Canada Drive Bridge: Ontario's First Post-Tensioned NU Girder Bridge

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Abstract

The Canada Drive Bridge is a newly completed vehicular and multi-use pedestrian bridge located in Vaughan, Ontario that enhances the city's transportation network and connects communities across the twelve-lane Highway 400. The project presented many technical challenges, including close collaboration with local regulatory authorities and fabricators, to modify the standard NU1400 girder to include post-tensioning – a first in Ontario.

The Canada Drive Bridge is a two-span slab-on-girder bridge consisting of two equal 40.5 metre spans with seven precast NU1400 girder lines at a spacing of 2.05 metres. Since the bridge ties into existing intersections on each side of Highway 400, there was a need to design the superstructure to be as slender as possible to keep the approach slopes acceptably gradual. While steel box girders would be a natural response to this constraint, schedule and cost considerations favoured a precast concrete solution. The addition of post-tensioning allowed the slenderness of the precast girder alternative to match that of a steel box superstructure. Extensive collaboration was required to ensure the design complied with all stakeholder requirements, and with local precast fabricators to adapt their processes and formwork to the addition of post-tensioning.

This paper presents the design and construction of the bridge, which provides an economical connection for the City of Vaughan, while extending the capabilities of NU Girders in Ontario.

Introduction

The Canada Drive bridge is located in Vaughan, Ontario, crossing over Highway 400 between Major MacKenzie Drive and Teston Road. It provides a new neighborhood connection between Canada Drive on the west and America Avenue on the east. The bridge carries two 3.5 metre wide traffic lanes with 0.5 metre shoulders and two 3.0 metre wide multi-use paths (MUPs) on each side for pedestrians and cyclists.

The project was procured by the City of Vaughan through a design-build contract, with ownership of the bridge to be assumed by the Ministry of Transportation of Ontario (MTO) upon completion. EllisDon Civil was awarded the contract in July 2023, with Entuitive providing structural design for the bridge and AECOM providing Civil, Landscape, Drainage, and Electrical design, as well as Project Management.

The bridge is the first post-tensioned NU-girder bridge in the MTO's inventory. The implementation of this non-standard design for the project was driven by the constrained site, with limited space available for embankments necessitating a shallow superstructure in order to keep the approach slopes manageable. While a steel superstructure would have been a more conventional solution, long lead times for steel fabrication presented a challenge to the project schedule. The Owner's openness to a new structural system for the jurisdiction enabled the project to be delivered economically and on-schedule.

Figure 1. Completed Canada Drive Bridge



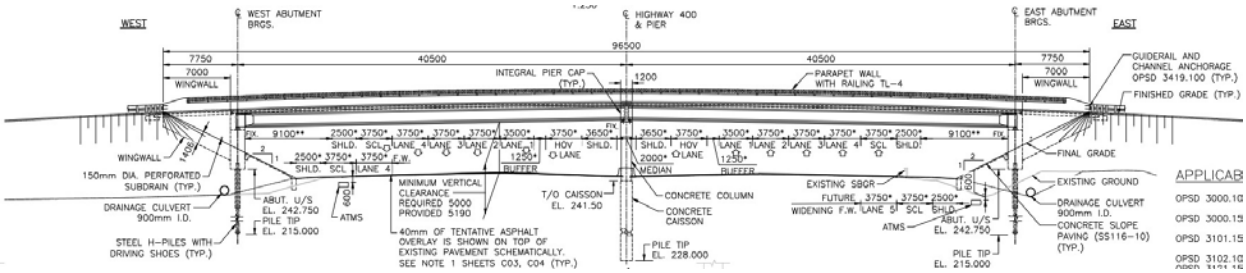
Source: Entuitive

General Arrangement

As shown in the General Arrangement elevation in Figure 2, the bridge consists of two equal 40.5 metre spans with integral abutments and an integral pier. The cross section, shown in Figure 3, has seven

NU1400 concrete girder lines at a centre-to-centre spacing of 2.05 metres with a total bridge width of 14.6 metres. The girders are composite with the 225 mm thick cast-in-place (CIP) concrete deck. The grade of the approach is a relatively steep 7.5% slope to accommodate the connection with the existing intersections on either side of Highway 400. The deck is overlaid with a 90 mm asphalt and waterproofing system within the vehicular lanes, and 240 mm thick sidewalks with cast-in-place barriers along the edges of the deck. The girders and deck are integral with the abutments and the pier.

Figure 2. Elevation Drawing of Structure

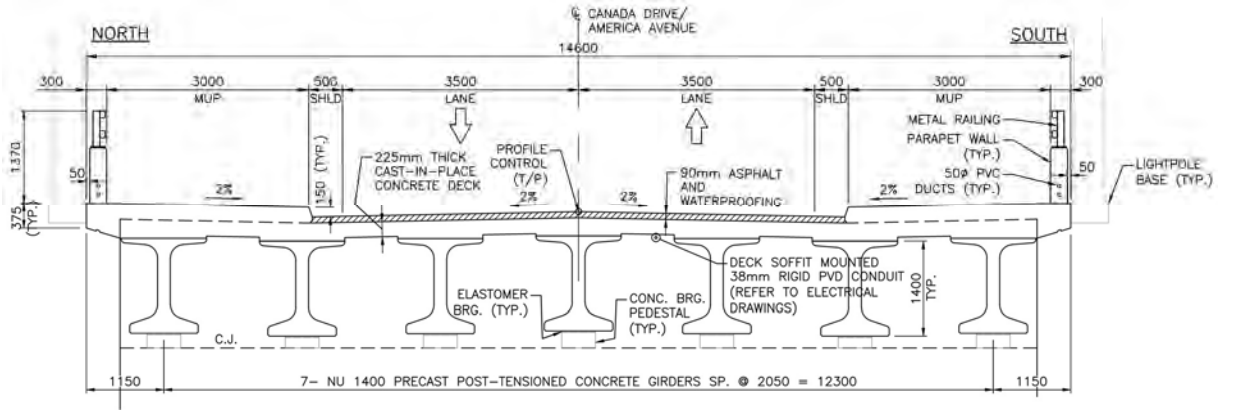


Source: Entuitive

Superstructure

To accommodate post-tensioning ducts, the Ontario-standard NU girder shape was modified by widening the web from 160 mm to 190 mm. This width allows a 91 mm diameter duct to be embedded in the web while maintaining 30 mm nominal cover each side. Additionally, the girder ends at the abutments have been modified with an end block to accommodate the post-tensioning anchorage hardware. Close collaboration occurred between the design team and the girder fabricator to modify the formwork and fabrication procedure in an efficient and cost effective manner. Girder detailing is described in further detail below.

