USING NSBA'S "LRFD SIMON" SOFTWARE FOR PRELIMINARY DESIGN OF A CURVED HAUNCHED STEEL PLATE GIRDER BRIDGE

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BIOGRAPHY

Thomas Densford, P.E. is a Sr. Principal Engineer with the firm Fay, Spofford & Thorndike based in Burlington, MA, and providing engineering in the New England region. Tom's highway bridge design experience includes structural steel, reinforced and post-tensioned concrete bridges, as well as project management of the design production.

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SUMMARY

This paper discusses the use of NSBA's "LRFD SIMON" software in the preliminary design of a two span, horizontally curved, parabolically haunched steel girder bridge located in Maine. While SIMON is a straight girder analysis program, the input data and results may be modified for preliminary curved girder design. Using SIMON's XML formatted output, the straight line-girder output was transferred to a spreadsheet for post-processing to include the effects of construction loads and curved girder behavior. The spreadsheet also included a graph (fish diagram) of required flange areas versus provided flange areas, which gives a simple method to quickly optimize plate sizes and splice locations. Final design using a 3-D finite element program verified the preliminary design.

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Purpose

This paper discusses the use of NSBA's "LRFD SIMON" (1) software for the preliminary design of a two-span, curved, parabolically-haunched steel girder bridge located in Turner-Greene, Maine. The preliminary girder design, using SIMON as an analysis tool, was very effective and contributed to being selected for this MaineDOT Design-Build project. Although SIMON is a straight girder analysis program, the input data and results were modified for preliminary curved girder design. Final girder design using two 3-D finite element programs verified the preliminary design. The intended audience of this paper is practicing bridge engineers. Accordingly, bridge design code references have been provided, and pertain to the AASHTO LRFD Bridge Design Specifications, 2012 (2), and will be shown within this paper as "LRFD-", with the relevant section number following.

SIMON Features and Limitations

SIMON is a line-girder analysis program for a single, one-dimensional, continuous composite steel plate girder. The program input consists of basic girder geometry, slab and rebar data, non-composite and composite dead loads, and live load distribution factors for one and two lanes.

From the input data, SIMON computes the girder self-weight, as well as the non-composite and composite dead load moments, shears, and deflections at the girder tenth points. The program computes composite and non-composite girder section properties along the girder, including splice points, and allows variable web depths.

Using input live load distribution factors, the program computes the live load moment and shear envelopes for service, strength and fatigue cases, as well as live load deflections.

SIMON makes numerous detailed section checks, as required by LRFD-6.10, for proportions, service, fatigue, and strength, including local and lateral torsional buckling. The program reports the ratio of factored load effects to factored resistances.

SIMON output is provided in XML format, which is useful for post-processing of the results. The XML file includes all input data, and output data including moments, shears and displacements, as well as stiffener and shear connector requirements. Tables of girder components, weights and costs are also provided.

The simple input and efficient output makes girder design iterations very quick and easy when compared to performing calculations by hand, or by more computationally intensive finite element girder design software. Iterations can include variations in the number of girders in a given cross-section, and girder plate arrangements in support of optimizing the design.

The live load distribution factors are limited to a single set, rather than providing for varying girder properties and the resulting change in the distribution factors along the girder. SIMON also does not check lateral flange forces on the exterior girder due to overhang bracket construction loads (LRFD-6.10.3.4) or wind load (LRFD-4.6.7.2), which need to be checked by the engineer.

Benchmarking SIMON

Prior to using SIMON, the program output was checked against AISC influence lines (3) and WSDOT's QCONBRIDGE software (4) to confirm SIMON's continuous beam analysis. SIMON's detailed output of girder properties and resistances were compared to hand calculations with good agreement.

Turner-Greene Bridge Data

The basic bridge layout is shown in Figure 1 and Figure 2.

The bridge consists of two 240 ft. spans (480 ft. overall length) over the Androscoggin River on the town line of Turner and Greene, Maine.

