LRFD IMPACTS ON THE DESIGN OF THE HATHAWAY BRIDGE

Christopher J. Mills, PE, HNTB Corporation, Orlando, Florida
Alexander M. Collins, PE, HNTB Corporation, Orlando, Florida

ABSTRACT

The new Hathaway Bridge, in Panama City, Florida, is a Design/Build project that consists of two, precast segmental box girders with seven spans of 330-ft and 200-ft end spans for each box girder. The AASHTO LRFD Code, 1998 with Interims through 2000 was used for the design. This paper compares the LRFD Design to the AASHTO 16th Edition Code requirements as well as the requirements of the Guide Specifications for Design and Construction of Segmental Concrete Bridges, 2nd Edition.

Keywords: Design/Build, Box Girder, Precast, Segmental, HNTB, and LRFD.

INTRODUCTION

The Hathaway Bridge project is a bridge replacement in Panama City, Florida, which was let by the Florida Deportment of Transportation (FDOT) using the Design/Build format. The bridges link Panama City and Panama City Beach, Florida across St. Andrew Bay.
The project consists of two precast concrete segmental box girders that share common foundations at each pier location. The top slab of each box is 80-ft wide, and the typical spans are 330-ft long. There are seven such spans for each box, with approach spans of approximately 200-ft.
The design of the bridge was accomplished using the AASHTO LRFD Code, 1998, with Interims through 2000. Several aspects of the design, including substructure design, superstructure transverse bending, superstructure longitudinal bending, and longitudinal shear are described here. The impact of the LRFD Code on the design with respect to the 16^{th} Edition of AASHTO and the Guide Specifications for Design and Construction of Segmental Concrete Bridges, 2^{nd} Edition, is discussed.

SUBSTRUCTURE DESIGN

The substructure for the new bridge consists of single columns beneath each box girder, supported on a combined waterline pile cap. Figure 1, below, shows one of the piers closest to land during construction of the cantilever for the Westbound Bridge.

Figure 1 – Combined Waterline Pile Cap

COLUMNS

The columns range from 14′-7″ near shore to 62′-5″ tall at the channel piers. Design of the columns was fairly straightforward, with Strength-III (wind at γ=1.40) controlling the taller columns. Strength-I, with its higher eccentric live load moment controlled the design of the shorter columns. This is as would be expected from AASHTO 16^{th} Edition Design. The reinforcing amounts were not noticeably different from what would have been expected under previous codes

FOUNDATIONS

The 60″ diameter concrete cylinder piles represent the most interesting substructure design feature. The pile caps were designed using the maximum pile loads, so that the load case that controlled the piling loads also controlled the pile cap design. Each foundation was analyzed for both vessel collision and LRFD Strength/Service load combinations separately. Vessel Collision loading did not control the design, since the pile caps were joined and the loads could be shared by both superstructures.
Most of the piles in each of the piers in water were battered, so that the moments in the piles, though higher in some load cases, were inconsequential in the design. The control for all piers was the axial capacity of the soil, as determined by the Geotechnical Engineer.

Table 1 shows a summary of the controlling load cases for each of the foundations. It can be seen that for the shortest piers, Strength-I controls, while Strength-IV controls for the intermediate piers, and Strength-III and Strength-V control for the tallest and fixed, piers respectively.

Table 1. Summary of Controlling Load Cases for Foundation Design

WB Pier No.	Column Height	100-Yr. Scour Elev. (ft)	LRFD Load Case
2	14’-7”	0	Strength-I
3	17’-2”	-29	Strength-I
4	15’-5”	-43	Strength-IV
5	27’-9”	-49	Strength-IV
6	40’-11”	-55	Strength-IV
7	54’-1”	-65	Strength-V
8	62’-5”	-74	Strength-V
9	62’-5”	-78	Strength-III
10	54’-1”	-69	Strength-III
11	40’-11”	-57	Strength-IV
12	38’-5”	-46	Strength-IV
13	30’-5”	-34	Strength-IV
14	23’-4”	0	Strength-I

The Strength-IV load case controlled the design of many of the foundations. While this load case has no equivalent load combination in previous codes, the maximum pile loads in the Strength-III, Strength-IV, and Strength-V combinations were all very close to each other for this structure. However, one could easily imagine a concrete structure with longer spans that would be penalized by the use of this Strength-IV combination.