Figure 3. Typical cross section



Source: Entuitive

Substructure and Foundation

Each integral abutment is founded on a single row of 14 - HP310x110 piles, with the weak axis of the piles oriented parallel to the bridge alignment. The pier consists of four 1067 mm diameter columns,

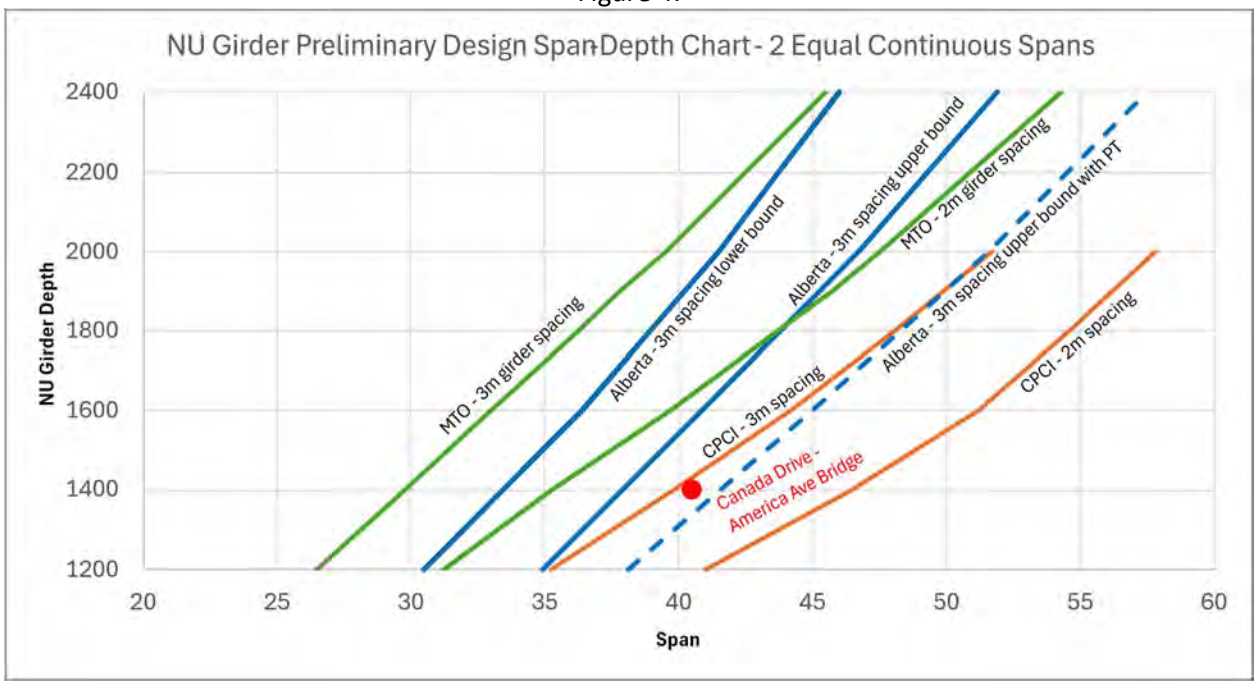
each founded independently on a 1500 mm diameter concrete caisson to eliminate the need for the excavation and temporary shoring that would be required to construct a pile cap.

Preliminary Design

The Reference Concept Design provided with the Design-Build RFP was for a steel box-girder superstructure, with 1450 mm deep steel girders. With an overall structural depth of approximately 1750 mm, the span-to-depth ratio of 23 is comfortably within economical range, and the superstructure depth is compatible with the highly constrained geometric design for the road profile and the clearance requirements over Highway 400. However, schedule constraints, along with fabrication lead-times and steel pricing at the time of bidding provided strong incentives to develop a concrete alternative. The challenge was that for typical girder spacing, the practice for NU girder bridge slenderness in Ontario is a span-to-depth ratio of approximately 19 – corresponding to NU1800 or NU1900 girders and a structure depth of over 2000 mm. This depth of structure was simply not possible given the geometric constraints of the site.

While a slenderness ratio of 23 is beyond typical practice in Ontario, it was recognized that it is commonly achieved in other jurisdictions. The design guide for NU girders published by the Nebraska Department of Roads[1], where the girders were developed, indicates spans of up to 48.5 m for two-span continuous NU1350 girders at 6' spacing – without post-tensioning. The preliminary design charts published by CPCI[2] are generally consistent with the Nebraska design charts. Within Canadian jurisdictions, both the Alberta[3] and Ontario[4] Ministries of Transportation have published design charts for NU girder bridges, including design charts for preliminary sizing of girders. The large variability in span ranges indicated by different sources reflects the sensitivity of the structural system to relatively small changes in design criteria or assumptions. This sensitivity arises from the fact that precast girder bridges are typically governed by SLS stress limits meant to ensure durability of the structure, either at transfer or in service.

Figure 4.



Data Source: MTO Prestressed Concrete Girder Guidelines (DRAFT) (2023) and Alberta Transportation NU Girder Bridge Design and Detailing Manual Vol I (August 2018)

While these charts are more conservative, due to jurisdiction-specific stress limits, available concrete strengths, and other aspects of local standard practice, there is precedent for post-tensioned NU girder bridges in Alberta with slenderness ratio of 23 and higher, which justified exploring the feasibility further through preliminary modelling and design checks. These checks indicated that 40.5 m spans were in fact feasible with NU1400 girders, for girder spacing up to 2.5m (6 girders). Important keys to achieving this span was to increase the concrete strength at transfer from 40 MPa to 45 MPa, and to recognize the benefit of the unusually tall haunches created by the vertical crest curve of the bridge deck (which offset the added demand due to the additional weight).

While NU1400 girders with pretensioning only were shown to be technically compliant with the SLS stress limits and ultimate limit states prescribed by the Canadian Highway Design Code[5] and the MTO Structural Manual[6], it was agreed, in consultation with MTO, that the addition of post-tensioning would allow for better control of deflections, more robust continuity over the pier, and assurance that despite deviating from typical local practice, the long-term durability of the bridge would be consistent with the Ministry's standards for new construction.

Detailed Design Considerations

As post-tensioning NU girders was a new approach for the owners, contractors and fabricators involved, additional analysis was carried out to demonstrate that the behaviour of the structure would be consistent with other comparable bridges in the MTO's inventory. The additional analysis focused on three main areas:

1. Camber of the girders during the structure's service life
2. Future roadway modifications
3. Live load methodology

Each of these is discussed in further detail below.

