Inspection and Load Rating of P-T Segmental Bridges

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1 Abstract

The interchange of the Veterans Memorial Tollway (I-355) and the Ronald Reagan Memorial Tollway (I-88) in Downers Grove, Illinois includes four post-tensioned (P-T) segmental concrete box girder bridges erected with different techniques. Built in 1988, they range in length from 170m (558') to over 610m (2,000') on horizontally curved alignments To maintain the integrity of this vital infrastructure, the Illinois Tollway tasked Ciorba Group to perform full in-depth inspection, material testing, assessment of the P-T condition, and load rating. As part of this project we performed non-destructive testing of the P-T strands at select locations using ground penetrating radar and impact echo technology.

After the detailed inspection, the bridges were load rated taking into account the deterioration noted and using a time dependent Finite Element Analysis. We re-calculated all stresses as "locked-in" during bridge erection. The ramp bridges were load rated for the current single lane configuration as well as a potential future two lane configuration. An overall load rating and condition report was prepared which evaluated various bridge repairs and strengthening options including the use of UHPC structural overlay.

Keywords: load rating, post-tensioning, segmental box girder bridges, bridge inspection, non-destructive testing, Ultra-High-Performance Concrete, UHPC

2 Introduction

The four concrete segmental box girder bridges which are part of the interchange of Veterans Memorial Tollway (I-355) and the Ronald Reagan Memorial Tollway (I-88) in Downers Grove, IL were built in 1988 and are currently owned and maintained by the Illinois Tollway. The location of these structures is shown in Figure 1.

Figure 1 - Location and aerial map

To determine the current condition of these bridges, a full load rating was undertaken for each of the bridges which included an arms-reach inspection of all four structures, non-destructive testing of the post-tensioning tendons, and load rating of the superstructures and substructures.

The substructure and superstructure elements for all four bridges were load rated utilizing the LFR method and HS-20 loading. The rating included any deterioration that was found during the hands-on inspection or non-destructive testing which would decrease the load carrying capacity. Load ratings for the three ramp structures were calculated for the current single striped lane of traffic, as well as for a potential future traffic configuration which would include two lanes of traffic.

3 Structure Descriptions

Below is a description for the three major structures of the four concrete segmental bridges that comprise the I-355/I-88 interchange which were load rated as a part of this project.

3.1 Bridge Number 1437

Bridge Number 1437 (Ramp EN) is a seven-span structure, approximately 276.76 m (908’-0”) long. The structure has a roadway width of 9.14m (30’-0”). All three ramps are currently striped to carry one lane of traffic. The structure is on a horizontally curved alignment with two curves of 659.01m (2,162.11’) and 408.37m (1,339.80’) radii located on the structure. To accommodate the curves, the structure has a superelevation of 4.2% to 5.8%.

The seven spans vary in length from 37.36m (122’-6 11/16”) to 36.87m (130’-9 ½”) with an out to out width of 10.11m (33’-2”). The concrete segmental box girders have a constant depth of 2.44m (8’-0”) and were erected utilizing a span by span method. The superstructure is supported by three-single column piers, two post-tensioned straddle bent piers spanning lanes of I-88, and a post-tensioned “C” shaped pier cantilevering over lanes of I-88. The piers vary in height from 9.75m (32’-0”) to 10.97m (36’-0”). The abutments are tall wall reinforced concrete abutments with adjacent concrete retaining walls. Elastomeric bearings are located at each support with expansion joints located at each abutment.

3.2 Bridge Number 1439

Bridge Number 1439 (Ramp SW) is a fourteen (14) span structure, approximately 598.93m (1,965’-0”) long. The bridge consists of three units, numbered from north to south: Unit 1 (Spans 1-4), Unit 2 (Spans 5-8), and Unit 3 (Spans 9-14). The structure varies in width from 12.55m (41’-2”) to 10.11m (33’-2”) from spans 1 thru 3 and is constant width of 10.11m (33’-2”) for the remaining spans. The structure is on a horizontally curved alignment with two curves of 579.12m (1,900.0’) and 231.65m (760.0’) radii on the structure.

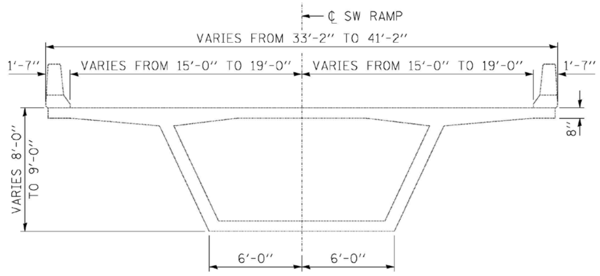

Figure 2: Bridge Number 1439 cross section, Bridge Number 1441 similar

The fourteen spans vary in length from 22.53m (73'-11") to 60.96m (200'-0"). The concrete segmental box girders have a constant depth of 2.44m (8'-0") with haunched locations over the three interior piers which are a 2.74m (9'-0") depth. Unit 1 was erected using the balanced cantilever method while the other two used the span-by-span method.

