Building on Our Growth Opportunities May 27 – 30, 2015
Miser sur nos opportunités de croissance REGINA, SK

DESIGN OF A TWO-SPAN SEMI-INTEGRAL ABUTMENT BRIDGE WITH CONTINUOUS CURVED STEEL GIRDERS

Armando Chocron[1], Tony Cicchetti[1], Patrick Clinton[1], Mehdi Khodadadian[1], Christopher Mercieca[1], Kayhan Turker[1], S. Salib[2], and K.Sennah[3]

[1]Capstone group students, [2]Industrial Advisor/Faculty Instructor, [3]Technical Advisor, Department of Civil Engineering, Ryerson University, Toronto, Ontario, Canada

Abstract: In Canada and within the Greater Toronto Area (GTA) in particular, there is constant demand to improve the transportation infrastructure. Herein, the subject project involves a new 2-span continuous curved steel girder bridge which merges a major road with a highway. The bridge is of a semi-integral abutment type which has been adopted frequently in North America over the past years due to its favourable characteristics. In these bridges, the deck extends beyond the abutments and bends vertically down as a ballast wall to shift the expansion joints outside the structure itself. This serves to protect both superstructure and substructure components, and especially the bearings, which are delicate and costly components to repair or replace. Moreover, in order to provide the most durable and cost effective bridge design, 'Box' and 'I' girder alternatives were investigated. During the design phase, as well as during construction, the complexity of curved bridges poses several challenges. Among them, are the significant torsional straining actions induced in the individual girders and the entire deck system due to their curved geometry. Therefore, the analysis, design and construction should accommodate the related stability concerns associated with girder erection and deck construction, as well as during the entire service life of the bridge. Further, the subject project is also challenged by the vertical clearance required for the vehicular traffic flow under the bridge. Through a sensitivity study, various deck design options were investigated from both technical and financial aspects in order to optimize the satisfaction of this requirement.

Key words: Semi-integral Abutment, Curved Bridge, Box Girder, Design Optimization, FEM

1. INTRODUCTION

1.1 Background

In 1938, the first integral bridge was built in Ohio, USA (Burke, 2009). Thereafter, the construction of bridges with no movable transverse deck joints spread around the world. In North America in particular, the use of integral and semi-integral bridges has become prominent over the past few years. The development of software has enabled engineers to analyze more complex indeterminate structures and in the case of semi-integral bridges, it allowed for sophisticated soil-structure interactions to be modelled and incorporated into the design. A semi-integral bridge features a single or multi-span continuous superstructure that extends beyond the abutments and then bends vertically down as a ballast wall. Consequently, the bridge expansion joints are relocated within the approach zones. Bridges with expansion joints at piers and abutments are often less durable than semi-integral bridges due to the vulnerability of bearings, superstructure and substructure components to the attack of leaked de-icing salts and water. Bearings, in particular, are of the most costly components on bridges to repair and/or replace. Therefore, while they are vital to provide sufficient lateral, longitudinal and rotational restraint to maintain the bridge stability, they are well protected throughout the entire service life of the bridge. Furthermore, jointless superstructures reduce the dynamic effect of traffic loading and that of the pavement G/P phenomenon (Burke, 2009).

The paper herein, first introduces the bridge and its components and then describes the two girder options being evaluated for implementation. A recommendation is made on the most suitable alternative considering factors such as, but not limited to, cost, ease of erection, feasibility, durability and aesthetics. Thereafter, a detailed analysis is carried out on the selected option to verify, revise and optimize the design of the recommended option.

1.2 Description of Bridge and Scope of work

The development of new and effective transportation systems in the GTA is vital towards supporting the continuous growth currently being experienced in the city. The subject project serves the purpose of joining a major road with a highway. The bridge consists of 2 continuous spans and is of semi-integral abutment type. As shown in Figure 1, each span is 50 m long and increasing in elevation at a constant slope of 6%. The bridge will host two lanes and their corresponding shoulders. As shown in plan, Figure 2, the bridge is curved with a radius of 300 m. The curvature is a governing factor in the design and cannot be neglected as per the Canadian Highway Bridge Design Code (CHBDC 2006) requirements for curved bridges.