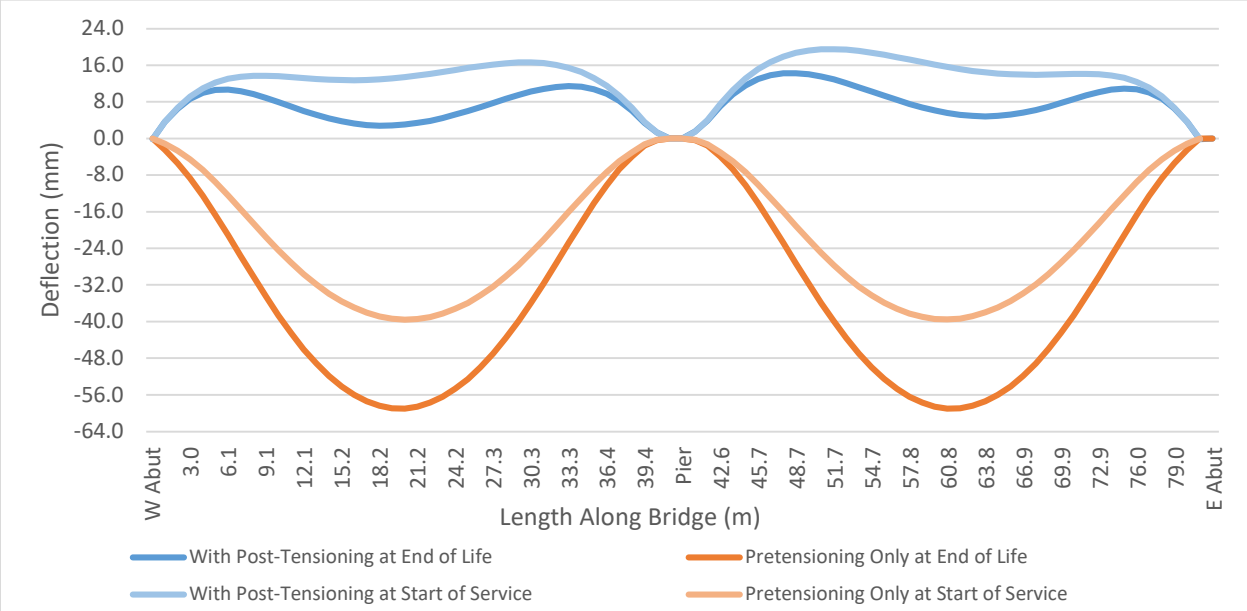
Girder Camber

A greater understanding of the camber of the girders throughout their service life was important for this bridge because of the tight geometry of the site and the required performance of the structural system. The clearance between the bridge and Highway 400 below had little tolerance for downward deflection of the girders. Furthermore, concerns were raised that any sag in the girders could be perceptible in the foreshortened view from the approaches, and that this could be alarming to the public, even though it did not signify any true deficiency in the structure.

Through a staged finite element analysis, it was determined that the bridge with a six pretensioned NU1400 girders would have an initial downward deflection of 47 mm at the start of service, with creep effects over a 75 year life increasing the sag to 60 mm at the end of life (Figure 5). For the reasons described above, the project team deemed this deflection to be unacceptable.

Post-tensioning provides an effective means of controlling the deflection of the bridge. It was determined through the finite element model that post-tensioning the girders would provide positive camber throughout the service life of the structure, with a final camber at the end of the bridge's service life of 3 mm upward deflection (Figure 5).

Figure 5. Deflection of Exterior Girder (Pretensioned vs Post-Tensioned)



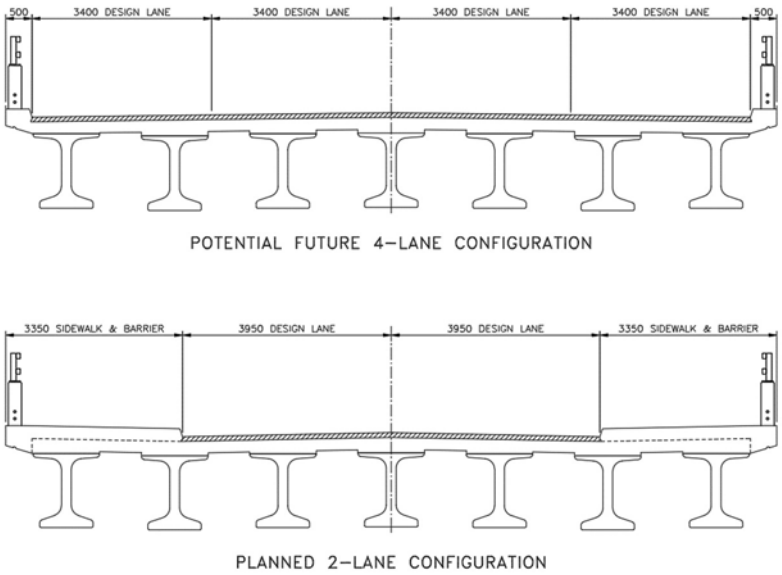
Source: Entuitive

Future Roadway Modifications

The provision of maximum flexibility for future use of the bridge was also a significant consideration for the MTO. In addition to the current multi-modal transportation configuration, a possible future configuration was considered, with four lanes of vehicular traffic and no sidewalks, as shown in Figure 6. The exterior girders are the most impacted by the future configuration. The current configuration imposes large permanent dead loads on the exterior girders from the weight of the 3.0 metre wide MUP

sidewalk, while the future configuration imposes lesser permanent loads but greater live loading from the additional lanes of vehicular traffic.

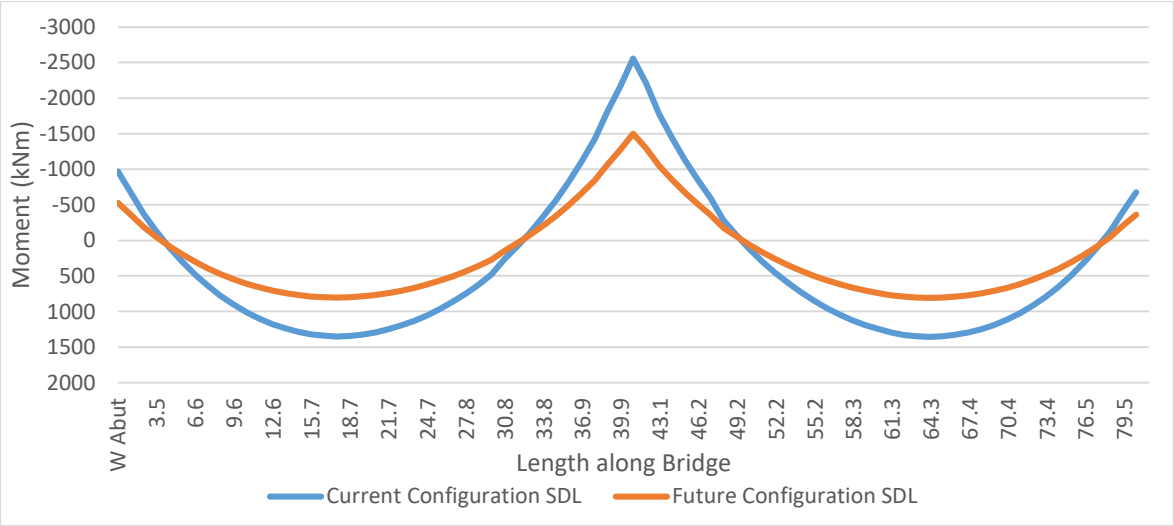
Figure 6. Considered Roadway Configurations