It should be noted that HNTB developed an additional service load combination that included wind at γ=1.00, to calculate the displacement of the structure under full wind loads, since Service-I includes only γ=0.30 for wind. This load case was used to calculate the tip elevations of the piles.

SUPERSTRUCTURE DESIGN

The three primary design concerns for the superstructure are transverse bending, longitudinal bending, and longitudinal shear. Each of these is discussed in the following sections.

TRANSVERSE BENDING

Segmental box girders typically use transverse post-tensioning in the top slab to produce longer cantilever overhangs and greater distances between webs than conventional girder bridges. In LRFD Design, the Service-III load group is used to calculate the spacing of the transverse tendons which will result in tensile stresses which are below an allowable stress dictated by the Owner. Then, the various sections are examined for flexural strength, as well. The process is similar under the 16th Edition of the AASHTO Code – Service load stresses are used to calculate the amount of post-tensioning required and ultimate strength is checked at each critical section

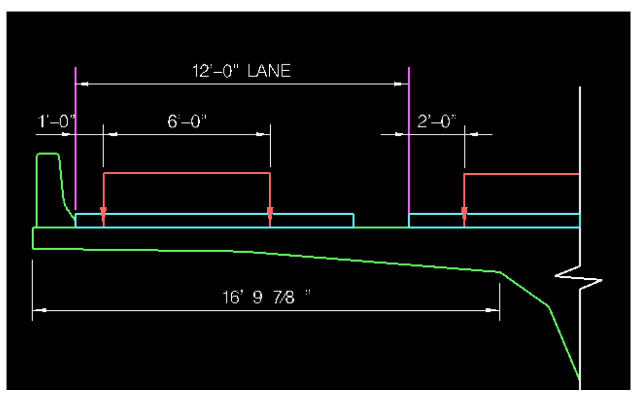

Figure 2 – Cantilever LRFD Loading; Partial Cross-Section of wing.

For The new Hathaway Bridges, the stress at the root of the cantilever wings controlled the required spacing of the transverse post-tensioning. Figure 2 shows the LRFD loading associated with the design of the element.

We compared the required spacing of cantilever tendons under 3 common types of loading for the cantilever.

- LRFD HL93 Loading, Service-III load group
- AASHTO HS20-44 Loading, Service-I load group
- (AASHTO HS20-44 Loading x 1.25) to give "HS25", Service-I load group

Table 2. Spacing of Transverse PT Tendons for Varying Live-Loads

Case	DL+PT Stress (psi)	LL+I Stress (psi)	Total Stress (psi)	Tendon Spacing (ft)
LRFD HL93	366	-596	-230	2.25
HS20-44	376	-605	-229	2.23
HS25	495	-726	-231	1.95

Some agencies have required HS-25 loading for bridges recently, thus it was included in the study.
Table 2, below, shows the results. The spacing of 4-strand, 0.6″diameter transverse tendons is virtually the same for both the LRFD Service-III load group with HL93 loading and the AASHTO HS20-44 loading. However, the "HS25" loading would result in approximately 15% more transverse post-tensioning for this bridge.

LONGITUDINAL BENDING

Similar to transverse bending, the quantities of post-tensioning required to resist longitudinal flexure are typically calculated for service load groups. Then, the ultimate capacity of the bridge is checked using strength groups. Below are two figures from Podolny and Muller[1] with the comparative quantities for the Hathaway Bridge indicated with respect to bands of typical quantities for segmental bridges of many different span lengths. Figures 3 and 4 show that the concrete and longitudinal post-tensioning quantities, respectively, for the bridge are acceptable - though somewhat lower than most structures.

