck, the cross frames must resist certain loads including lateral wind load and dead load of wet concrete on the bridge overhang.

A lateral wind load assumed to be equally applied to each fascia-girder flange is no problem. The individual members of the simple "X" must each be adequate to carry the horizontal wind load on each flange. Thusly, the load can be distributed to all of the girders comprising the bridge cross section.

The load on the bottom flange of the fascia girder due to a temporary cantilever forming bracket carrying the wet concrete of the overhang is not so easily accommodated. First, the *LRFD Specifications* recommend in the commentary to Article 6.10.10, entitled Constructability, that "forming brackets should be carried to the intersection of the bottom flange and the web." Herein, this recommendation is considered a must, and the assumption is made that this recommendation has been met. A free-body diagram of the simple "X" cross frame between girders loaded at the bottom flange of the fascia girder demonstrates that cross-bending of the top flange must carry a load equal to the horizontal component of the cantilever bracket reaction.

In the final condition with a reinforced-concrete deck in place over the plate girders forming a composite section, the simple "X" cross frame, sized to carry wind load during construction, is quite adequate.

Thus, a simple "X" cross frame is only adequate if the designer can be assured that cross-bending of the top flange will resist the dead load of the wet concrete on the overhang during deck casting. In the simple case of a short-span rolled-beam bridge, this is quite possible. For more complex bridges, if the girder flange is not adequate, a temporary brace may accomplish the same goal.

"X" WITH BOTTOM STRUT CONFIGURATION

The simple addition of a bottom strut between girders, transforming the simple "X" configuration into the "X" with bottom strut configuration can enhance the performance of the cross frame during construction. This bottom strut provides a more rigid path across the bridge cross section engaging all of the girders' bottom flanges.

Again, investigating the bridge during construction, reveals that equal horizontal wind loads on each fascia-girder flange can be carried across the bridge cross section most likely migrating down to the bottom strut. In the case of the cantilever bracket load, the load is carried by the more rigid load path of the bottom strut.

From a fabrication point of view, the "X" with bottom strut configuration is obviously more costly, but now the top flange of the fascia girder is not required to participate in carrying the cantilever bracket load.

"X" WITH BOTH TOP AND BOTTOM STRUT CONFIGURATION

With the addition of a top strut, the designer is assured that both the top and bottom flanges of all of the girders of the bridge cross section are engaged in carrying the lateral loads applied to the fascia girder during construction, both wind loads and cantilever bracket loads. This configuration with both top and bottom struts is only needed for deeper girders or longer diaphragm spacings.

In general, as plate girders get deeper, and as span lengths and/or intermediate cross-frame spacings get longer, the need for bottom struts, and subsequently additional top struts, grows.

Detailing Based Upon Construction Methods

Typically, individual girders are lifted into place onto the bearings with the cross frames subsequently put into place between the girders in the air. The first girder must be temporarily stayed while the second girder and cross frames are connected to it to create a stable unit. In some more isolated cases, cross-frames may be assembled between two girders on the ground with the resultant assemblage of girders then lifted into place on the bearings. This procedure is possible where cranes of sufficient capacity to lift two girders together are available, reducing by a factor of two the number of lifts necessary to erect the bridge.

Intermediate cross-frame diaphragms are erected using one of two common methods. Cross frames are erected between girders either as broken-down individual members or as a pre-assembled complete cross-frame unit.

ERECTION AS BROKEN-DOWN COMPONENTS

In the case of cross frames erected as individual members, the members are typically field bolted to tabs shop welded onto the transverse connection plates of the plate girders. The tabs are not required where sufficient length is available on the connection plates themselves to make the field bolted joint.

ERECTION AS PRE-ASSEMBLED UNITS

Much economy can be achieved by using pre-assembled cross-frame diaphragm units, as less handling and obviously field assembly is required.

When cross-frame diaphragms are erected as pre-assembled units as shown in Figure _, the members are typically shop welded to each other through a gusset plate at the intersection of the "X" and to end tabs at the end of each member which are subsequently field bolted to the transverse cross-frame connection plates on the girders.

Additional Connection Details

TRANSVERSE CONNECTION PLATE TO PLATE GIRDER CONNECTION

Transverse connection plates for cross-frame diaphragms, or transverse stiffeners used as connection plates, must be welded or bolted to both the compression and tension flanges of the plate girders. Based upon distortion-induced fatigue-cracking considerations, Article 6.6.1.3.1 of the *LRFD Specifications* state that "connection plates shall be welded or bolted to both the compression and tension flanges of the cross section where . . . connection diaphragms or cross frames are attached to transverse connection plates or transverse stiffeners functioning as connection plates. . ." This provision first appeared in the *Standard Specifications* in the mid-1980's after an epidemic of distortion-induced fatigue cracking was observed where such connection plates are not attached to the tension flange of the girder. Further, Article 6.6.1.3.1 suggests that the connection should be designed to resist an assumed lateral force of 20 kips, for the class of bridges being discussed herein.

The problem of distortion-induced cracking of the unstiffened web gaps at diaphragm connection plates on welded plate girders is often cited as a reason to eliminate regularly spaced cross-frame diaphragms from the designs of new plate-girder bridges. This is not a legitimate reason. The problem of distortion-induced fatigue cracking at diaphragm connection plates in new construction was solved in the 1980's with the introduction of the provision discussed above. A diaphragm connection plate detailed and designed in accordance with either the current *Standard Specifications* or the *LRFD Specifications* will not crack due to the secondary stresses typically termed distortion-induced stresses. Unfortunately, our patrimony of existing welded steel bridges designed prior to the mid-1980's is quite another story.