Source: Entuitive

Permanent loads have significant influence over the long-term creep and prestress effects, which in turn impact the long term deflection of the structure. Designing the structure for these two conditions while maintaining similar deflection performance posed a significant challenge as the permanent moment in the exterior girders with the MUP decreases by nearly 40% when converted to the future configuration (Figure 7). In comparison to the interior girders, the exterior girders required additional post-tensioning to accommodate the permanent load of the MUP. As such, the exterior girders were designed with 3 – 12 strand post-tensioned cables while the interior girders were designed with 2 – 12 strand cables.

Figure 7. SDL Moment along Exterior Girder – Current and Future Roadway Configurations



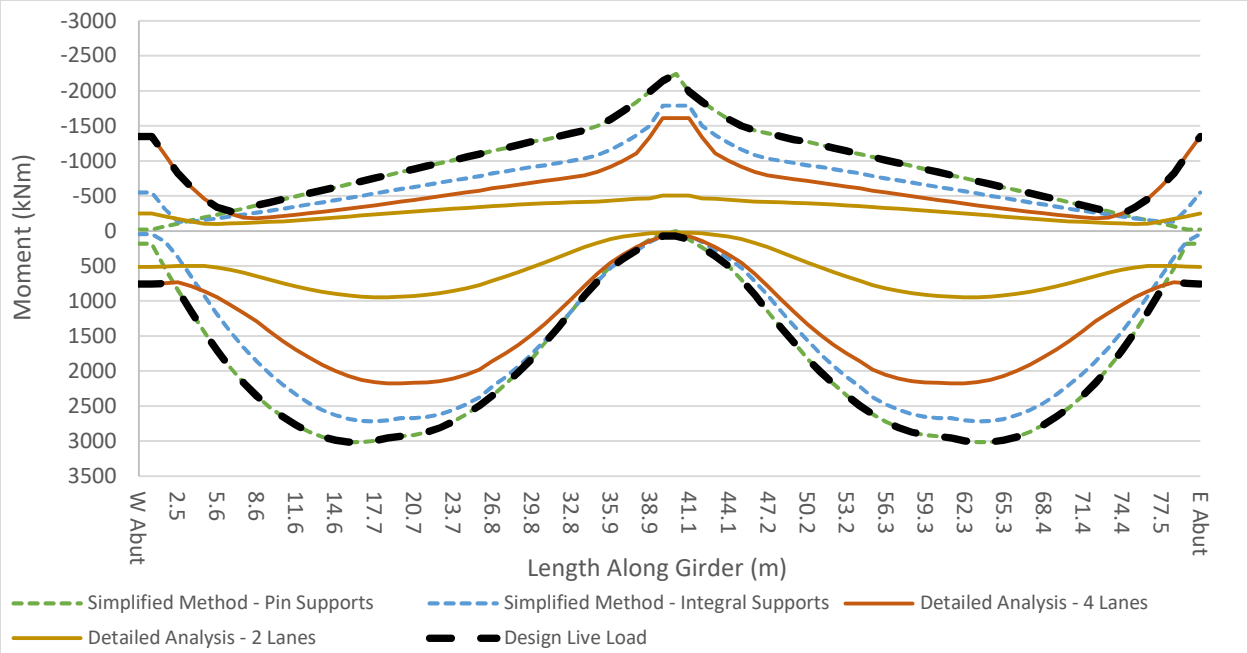
Source: Entuitive

Live Load Methodology

A comparative analysis of live load methodology was also performed on this structure. The comparison focused on the application of the CL-625-ONT vehicular loading per CSA S6:19 Canadian Highway and Bridge Code (CHBDC) through the Simplified Method and finite element analysis (FEA) using CSiBridge structural analysis software. The design live load for each girder is the envelope of the following analysis cases and shown in Figure 8:

- Detailed Analysis – Two Lane, CSiBridge model
- Detailed Analysis – Four Lanes, CSiBridge model
- Simplified Analysis – Pin Supports, Simplified Method neglecting rotational restraint at abutments and piers
- Simplified Analysis – Integral Supports, Simplified Method including rotational restraint at abutments and piers

Figure 8. Design Live Load Envelope for Exterior Girders



Source: Entuitive

The live load moment from the Simplified Method was calculated for the interior and exterior girders by Clause 5.6 in CSA S6:19, for the four-lane future configuration. The Simplified Method is not applicable to bridges with wide sidewalks; therefore, it was not used for the current multi-modal configuration. This method used a simplified model for the bridge superstructure that is a two-span continuous beam of constant stiffness. Two support conditions were considered to determine the upper- and lower-bound moments along the girders' entire span: two-span continuous with pin-supported ends, which allow unrestricted rotation under applied loads, and two-span continuous with integral-supported girder ends, which provide some rotational stiffness. The pin-supported case is used to calculate the maximum positive moment at midspan, while the integral support case is used to calculate the maximum negative moment at the girder supports. The simplified methods provide a conservative bound of the expected live load moments experienced by the bridge.