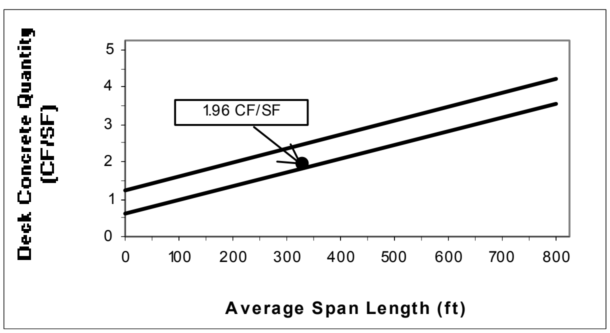

Figure 3 – Average quantities of deck concrete.

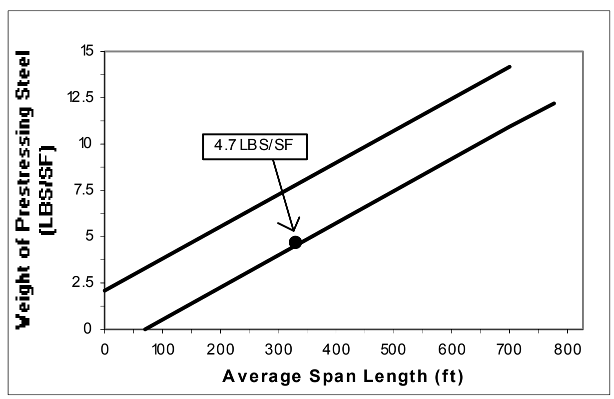

Figure 4 – Average quantities of longitudinal post-tensioning steel.

LONGITUDINAL SHEAR

One of the more interesting studies when assessing the effects of LRFD Design on segmental box girder bridges, is the longitudinal shear design.

During the course of the load rating, HNTB compared the Shear Designs under the 1) LRFD 2000 Interim, 2) AASHTO 16th Edition Chapter 9, and 3) the Guide Specifications for Design and Construction of Segmental Concrete Bridges – 2nd Edition. For brevity, these three codes will be referred to as "LRFD", "Chapter 9", and the "Guide Specs", respectively, in the text below. Also, the load rating process highlighted the critical location in the bridge for shear design. All discussions are based on the calculations at that location.

Shear Design Methods – All three methods use the following equation as their basis:

$V_u \leq \varphi V_n = \varphi(V_c + V_s)$
Or,
Load ≤ Resistance.
The differences on the load, or V_u, side of the equation are well known, with LRFD using different load factors, impact, multiple presence factors, and combinations than Chapter 9 and the Guide Specs.
At the critical node, the applied shear was calculated as follows:

Chapter 9 and Guide Specs:
V_u = 3975 kips (Strength-I)
LRFD:
V_u = 3955 kips (Strength-I)
The two loads are virtually the same at this node. Also, the φ factors are not the object of this discussion. Instead, we will concentrate on the resistance side of the equation, expressing the various resistances in terms of $\sqrt{f'c}$.
In general, the three design methods place the following limits on the calculated shear capacity of a section:

Chapter 9:
$V_c = V_{cw}$ or V_{ci} , is not explicitly limited, with V_{cw} typically controlling near supports. V_{cw} increases with increasing axial force on the section.

$V_s \leq 8\sqrt{f'c}$ * bd, and is the same as the equation for reinforced concrete, which implies a crack angle of 45°.

V_n = unlimited by virtue of V_c being unlimited.

Guide Specs:
$V_c = 2K\sqrt{f'c}$ * bd, with K normally at its maximum of 2.0 = $4\sqrt{f'c}$ * bd

V_s = unlimited, due to the fact that the code implies that the designer can calculate different angles than 45°.

However,
V_n max = $12\sqrt{f'c}$ * bd

LRFD:
In LRFD, moment and shear calculations are interrelated. The calculation is iterative, with the designer selecting a trial crack angle, θ, then comparing results to his estimate and refining. Also required is a check that the total longitudinal reinforcing resisting tension is sufficient for both moment and shear tension demands [LRFD 5.8.3.5].