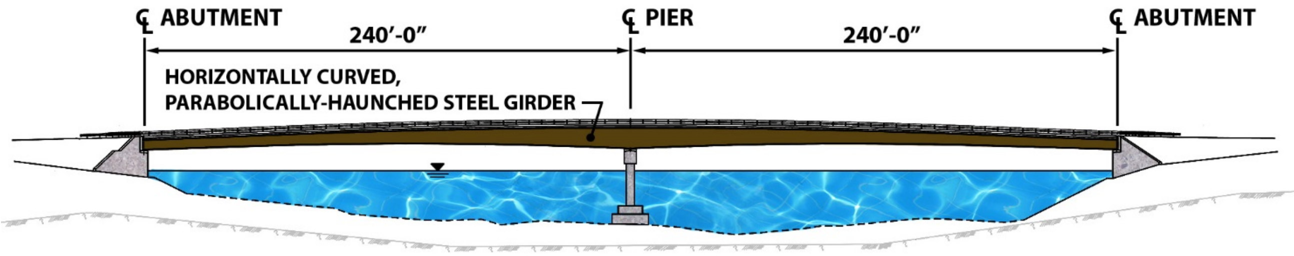

Figure 1. Bridge Elevation

The bridge roadway alignment is on a 1,240 ft. radius horizontal curve, and has a deck width of 35.33 ft.

Five lines of Grade 50 steel plate girders were selected so that the girder erection pick weights would not require a heavy crane in the water, which would have been required had a four girder system been used.

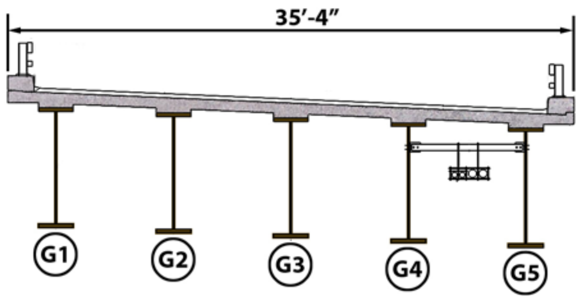

Figure 2. Typical Bridge Section

The overall haunched depth of the superstructure varies from 6.0 ft. to 11.5 ft. The web depth of the haunched girders varies from 4.5 ft. to 9.83 ft.

Project geometric constraints included tying bridge ends into the existing grade immediately outside the bridge footprint, thus minimizing right-of-way takes and disturbance to the aquatic resources, providing a minimum low chord elevation for flood passage, and providing a 1% minimum tangent grade on the roadway profile.

Conventional wisdom says that haunched girders are not economical for spans less than about 300 ft. However, a haunched girder can provide a much shallower depth at the abutments (say L/40), if a deeper section is provided at the pier (say L/20). Thus a haunched girder was a good solution to provide shallow girder ends, and the crest curve accommodates the extra pier depth required, while maintaining a minimum low chord elevation.

Developing the SIMON Input Data

As noted above, SIMON is a single line girder analysis tool. This method is considered an "approximate" method in contrast to a "refined" analysis, as defined in LRFD-4.6. That is, the program does not calculate a "refined" distribution of loads to an interconnected system of multiple girders, nor does it consider the effects of girder curvature.

Major axis bending moments due to curvature may be ignored for girder curvature which does not exceed 0.06 radians per LRFD-4.6.1.2.4b, but lateral flange bending effects must be considered. The subject bridge has a curvature angle of about 0.16 radians, thus both major axis bending and lateral flange bending effects due to curvature must be computed. LRFD-4.6.2.2.1 notes that the V-load method for determining forces due to girder curvature is a good starting point. LRFD-4.6.3.3.2 recommends refined analysis methods for curved girder bridges. For final design, refined analyses using two independent commercial finite element bridge design programs were used, and confirmed the preliminary design with minor modification. However, the more complex analyses required much more time and expertise to achieve a viable solution. SIMON provided a quick, verifiable analysis engine for the preliminary design.

SIMON load data input consists of data needed for the analysis of straight girders, adjusted for girder curvature effects. Load data consists of non-composite dead loads (framing, deck), composite dead loads (curb, rails, wearing course), and live load distribution factors per LRFD-4.6.2.2.

Using the V-load method to determine additional loads due to girder curvature is relatively simple, and is described in several references (5)(6). The V-load method relies on simple relationships of geometry and loading. As with a straight girder, the girder flange axial forces are roughly equal to the moment divided by the girder depth. However, due to flange curvature, the flange axial forces are not collinear, and forces perpendicular to the flange are required for equilibrium.