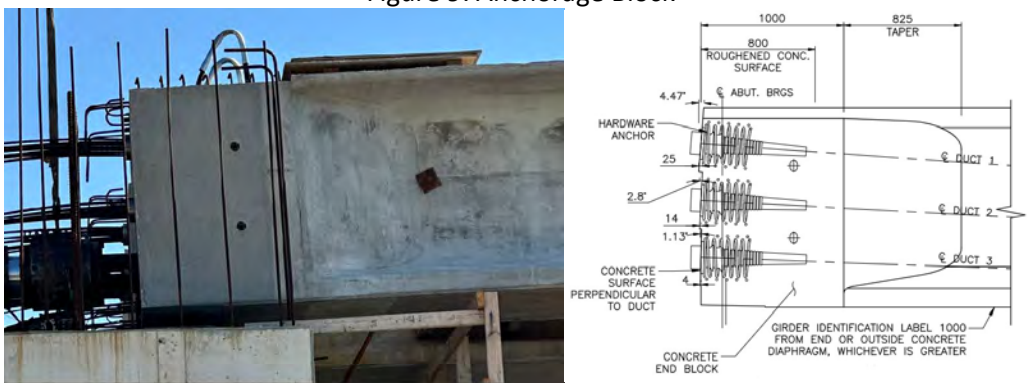
Live load moments were also calculated using CSiBridge finite element analysis software. Two cases were considered with the method, one with two traffic lanes and another with four traffic lanes to represent the initial and future configurations of the bridge, respectively. The CSiBridge model provides a more accurate depiction of the distribution of live load among the girders and accounts for varying stiffness in the deck from cracking in negative moment regions over the pier. With this method, a lower positive moment is developed at midspan of the girders, however a larger negative moment is developed at the abutments due to additional restraint cause by the stiffness of the wingwalls.

Modifications to Typical NU Girder Construction in Ontario

Anchorage Blocks

The critical modification to the standard NU girder forms required to allow post-tensioning is the anchorage block at the stressing end of the girders. As the girders for this project were the first post-tensioned NU girders fabricated in Ontario, the design team worked closely with the MTO and the precaster, Decast, to design and detail the anchorage blocks. The primary objective of the anchorage block is simply to provide sufficient width to accommodate the anchorage hardware and local-zone bursting reinforcement. In principle it would be advantageous to keep the block as narrow as possible, to limit the "general zone" bursting stresses caused by transverse spreading of the compression forces within the block. However, to simplify the forms and to limit the unavoidable reinforcement congestion, the anchorage block was designed to be the same width as the bottom flange. The top flange, which extends 125 mm beyond the bottom flange on each side, is simply terminated where the end block begins. A 2:1 sloped taper provides a smooth transition between the anchorage block and the web. Once the integral abutment is cast, the anchorage blocks are entirely concealed, cast into the abutment stem. Two holes are provided in the block for dowels to provide reinforcement across the sidewall interfaces between the girders and the diaphragm.

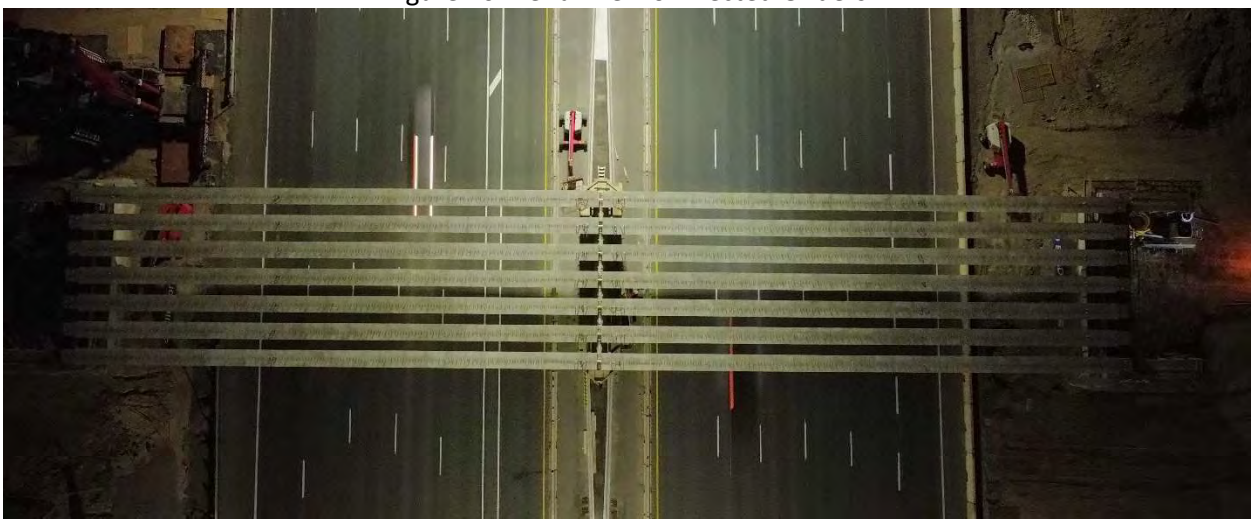
Figure 9. Anchorage Block



Source: Entuitive

Construction Sequence

Figure 10. Aerial View of Erected Girders



Source: Ellis Don

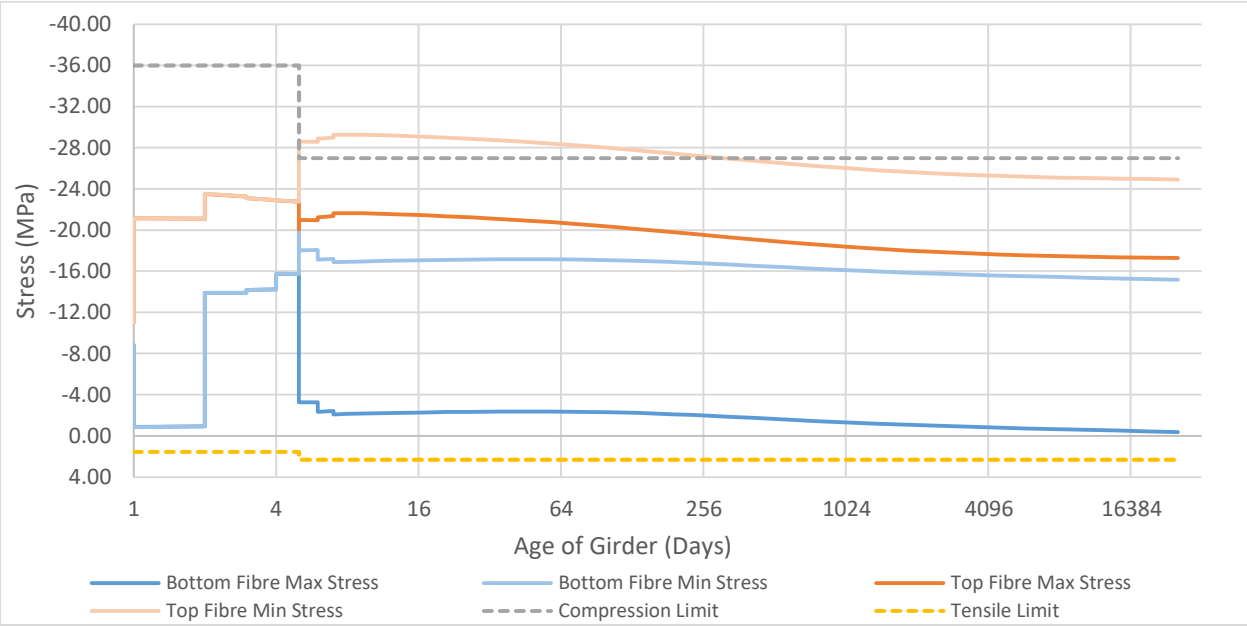
The extended span range of the NU1400 precast girders in the Canada Drive Bridge required detailed analysis of the construction sequence for pre-tensioning and post-tensioning elements to meet all design criteria including stress and deflection limits. The sequence of construction was governed by two criteria that were required to remain within the specified limits: the stress in the extreme fibres of the girder, and the deflection of the girder during service.