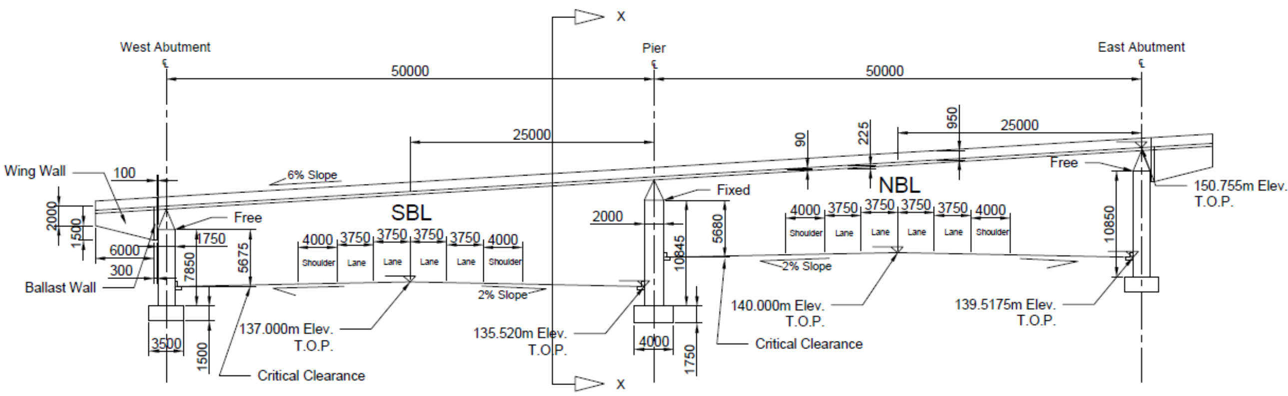


Figure 1: Bridge elevation

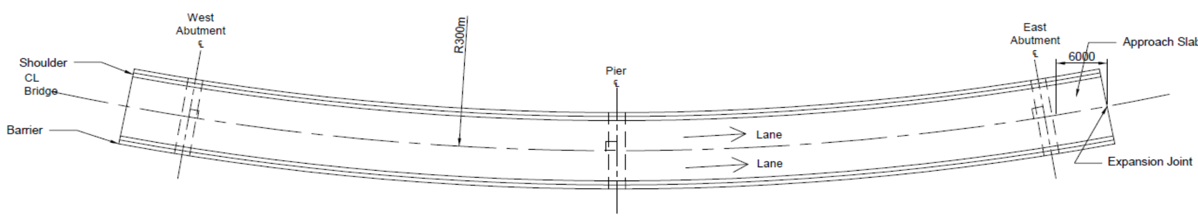
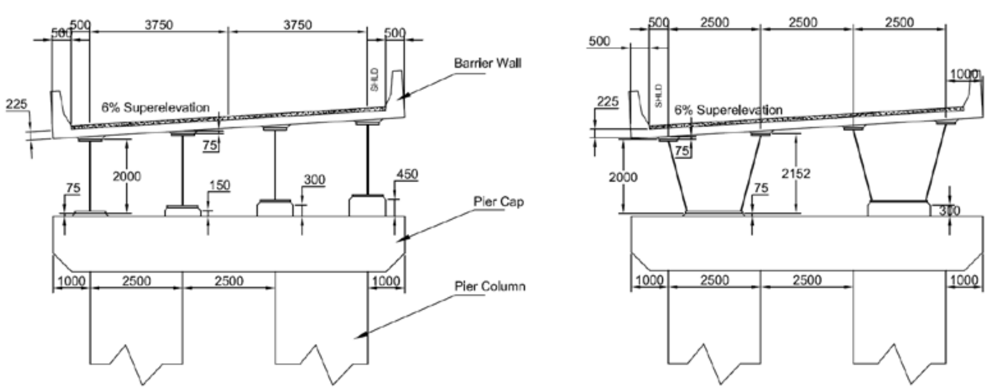


Figure 2: Bridge plan

The bridge cross section is 9.5 m wide and features a reinforced concrete deck on steel girders. Two options were investigated for the girders. "Option A" and "Option B" make use of I-girders and box girders respectively (See Figure 3). The paper herein is intended to compare the two alternatives and find the most suitable one for implementation and optimization.