From V-load analysis, the curvature results in an approximate vertical force, V_i, applied at each crossframe/web intersection in the plane of the web. The exterior girder V-loads are downward at the girder on the outside of the curve, and upward on the girder at the inside of the curve. Per (6), at a given crossframe, the exterior V-load is:

$$V_i = \frac{\sum M_{primary}}{CK}$$

in which:

$$C = \frac{N_g(N_g + 1)}{6(N_g - 1)}$$

$$K = \frac{RD}{d}$$

where:

$N_g =$ number of girders
$D =$ transverse girder spacing
$d =$ longitudinal crossframe spacing along the arc length of the girder

For a line girder analysis with approximately the same girder lengths and primary (major axis) moments for all girders, $\sum M_{primary} = N_g \times M_{primary}$ of the girder being analyzed. V_i loads for the interior girders can be taken as the ratio of the interior girder offset to the exterior girder offset. For the subject bridge, with 5 girders, girder G1 has the largest V-load, girder G2 has half that V-load, and girder G3 has no V-load because it is at the center of the crossframe rigid body rotation. Girders G4 and G5 are on the inside of the curve, and theoretically have V loads which are opposite to the gravity loads.

For girder G1, a unit distributed load was applied to the straight line girder, and the resulting primary moments were used to determine the various V-loads at the crossframe locations. These V-loads were applied to the same line girder model in a separate load case. The resulting moments from the applied V-loads were seen to be about 22% to 35% of the primary unit load moments, with an average of about 25%. This 25% moment increase was used to increase the computed non-composite and composite dead loads in the SIMON input, and the resulting SIMON output then included the major axis effects of curvature. The 25% increase is consistent with the design aid provided in the CUGAR approach in (5). Girder G2 dead loads were increased by half of the 25%, due to the lesser eccentricity of the G2 line. Girder G1 live load distribution factors were increased by 20% for girder G1, and 10% for girder G2 to account for curvature effects. Typical increases in dead (Figure 5) and live (Figure 6) loads were provided in (5) and may be used in lieu of V-loads.

Post-Processing SIMON Results

To complete the analysis, a few additions must be made to the SIMON code checks at the sections along the girders. These additions include lateral flange bending due to curvature for all girders, and, for the exterior girders, the overhang bracket lateral flange forces generated during construction (LRFD-6.10.1, LRFD-6.10.3.4), as well as lateral flange forces for the wind load case (LRFD 4.6.2.7.1). The resulting lateral flange stresses are added to the major axis results.

Flange curvature causes a varying lateral flange distributed load, normal to the flange longitudinal direction. The distributed load is equal to the flange axial force divided by the flange radius of curvature. Each flange spans laterally between crossframe locations as a continuous beam, as noted in LRFD-4.6.1.2.4b:

$$M_{lat} = \frac{M\ell^2}{NRD} \qquad \text{(C4.6.1.2.4b-1)}$$

where:

$M_{lat} =$ flange lateral bending moment (kip-ft)
$M =$ major axis bending moment (kip-ft)
$\ell =$ unbraced length (ft)
$R =$ girder radius (ft)
$D =$ web depth (ft)
$N =$ a constant taken as 10 or 12 in past practice

The derivation of the equation above is as follows. The flange longitudinal axial force due to primary major axis bending is $F = M/D$, where M is the major axis bending and D is the depth of the girder. The flange lateral distributed load due to curvature, is the axial force divided by the girder radius: $w = F/R = \left(\frac{M}{D}\right)/R$. The lateral flange moment due to curvature, in the flange at the crossframe support, is therefore given by:

$$M_{lat} = \frac{w\ell^2}{N} = \frac{\frac{\left(\frac{M}{D}\right)}{R} \times \ell^2}{N} = \frac{M\ell^2}{NRD}$$

The lateral flange stress is $f_\ell = \frac{M_{lat}}{S_{flange}}$ where S_{flange} is given by $\left(t_f \times b_f^2\right)/6$

where t_f is the thickness of the flange, and b_f is the flange width.