The superstructure is supported by reinforced concrete single or two column piers and stub abutments. The first pier of the first unit has vertical post-tensioning. The piers vary in height from 6.10m (20'-0") to 17.91m (58'-9"). The interior piers of the second unit are rigidly fixed to the superstructure; elsewhere the piers have elastomeric bearings or high load multi-rotational (HLMR) bearings. Expansion joints are located at each abutment and over the two piers (Piers 4 and 8) between the three units.

3.3 Bridge Number 1441

Bridge Number 1441 (Ramp SE) is a fourteen (14) span structure, approximately 614.17m (2,015') long. The bridge consists of three units, numbered from west to east: Unit 1 (Spans 1-5), Unit 2 (Spans 6-10), and Unit 3 (Spans 11-14). The structure is on a horizontally curved alignment with curves of 231.65m (760.0') and 308.21m (1,011.18') radii located on the structure.

The fourteen spans vary in length from 27.89m (91'-6") to 68.58m (225'-0") with an out to out width of 11.79m (38'-8") and depth of 2.44m (8'-0") and haunched to a 2.74m (9'-0") depth over the interior piers. Units 1 and 2 were erected using the span-by-span method while the Unit 3 used the balanced cantilever method.

The superstructure is supported by post-tensioned "C" shaped piers, and single or two column piers which vary in height from 7.01m (23'-0") to 15.93m (52'-3"). The bearings are similar to Bridge 1439 with HLMR and elastomeric bearings.

4 Field Inspection and Physical Evaluation

4.1 Scope of Inspection

Ciorba Group engineers completed a full in-depth inspection which included an arms-reach inspection of all four structures utilizing snoopers and manlifts. The substructure and superstructure elements were hammer sounded throughout, specifically at locations where deterioration was evident.

As a part of the in depth inspection, a thorough investigation and testing of the post tensioning strands through non-destructive testing (NDT) throughout all four bridges was performed by Vector Corrosion Services (Vector). Vector utilized ground penetrating radar and impact echo testing to locate the internal strands and determine the presence of voids in the HDPE ducts.

4.2 Inspection Findings

In general, the structures were in overall satisfactory to good condition. Deteriorated areas in the superstructure and substructure were present at locations of water runoff such as expansion joints. There were delaminations in the piers and at the precast segment joints directly adjacent to the expansion joints however no signs of openings were noted at the segment joints.

Vector performed NDT of the transverse and longitudinal post tensioning tendons throughout the superstructure and select tendons in the substructure that were accessible. Their NDT results showed that there were small lens voids present at high points in the ducts, but there was no corrosion present on the strands. In select locations at the expansion joints, the anchor block concrete was removed to verify if any corrosion was present in the tendons or anchorages due to water runoff and chloride contamination. With the concrete removed, corrosion was noted at the anchorages, but it didn't affect the tendons or the wedges themselves.

There was no evidence that the deterioration noted in the structures decreased the capacity of the structures from the original design, therefore the full load carrying capacity was used in the load rating analysis.

5 Methods of Analysis

5.1 Structural Analysis

The longitudinal analysis was performed using the LARSA 4D (LARSA) structural finite element analysis (FEA) software. The superstructure was modeled using beam elements along the curved horizontal alignment. The post-tensioning (P-T) tendons were modeled in the FEA software, which calculates short term tendon losses due to geometry, friction, and anchor set and generates a set of equivalent internal member forces. Each of the piers was modeled with beam elements. P-T tendons were also modeled for substructure elements which included them. Abutments were not modeled, but rather a fixed support with a bearing used. At piers and abutments with elastomeric bearings, the bearings were modeled with lateral linear equivalent springs which allowed rotation and translation along the longitudinal axis.

A staged, time-dependent construction model was created to accurately depict the stresses involved in the erection of the bridge. The models follow the construction sequence as shown on the as-built plans. Ramp SW Unit 1 and Ramp SE Unit 3 were built using the balanced cantilever method. For the other bridges and units, span-by-span construction was used. In balanced cantilever construction, a pair of segments was placed on each side of a pier block and permanent post-tensioning is installed to connect them. In span-by-span construction, an erection truss was used to assemble and support the entire precast span until the full span was completely assembled and then stressed. Closure pours then connect the span to the pier segments and post-tensioning is installed to create continuity. Creep and shrinkage effects were also modeled per the CEB-FIP 90 code [1]. Since casting and installation records were not available, a reasonable construction schedule of one week to erect a full span in span-by-span construction and 1 day to erect two segments in balanced cantilever construction was assumed.