The stress limits in the girders are as follows:

1. Stress in strands:
    a. Stress $> 0.45f_{pu}$ after all losses
    b. Stress $< 0.45f_{pu}$ immediately prior to transfer
2. Stress in concrete:
    a. Compression $< 0.6f'_{ci} = 27\ MPa$ at transfer
    b. Compression $< 0.6f'_{c} = 36\ MPa$ during construction and service life
    c. Compression $< 0.45f'_{c} = 27\ MPa$ due to dead load and prestress after all losses
    d. Tension $< 0.6f_{cri} = 1.34\ MPa$ at transfer
    e. Tension $< 0.5f_{cr} = 1.55\ MPa$ at transfer
    f. Tension $< 0.75f_{cr} = 2.32\ MPa$ during service life

Typical construction sequencing in other jurisdictions with more common use of NU post-tensioned systems were considered in the design. In Alberta, it is common to have the cast-in-place deck constructed before the post-tensioning operations. This sequence was considered for the Canada Drive bridge with the stress in the girders plotted over time, as shown in Figure 11. As indicated in the figure, the stress in the top fibre of the girder exceeds the compressive limit for approximately 250 days at the beginning of service before creep effects become significant. This exceedance is caused by the concrete strength of 60 MPa used for precast NU girders in Ontario, compared to 70 MPa used in Alberta.

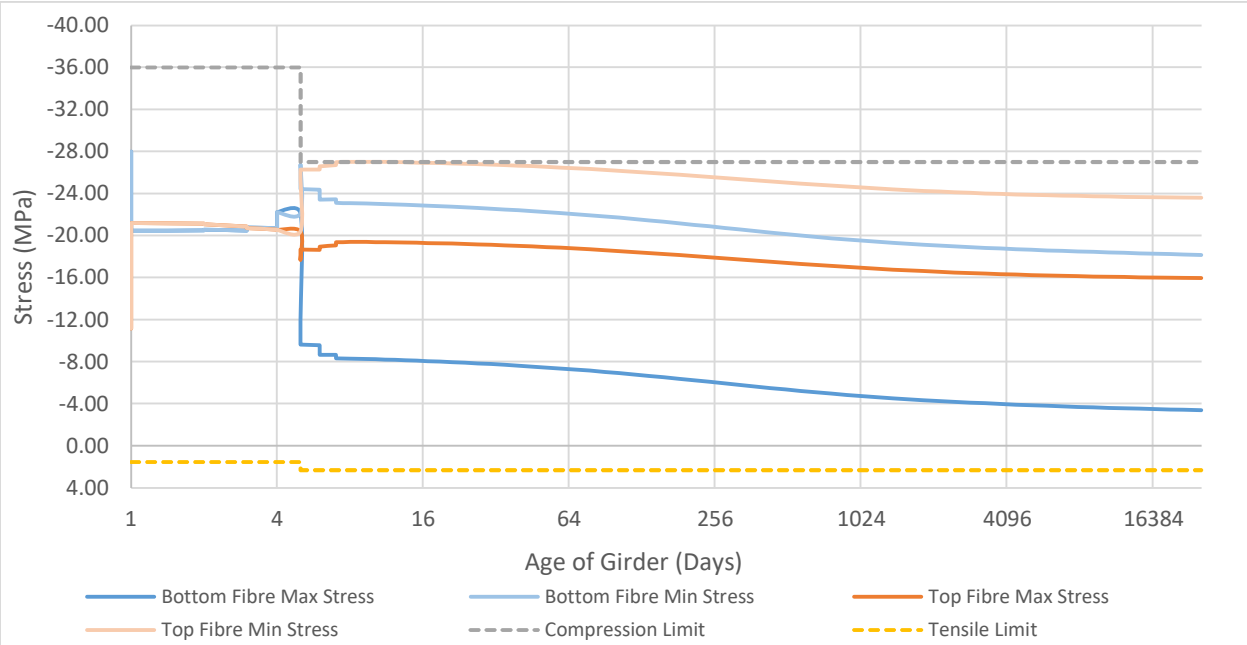
Figure 11. Total SLS Stress in Top and Bottom Fibres of the Girder Over Time at Midspan – Cast Deck Before Post-Tensioning Sequence (From Deck Cast to End of Life)



Source: Entuitive

To maintain compliance with concrete stress limits and standard practice for the MTO, the construction sequence was modified to cast the deck after post-tensioning operations. As shown in Figure 12, the stress at the start of service is reduced, as the post-tensioning is able to participate in resisting the weight of the deck.

Figure 12. Total SLS Stress in Top and Bottom Fibres of the Girder Over Time at Midspan – Cast Deck After Post-Tensioning Sequence (From Post-Tensioning to End of Life)