LRFD-6.10.3.4 discusses lateral flange forces due to eccentric overhang loads from wet concrete, formwork, and screed rails. The weights of these elements, multiplied by their horizontal eccentricities to the web, become torsional forces which are resisted by a couple on the eccentric flanges. That is, lateral flange force F_ℓ equals element weight times eccentricity divided by the web depth. Figure 3 (7) shows a typical overhang form detail.

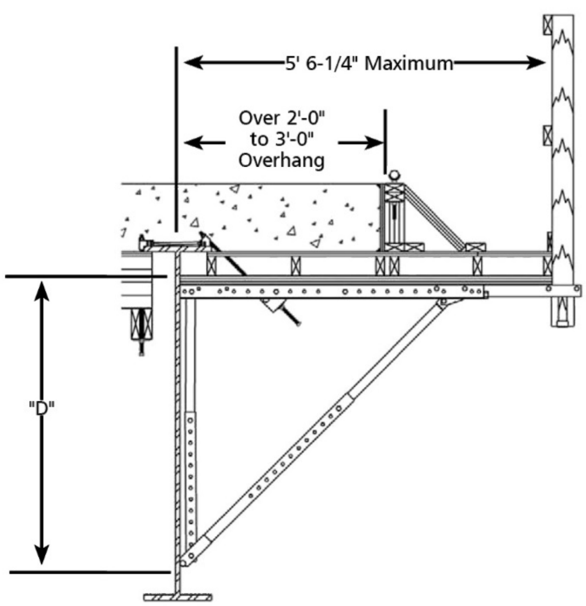

Figure 3. Typical Overhang Form Bracket

Lateral flange stress caused by the form bracket loads is additive to the exterior girder G1 stresses, and may be computed per LRFD-6.10.3.4:

$$M_\ell = \frac{F_\ell L_b^2}{12} \qquad \text{(C6.10.3.4-2)}$$

where:

$M_\ell =$ lateral bending moment in the flanges due to the eccentric loadings from the forming brackets (kip-in.)

$F_\ell =$ statically equivalent uniformly distributed lateral force from the brackets due to the factored loads (kip-in.)

$L_b =$ unbraced length (in.)

Similarly, horizontal wind pressure on the outer webs are resisted by lateral flange bending, as noted in LRFD-4.6.2.7.1:

$$M_w = \frac{WL_b^2}{10} \qquad \text{(C4.6.2.7.1-2)}$$

where:

$M_w =$ maximum lateral moment in the flange due to the factored wind loading (kip-ft)

$W =$ factored wind force per unit length applied to the flange (kip/ft)

$L_b =$ spacing of the brace points (ft)

The lateral flange stresses due to these lateral flange forces increase the demand on the flanges per the formulas in LRFD-6.10.7 and -6.10.8. This has the mathematical effect of decreasing the flange resistances available for primary major axis bending effects. Thus the code checks performed by SIMON are approximately applicable when the steel yield stress input into SIMON is reduced to account for lateral flange stress demands.

Using the XML format of the SIMON output, a post-processing spreadsheet was used to check the girder sections against the code requirements in LRFD-6.10, and included the addition of the flange forces discussed above. SIMON reports forces at girder tenth points, so interpolation was used to generate forces between tenth points for plate size changes and splices.

The web plate size is determined very quickly using SIMON. The flanges, however, have a wide range of parameters which may be varied to achieve an optimal steel weight.

Flange Plate Optimization

Girder flange plates were optimized by using the following criteria:

1. In each girder field section, provide constant width flanges. Step flange thicknesses to follow the stress demand.
2. Keep the plate thicknesses similar between girders, so that wide slabs of plate may be ordered and cut to width for multiple flanges.
3. Do not introduce a plate butt splice to reduce flange area unless about 1,000 pounds of steel may be saved.
4. Area fabricators indicate that 80 ft. to 85 ft. is maximum plate length for thicker flange plate. Therefore, consider using this length to determine plate splice locations, since a splice is inevitable once the maximum plate length is exceeded.
5. Plate availability industry-wide is further discussed in Christopher Garrell's article in *Modern Steel Construction* (8).