Live loads were determined using influence lines. All bridges were analyzed for one and two lanes of traffic with the lanes offset to the left or right to maximize torsion effects.

5.2 Longitudinal Load Rating

The longitudinal superstructure load rating checked the stress, flexural strength, and shear strength for the inventory and operating levels.

Stress Check

The following three Inventory-level conditions were checked per MBE [2] 6B.5.3.3:

$RF = [0 \text{ ksi} - (F_d + F_{cs} + F_p + F_s)] / F_l$ (Concrete Tension)

$RF = [0.6f'_c - (F_d + F_{cs} + F_p + F_s)] / F_l$ (Concrete Compression I)

$RF = [0.4f'_c - 0.5(F_d + F_{cs} + F_p + F_s)] / F_l$ (Concrete Compression II)

Where:

F_d = Stress due to dead load

F_{cs} = Stress due to creep and shrinkage

F_p = Stress due to post-tensioning primary forces

F_s = Stress due to post-tensioning secondary forces

F_l = Stress due to live load plus impact

The allowable concrete tension is taken as 0 ksi for Type A joints (joints without minimum bonded reinforcement) per 9.2.1.2b of the Segmental Specifications [3] instead of 6 $\sqrt{f'c}$ in the MBE. When creep and shrinkage stresses improved the rating factor, a 0.5 load factor was conservatively applied to account for the uncertainty in creep and shrinkage models. Forces from the FEA output were taken and stresses were calculated by using a reduced area and moment of inertia based on the effective flange width provision according to section 3.3 of the Segmental Specifications[3]. Stress rating factors were calculated at every segment joint.

Flexural Strength

The following flexural strength conditions were checked per MBE 6B.5.3.3:

$[\phi_f M_n - (1.3M_D + M_{CS} + M_S)] / 2.17M_L (1 + I)$ (Flexural Strength Inventory)

$[\phi_f M_n - (1.3M_D + M_{CS} + M_S)] / 1.3\ M_L (1 + I)$ (Flexural Strength Operating)

Where:

M_D = Dead load moment

M_{CS} = Creep and shrinkage moment

M_S = Secondary prestress moment

M_L = Live load moment

I = Impact factor

The nominal moment capacity is calculated from either equation 9-13 of the Standard Specifications [4] when the depth of the compression block did not exceed the flange thickness or equation 9-14 when it did. The full flange width was used in this calculation. P-T tendons located entirely within the flange of the box (internal) were considered as bonded; all other P-T tendons (external) were considered unbonded. The strength reduction factor for flexure, ϕ_f, was taken as 0.90 per Table 7-1 in the Segmental Specifications[3]. When creep and shrinkage loads improved the rating factor, a 0.5 load factor was applied to them, similarly to the stress check. Flexural strength was checked at five locations within each span: at the face of the pier segments and at each quarter point.

Shear Strength

The following conditions were checked per MBE 6B.5.3.3:

$[\phi v V_n - (1.3V_D + V_{CS} + V_S)] / 2.17V_L (1 + I)$ (Shear Strength Inventory)

$[\phi v V_n - (1.3V_D + V_{CS} + V_S)] / 1.3V_L (1 + I)$ (Shear Strength Operating)

Where:

V_D = Dead load shear

V_{CS} = Creep and shrinkage shear

V_S = Secondary prestress shear

V_L = Live load shear

The nominal shear capacity is calculated by the procedure in 9.20 of the Standard Specifications. The effect of torsion is included by resolving the torsion into an equivalent shear force and adding it to the shear.

The shear force due to torsion in one web is calculated from the equation:

$V = Td_w/A_o$

Where:

V = Shear in one web

T = Torsion

d_w = Web depth

A_o = The area enclosed within the centerlines of the webs and flanges

The strength reduction factor, ϕ_v, was taken as 0.85 per Table 7-1 in the Segmental Specifications. When creep and shrinkage loads improved the rating factor, a 0.5 load factor was applied to them, similarly to the flexural and stress checks. Shear strength was checked at every node at a distance d_w or more from the bearing. The controlling shear location is shown in the results tables.