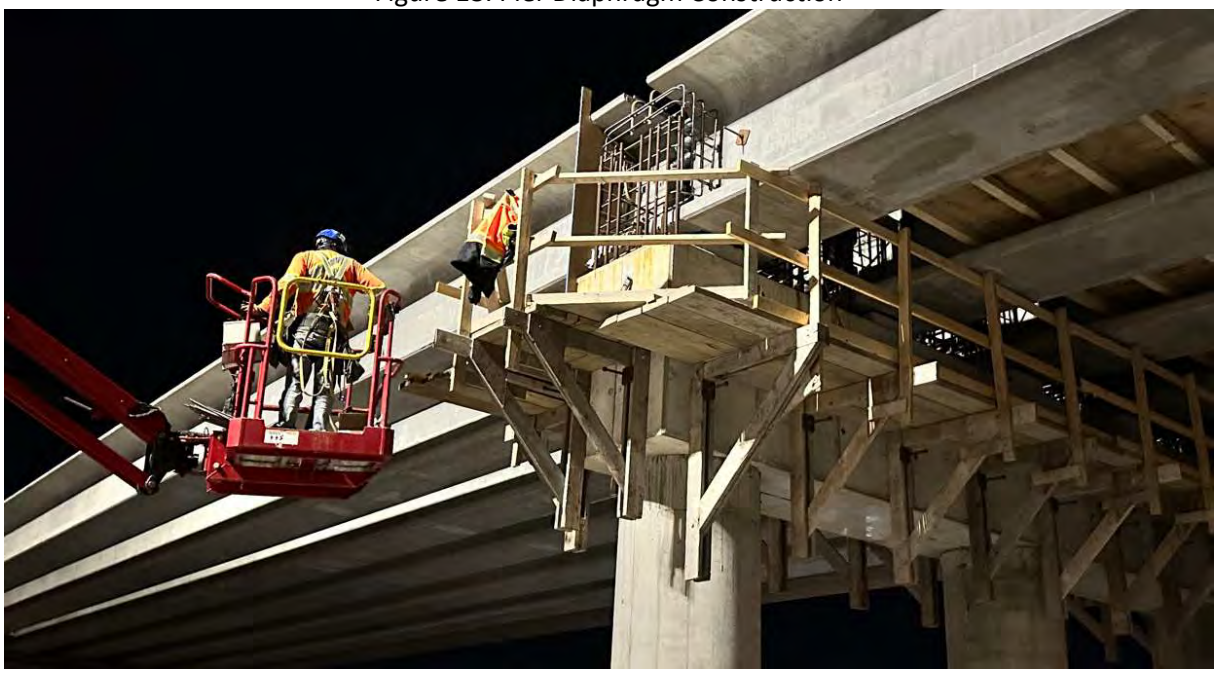


Source: Entuitive

Casting the deck after post-tensioning introduces the need to cast the pier diaphragm as a separate pour, to allow the ducts to be spliced, post-tensioning strands to be pulled across the two continuous strands, and the girders to be made continuous prior to the post-tensioning operation.

This sequence of work, with the deck being cast last, is also consistent with typical MTO practice to cast decks in an unshored condition, which simplifies any future rehabilitation involving full or partial deck replacement.

Figure 13. Pier Diaphragm Construction



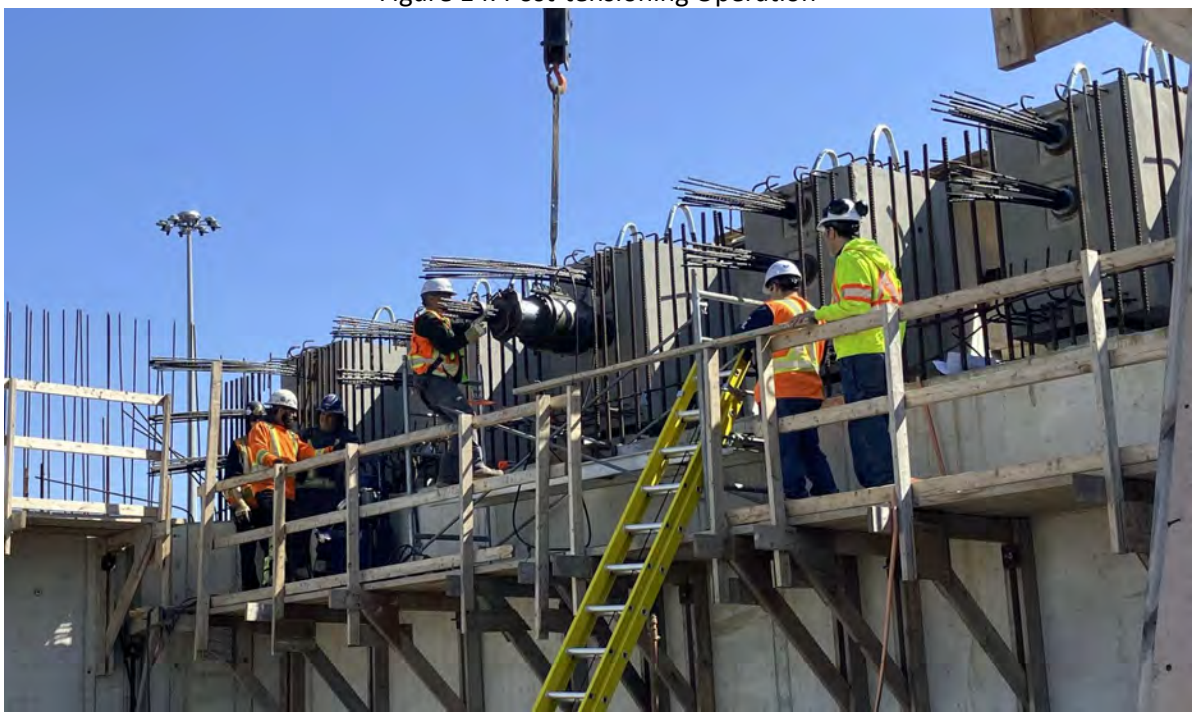
Source: Entuitive

Implications for Integral Abutment Design

Integral abutments are designed to accommodate longitudinal movement of the bridge through bending of the flexible foundation piles. While the magnitude of movement produced by temperature fluctuations can be accommodated in this way, the elastic shortening of the girders due to post-tensioning operations would create movements too large to accommodate by flexing the piles, and the corresponding restraint would cause an undesirable loss of prestress. For these reasons, the girders are first erected on laminated elastomeric bearings. These bearings are designed to allow the longitudinal movement due to the initial post-tensioning with minimal restraint, thus preventing load transfer into the substructure and ensuring that the girders see the full benefit of the post-tensioning force. The abutment diaphragm is cast with the deck and the full abutment is made integral with the superstructure after the post tensioning operation is complete.

This sequence for post tensioned integral abutments has the additional benefit of providing protection for the post tensioning anchorages from environmental exposure over the life span of the structure.