A simple and powerful graphical comparison of flange "area required" versus "area provided" was prepared in the post-processing spreadsheet, using the flange load effect demands and resistances for the top and bottom flanges for the strength load combination. Figure 4 shows the plot for Girder G1. In the plot, the "provided" flange plate cross sectional areas along the length of the half-girder are plotted. Due to the symmetry of the structure, the half-girder was sufficient to determine plate sizes for the entire structure.

The flange areas "required" are also plotted, as approximated by the simple formula of:

$$A_{\text{required}} = A_{\text{provided}} \times \frac{(\text{Factored Load Effect})}{(\text{Factored Resistance})}$$

For the compression flange, whether the top flange near midspan or bottom flange over the pier, the factored resistance is the lateral torsional buckling stress resistance. For the tension flange, the yield stress generally governs the factored resistance.

The "area required" curve has some discontinuities at the flange changes, because the relationship to "area provided" is approximate, and diverges as the load effect departs from the resistance. However, the relationship is conservative.

One notable property of the diagram, is that the area between the "area provided" curve and the "area required" curve is the steel waste. That is, shortening a large flange in length, results in an increase in the adjoining smaller flange area required, and vice versa. Thus, by applying the optimization criteria for plate length and equivalent splice weights, an efficient flange layout may be developed very quickly by minimizing the area between the curves. The "area required" curves make the shape of a fish for a two-span, symmetric structure, and the plot has taken on the name of a "fish diagram". Iterations and convergence of the design for least weight, fabrication, constructability, while meeting code requirements, can be performed very quickly. In fact, the original design tried four girders, and found that the pier section exceeded the crane capacity on hand, and was changed to five girders.

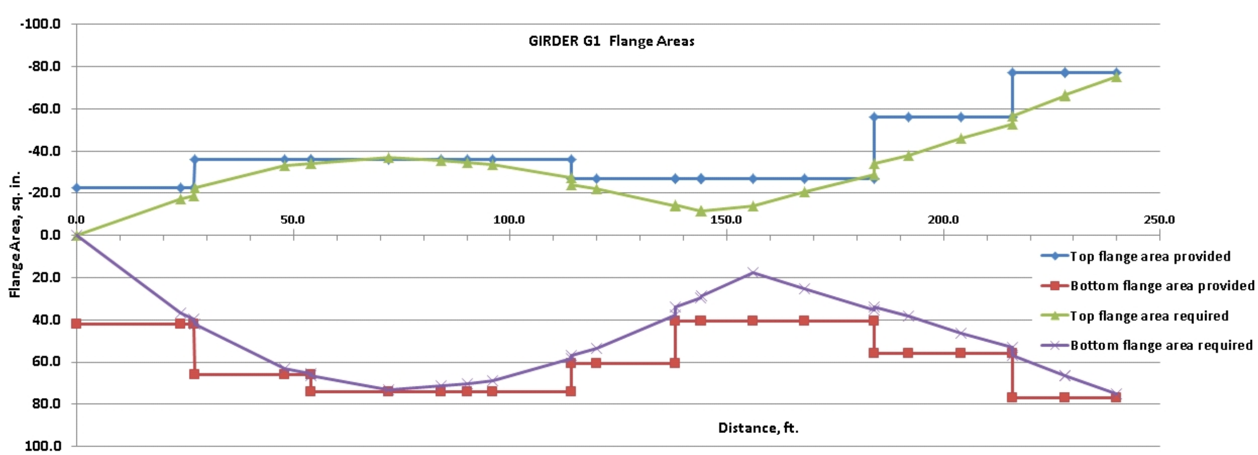

Figure 4. Girder G1, Required and Provided Flange Areas along the Half-Girder

Final Design vs. Preliminary Design

The efficient steel plate girder design contributed to our contractor, Reed & Reed Construction, Woolwich, Maine, preparing the winning bid. LRFD "Refined Analysis" was performed during final design, along with more plate optimizing via conversations with the fabricator, Bryon Tait of Casco Bay Steel Structures, South Portland, Maine. Two commercial finite element steel girder programs were used to perform three-dimensional refined analyses, modeling the curved girders and crossframes, properties of the non-composite and composite sections for the staged construction, and iterating girder plate sizes. The final design was reasonably consistent with the preliminary design using SIMON, with no increase in final fabricated steel cost. Some larger plate lengths were increased to avoid shop splices. SIMON proved to be a simple and powerful design tool, and we look forward to enhanced features in future releases.