5.3 Transverse Load Rating

The transverse superstructure load rating checks the stress and flexural strength across the top flange of the precast segment. The equations used are the same as in the longitudinal rating, except that the allowable tension in concrete is 3 $\sqrt{f'c}$ in the transverse direction per 9.2.2.3 of the Segmental Specifications. The box section is the same for all bridges, except the length of the cantilevers varies. The typical transverse post-tensioning layout for all bridges, is two 4-strand tendons per 10' segment. In Ramp SW, Unit 1, the post-tensioning is increased to three tendons per segment for top flange widths between 11.68m (38'-4") and 12.45m (40'-10"). Therefore, a total of three sections were investigated: A 10.00m (32'-10") wide top flange (EN and the majority of SW), a 11.68m (38'-4") top flange (SW Unit 1 and all of SE) and a 12.45m (40'-10") top flange (SW Unit 1). The sections were rated for one and two lanes. The rating is checked at three critical locations: the outside and inside faces of the web (take at the point where the fillet begins) and at the center of the top flange. Additional mild reinforcement was added to the cantilevers for construction purposes which were not shown in the as-built plans; however, mild reinforcement has a negligible effect on stress rating which controlled.

5.4 Substructure Rating

Substructure elements with post-tensioning such as straddle bents and "C" piers were checked for stress, flexural strength, and shear strength, similar to the superstructure. The allowable stress is 6 $\sqrt{f'c}$ per 9.15.2.2 of the Standard Specifications. Since the MBE doesn't provide a procedure for load rating normally-reinforced columns, a simplified procedure was developed to calculate the load rating similarly to the superstructure rating. A column interaction diagram was created giving a $\phi_f Mn$ for each load case. The rating factor was then calculated using the following equations:

$$[\phi_f Mn - (1.3M_D + M_{CS} + M_S)] / 2.17M_L (1 + I)$$
(Flexural Strength Inventory)

$$[\phi_f Mn - (1.3M_D + M_{CS} + M_S)] / 1.3\ M_L (1 + I)$$
(Flexural Strength Operating)

Braking forces were not included in the analysis since they are not included in the Group I load case in the Standard Specifications. Centrifugal forces were also not included since they are only included in design truck loads, but design lane loads controlled substructure live load in the majority of locations. The effects of slenderness for tall columns were included by running the controlling load cases using a non-linear static analysis. The non-linear static analysis includes updated displacements in its equilibrium solution (as opposed to the small-displacement assumption). This essentially calculates the "P-delta moment" in columns due to lateral forces and moments.

6 Acknowledgements

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7 Conclusions

Based on the analysis described above, the load rating results for each bridge is detailed in Table 1.

Table 1.

HS-20 controlling inventory rating for one lane of traffic		
Bridge	Superstructure	Substructure
1437	1.64	1.28
1439	1.00	1.23*
1441	1.00	1.24*
HS-20 controlling inventory rating for two lanes of traffic		
1437	1.10	0.85
1439	0.50	1.23
1441	0.50	1.24

*Mild reinforced concrete piers were analyzed for two lanes only

For the current traffic configuration for each bridge, there were no deficiencies noted which would require strengthening or load posting.

When analyzed for potential two lanes of traffic, there were isolated deficiencies in three of the bridges. The noted deficiencies in Bridges 1439 and 1441 were in the negative moment over the webs for the transverse rating. Adding fiber wrap or additional post-tensioning to improve the transverse load rating is not feasible for the location of this deficiency. One possible method is to add a 38 mm (1½") thick Ultra-High Performance Concrete (UHPC) overlay. The overlay bonds and acts compositely with the existing deck, increasing the section modulus. It also has a much higher allowable tension. The addition of the 1½" structural overlay would increase the inventory rating for two lanes of traffic over 1.0. However, with the cost of UHPC and the size of the structures, this repair would be cost prohibitive.

The deficiencies in the substructure elements of Bridge 1437 could be improved by adding additional external post-tensioning. Adding fiber wrap could improve the tensile stress rating by only a marginal amount and wouldn't be effective in increasing the inventory rating. This was observed in the load rating calculations as there is already fiber wrap in place at Pier 3. Similarly, to the overlay, the additional external post-tensioning for these substructures units could be cost prohibitive.

8 References

[1] Comite Euro-International du Beton (CEB) and the Federation International de la Precontrainte (FIP), *Model Code 90;* 1993.

[2] American Association of State Highway and Transportation Officials (AASHTO), *Manual for Bridge Evaluation*. 2nd Edition; 2011

[3] American Association of State Highway and Transportation Officials (AASHTO) *Guide Specifications for Design and Construction of Segmental Concrete Bridges*. 2nd Edition; 1999 Interim Revisions

[4] American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges.* 17th Edition; 2002