Figure 14. Post-tensioning Operation



Source: Entuitive

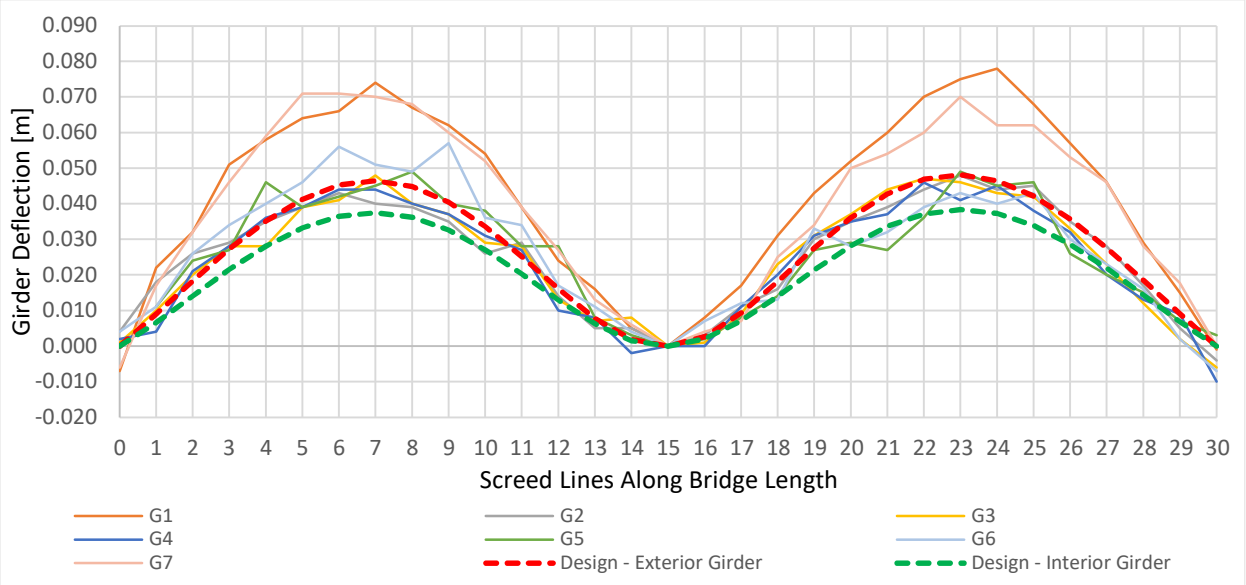
Construction Survey

The condition of the structure was surveyed throughout all stages of construction to ensure the behaviour was aligned with the design criteria. Elevation surveys of the girder positions were conducted at three significant construction milestones:

1. Before the post-tensioning operation, after the pier diaphragm was cast
2. After the post-tensioning operation
3. After the top flange prestressed strands were cut

These surveys showed a larger camber in the girders than what was initially expected in the design. The interior girders had on average 7 mm of additional camber while the exterior girders had on average 23 mm of additional camber, as shown in Figure 15.

Figure 15. Deflection of Girders from Post-Tensioning Operation (As-Built vs. Design)



Source: Entuitive

It was determined through analysis that several factors were contributing to the discrepancies between the surveyed camber and the design camber of the girders:

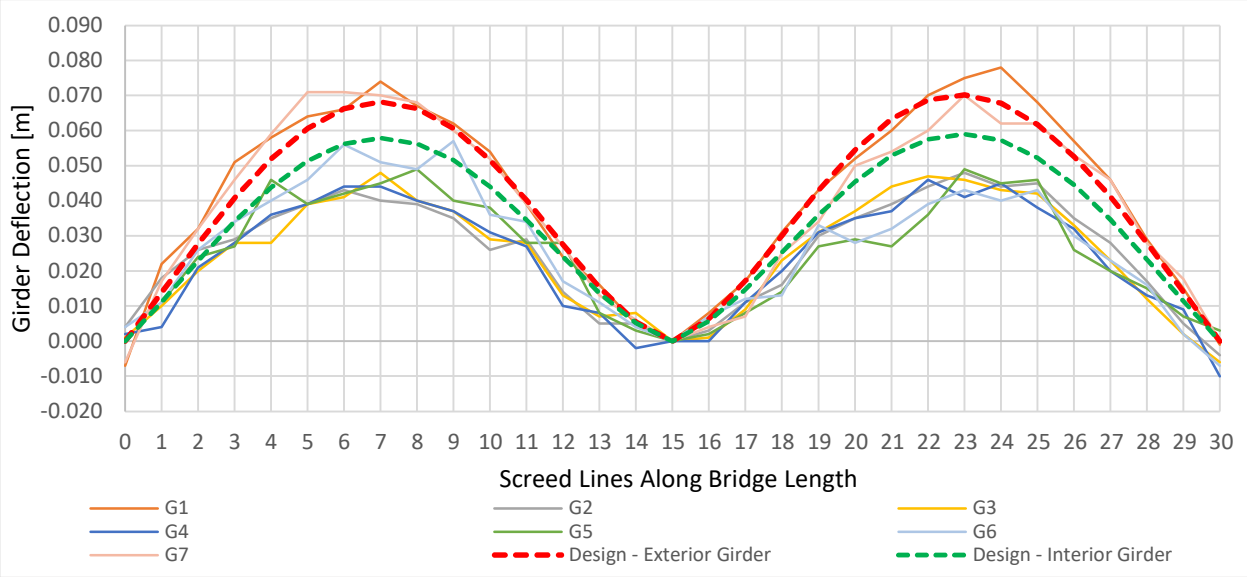
- Lower than anticipated post-tensioning losses
- Additional time than anticipated for concrete creep effects (prolonged construction schedule)
- Lower than assumed stiffness of the girder concrete by approximately 6%
- Variations in the predicted creep effects of the girder concrete
- Additional deflections due to thermal gradient in the girders at the time of the survey caused by hot sunny days

Variability in the fabrication parameters of the girders was one major element contributing to the discrepancies. Material properties of the concrete such as stiffness and creep can vary in as-built conditions from predicted models. Creep effects in concrete are known to be difficult to predict accurately and can have wide variations in magnitude. A sensitivity analysis of the creep in the girders was performed with the effects adjusted by ±25% from the original design assumptions. It was determined that the girders were approximately 100 days older than the design assumptions when erected. Age-adjusted effects were applied to the effective modulus of the girder concrete based on the actual age to more accurately estimate the stiffness.

It was measured on site that during a sunny day, the temperature differential between the top flange and the bottom flange of the girder could be up to 10°C. This would cause the girders to deflect upwards as the top of the girder would increase in length from the heat more than the bottom of the girder. This deflection from the temperature differential was determined to provide an additional 14 mm of camber at the time of the survey.

With the design parameters adjusted to more accurately reflect the as-built condition of the structure, the predicted camber more accurately reflects the surveyed results of the structure, as shown in Figure 16.

Figure 16. Deflection of Girders from Post-Tensioning Operation
(As-Built vs. Adjusted Design with Temperature Effects)



Source: Entuitive

Conclusion

Construction of the Canada Drive Bridge in Vaughan, Ontario was completed, on-schedule, in Summer 2026. The design of the bridge responded to the particular challenges of the site and the project's context: constrained roadway geometry, an inflexible schedule, economic and procurement realities, and the need to consult with and meet the standards of both municipal and provincial government stakeholders. The result is an economical, constructible design which sets a precedent for post-tensioned NU girder construction in Ontario.

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