V_c = from tabulated values, maximum = $6.3\sqrt{f'c}$ * bd (in psi - to be consistent with above). This is with β= 6.32.

V_s = unlimited

V_n = 0.25 (f'c)bd [LRFD 5.8.3.3-2] (for web strut crushing)
Which, for f'c=6000psi concrete = $19.4\sqrt{f'c}$ (in psi)

Investigation at the Critical Node – Below, the critical node for shear in the structure is investigated to show the differences among the three methods of shear capacity calculation.

Chapter 9:

d=9.5 ft, b=2.67 ft

V_c=V_{cw} = 1891 k ≅ $6.7\sqrt{f'c}$ *bd
V_s = 2112 k ≅ $7.5\sqrt{f'c}$ *bd (based on reinforcing supplied and d above)
V_n = 4003 k ≅ **14.2** $\sqrt{f'c}$ *bd

Guide Specs:

d=9.5 ft, b=2.67 ft

V_c = 1129 k =$4\sqrt{f'c}$ *bd
V_s = 2112 k ≅ $7.5\sqrt{f'c}$ *bd (based on reinforcing supplied and d above)
V_n = 3241 k ≅ **11.5** $\sqrt{f'c}$ *bd

LRFD:
The following is a summary of the calculations for the critical node, at the actual level of longitudinal compression and reinforcing:

d=8.55 ft, b=2.67 ft, θ=27°, β=2.43

V_c = 618 k ≅ $2.4\sqrt{f'c}$ *bd

V_s = 4142 k $\cong 16.3\sqrt{f'c}$ *bd

V_n = 4760 k $\cong \mathbf{18.7}\sqrt{f'c}$ *bd

In summary, the calculations for this node in the bridge show that the capacity of the section is greater under the LRFD Code than under the other two methods. Thus, the webs need not be as massive as in previous codes to carry the same amount of load.

PRINCIPAL TENSION

Another interesting study was performed using the Service-III load group at the critical node. The well known Mohr's circle calculation was applied to determine the principal tension under service loading. Under the applied axial force and shear, the principal tension was determined to be $7.1\sqrt{f'c}$, with a corresponding angle of 33.3°. It is interesting to note that in this case, with V_n approaching its maximum value, the calculated principal tension approaches the cracking limit for concrete. However, this principal tension value is well in excess of the normally allowed values for service principal tension. In order to counter this result, we added vertical post-tensioning to the webs of the box girders in several regions. With the addition of the vertical post-tensioning, the calculated principal tension reduced to $5\sqrt{f'c}$, and the calculated angle increased to 37°.

There is no provision in the LRFD Code, or in the other two US codes that allows vertical post-tensioning and its associated increase in angle to be accounted for.

CONCLUSION

The new Hathaway Bridge is composed of two segmental concrete box girders, with 80-ft wide boxes and repetitive 330-ft spans. The boxes share a common foundation at each pier. The LRFD design of the bridge showed that for this structure, the following were true:

- The Strength-IV load group in many cases controlled the axial loads in the piling. This load group has no equivalent group in other US Codes.
- The pile tip elevations were controlled by a service load group we developed that included γ=1.00 for wind load, since the only service group that contains wind on structure in LRFD is Service-I, which includes γ=0.30 for WS. This was based on horizontal displacements of the superstructure under wind loading.
- For Transverse Bending, the amount of transverse post-tensioning required under the LRFD Code was approximately equal to the amount that would have been required under HS20-44 Loading. It was significantly less than would have been required under "HS25" Loading.
- Longitudinal Bending results showed that the quantities of concrete and longitudinal post-tensioning were consistent with past experience, and were actually on the low end of typical historical ranges.
- Shear studies showed that the LRFD Code, in this case, calculates that the webs of the box girder have more capacity than they would under the Chapter 9 or Guide Spec provisions.

REFERENCES

1. Podolny, W.; Muller, J.M. Construction and Design of Prestressed Concrete Segmental Bridges. New York: John Wiley & Sons, Inc; 1982.