DIAPHRAGM MEMBER TO CONNECTION PLATE CONNECTION

The connection of the cross-frame members with the plate girder need not be considered as a truss joint, with the lines of action of the cross-frame members coincident with the junction of the flange and the web. The relatively stiff transverse connection plate removes this joint from the realm of a truss joint. The connection geometry should be designed for economic efficiency not to maintain this false truss analogy.

PROPORTIONING OF INTERMEDIATE CROSS-FRAME DIAPHRAGM MEMBERS

The *LRFD Specifications* state in Article 6.7.4 entitled Diaphragms and Cross Frames that "as a minimum, diaphragms and cross frames shall be designed to transfer wind loads . . . and shall meet all applicable slenderness requirements. . ." Of course, all of the provisions of specifications are minimum requirements, but the specific wording of this provision reflects the paradox of observed cross-frame diaphragm behavior versus calculated behavior confronting the codewriters.

Cross-frame diaphragms for all steel plate-girder bridges save curved-girder bridges have traditionally been proportioned based upon lateral wind load analysis and slenderness requirements only, ignoring live load effects. Primarily based upon the difficulty, if not impossibility, associated with accurately estimating cross-frame forces from the simple live-load distribution-factor approach to girder design, in which longitudinal behavior is uncoupled from transverse behavior, live-load forces are ignored when proportioning cross-frame diaphragm members. In addition, field measurements of strains in cross-frame members of in-service bridges reveal relatively moderate stress under random and design live load.

With the advent of bridge design using refined analysis, calculated cross-frame diaphragm member forces have become available to more sophisticated bridge designers. Yet, the magnitude of these calculated member forces is troubling. These calculated forces suggest that our traditionally proportioned cross-frame members are woefully inadequate, yet no distress is observed in cross frames of in-service bridges. Proponents of refined analysis for cross-frame member design rebut the paradox suggesting that our bridges never experience the loads for which they are designed.

From the point of view of uncertainty, the greater uncertainty does not lie with the determination of the HL-93, the notional live-load model defined in Article 3.6.1.3, but with the various analysis methods available to the designer. The factored HL-93 live-load model can be expected to be exceeded a handful of times during the 75-year design life of the bridge. This factored live load of the *LRFD Specifications* is not, in practice, much greater than the factored HS-20 live load of the *Standard Specifications*, thus past observations are applicable to bridges designed to the *LRFD Specifications*.

As a beginning point, the designer should proportion the cross-frame diaphragm members applying the slenderness-related provisions of Articles 6.9.3 and 4.6.2.5 of the *LRFD Specifications*. Thus, the initial member sizes are based upon maintaining a $Kl/r = 140$, assuming all members can under some loading be in compression, with $K = 0.75$ for welded or bolted end connections. Next, the designer must check these sizes to be assured that a sufficient load path exists to distribute the specified wind loads to the bearings.

CROSS-FRAME DIAPHRAGMS AS ERECTION AIDS

Erectors use cross-frame diaphragms as aids during erection. Cross-Frame Diaphragms serve four functions during erection of the steel superstructure:

1. bracing the compression flange (top flanges in positive-moment regions, bottom flanges in negative-moment regions),

2. providing a horizontal spacer between girders,

3. maintaining the cross-slope or vertical alignment of the girders, and

4. maintaining plumbness of the girder webs.

The first provides stability of the girder under its own self-weight alone, prior to the placement of the composite concrete deck; the latter two functions provide simple geometry control for the erector.

With traditional cross-frame spacings of 25 feet, erectors have installed every second or third diaphragm during erection to serve these functions. Upon erection of the entire steel superstructure, the erectors go back and fill in the remaining cross frames, prior to the placement of the composite concrete deck. The addition of the remaining cross frames provides the required bracing of the top flange under the additional self-weight of the wet concrete deck.

SUMMARY

Critics of the LRFD cross-frame diaphragm provisions view them as a unilateral attempt by the code writers to eliminate cross frames. Proponents view the provisions as a mandate to eliminate cross frames. The intent of the code writers was not so black or white. As with the rest of the provisions of the *LRFD Specifications*, the intent is to free bridge designers to be more innovative and to produce more cost-effective, and better performing bridges.

Elimination of the traditional 25-foot maximum spacing of cross-frame diaphragms has two effects: (1) innovative bridge designers are free to explore options to, and increased spacings of, cross-frame diaphragms, and (2) more conservative designers who wish to use more traditional regularly spaced cross-frame diaphragms are free to use regular spacings marginally greater than 25 feet if practical. For example, in the past, strict adherence to the *Standard Specifications* would preclude a logical arrangement of 3 diaphragms spaced at 25'-6" for a 102-foot span, and force the designer to use 4 cross-frame diaphragms instead.

The Guide has concentrated on the first effect discussed above, that of exploring more cost-effective steel bridges through increased spacing of permanent intermediate cross-frame diaphragms and through the consideration of temporary bracing to eliminate cross frames required only during construction. The designer should not lose sight of the fact that, if deemed more cost-effective, steel bridges designed as with the *Standard Specifications* with regularly spaced cross-frame diaphragms are acceptable. Less sophisticated designers, who are designing steel bridges not in cost competition with concrete bridges, may continue to use the traditional arbitrary 25-foot spacing and rest assured that their bridges are safe and relatively cost effective. The more sophisticated designer is merely being implicitly asked by the codewriters to consider increased cross-frame spacings and temporary bracings during construction to reduce the costs of steel plate-girder bridges.

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