References

(1) NSBA Steel Bridge Design Suite. LRFD SIMON (Version 10.1.1.6), 2012.
(2) *AASHTO LRFD Bridge Design Specifications*, 2012.
(3) *AISC Moments, Shears, and Reactions for Continuous Highway Bridges*, 1986.
(4) WSDOT. QCONBRIDGE Software, 2005.
(5) FHWA/Richardson, Gordon & Assoc. *Curved Girder Workshop (CUGAR)*, 1978.
(6) Xanthakos, Petros P. *Theory and Design of Bridges*, 1994: Chapter 6.
(7) Dayton Superior. *Bridge Deck Handbook*, 2012.
(8) Garrell, Christopher. "Steel Plate Availability for Highway Bridges." *Modern Steel Construction*, September 2011.

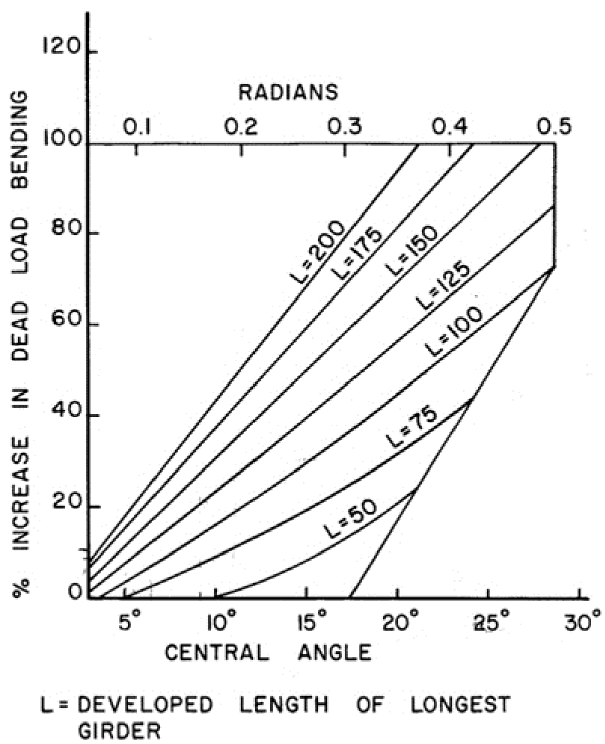

Figure 5. Percent Increase in Dead Load Bending Moment in Longest Curved Girder (5)

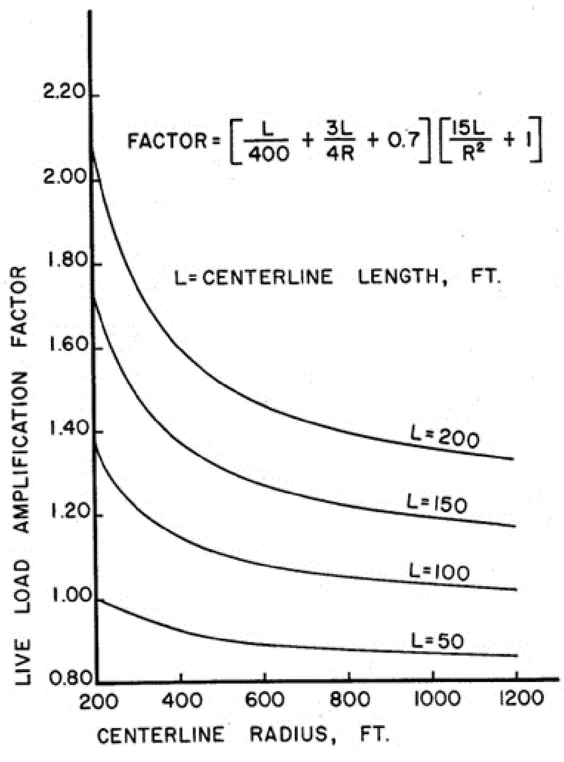

Figure 6. Amplification Factor for Live Load Bending Moment in Longest Curved Girder (5)