a) Option "A"; I-girders

b) Option "B"; box-girders

Figure 3: Bridge cross-section and investigated girder options

1.3 Design and Implementation Challenges

Other than the typical design challenges semi-integral bridges face in regards to lateral and longitudinal restraint, there are several factors particular to this project that make it significantly more complex. The clearance height is one main limiting constraint. As shown in Figure 1, a maximum bridge elevation of 150.755 m needs to be maintained while satisfying the minimum vertical clearance required for vehicular traffic flowing underneath the bridge. This limits the steel girder depth to a maximum of 2 m, making it significantly more challenging to satisfy the serviceability requirements and designing the section to resist the applied factored forces, specially the factored positive and negative moments.

Furthermore, due to the curved geometry of the bridge, significant torsional straining actions induced in both individual girders and the entire deck system add even more to the complexity of the analysis and design. During construction, individual members tend to twist about their longitudinal axis if not properly restrained. As a result, each girder needs to be held in place during construction until the entire system acts as one composite unit. Hence, the precarious nature of the construction phase. Therefore, the analysis, design and construction should accommodate the related stability concerns associated with girder erection and deck construction, as well as during the entire service life of the bridge. The emphasis on analyzing the bridge from both a holistic and elementalistic point of view cannot be underestimated. Forensic studies have shown how the torsional effects have led to the collapse of bridges even before installation of the bridge deck, which is why special care was taken in ensuring that the loads can be sustained in every stage of the project (Davidson 2004, AASHTO 2003).

2. MAIN DESIGN CONSIDERATIONS

2.1 Lateral and Longitudinal Restraint

Listed below are several factors that contribute to the lateral and longitudinal restraint of the superstructure, which, as mentioned, is free to move independently from the substructure:

- First, as shown in Figure 1, the ballast wall and wing wall are integral with the superstructure and are subjected to soil pressure/confined by the soil. This confinement is a significant contributor to the lateral and longitudinal restraint of the superstructure. Moreover, special care should be taken when backfilling the soil against the ballast walls. Both sides of the superstructure need to be backfilled simultaneously or else differential horizontal forces can develop from backfilling only one side of the bridge, which can eventually lead to the collapse of the bridge by sliding of the superstructure over the abutments.
- Second, the hinged bearings of the superstructure at the middle pier along with the design of the pier footing as fixed base will help prevent the bridge from sliding or overturning.

2.2 Effect of Curvature

2.2.1 Torsional Forces on Girder

The effect of torsion, for preliminary calculations only, was accounted for by introducing a moment along the cross section of the bridge.

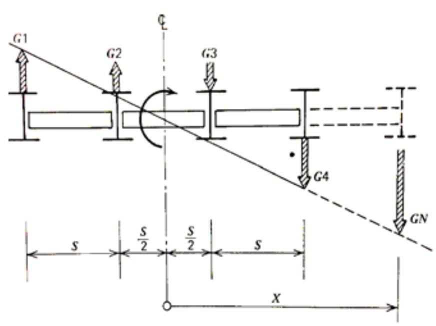


Figure 4: Torsional effect on girders (Heins and Firmage, 1979)

As shown in Figure 4, the moment is resolved into vertical forces on the girders (Heins and Firmage, 1979). The outer girders (right of center line, girders G3 and G4) are subject to added forces and therefore are the critical girders for which the design was conducted. The girders to the inside of the curve center line benefit from the torsion effect as shown in the figure (Girders G1 and G2 are experiencing uplifting due to torsion).

3. PRELIMINARY MODELING AND ANALYSIS

3.1 General

This phase was directed to compare between the proposed two girder alternatives. 2D-FEM was utilized for the structural analysis of the bridge through the software SAP2000. After the most suitable alternative was selected, a detailed analysis was carried out using a 3D-FEM model as will be discussed in details later in the paper.

- The structural loading and design of the bridge are in accordance to the CHBDC 2006
- No seismic activity was considered based on the location of the bridge as per CHBDC guidelines

3.2 Loads and Loading Cases

3.2.1 Dead Load

All the load carrying components, such as the girders and reinforced concrete deck, were considered as dead load. All non-structural components, such as the wearing surface, were considered as superimposed dead load. The dead loads and superimposed dead loads were multiplied by their corresponding load factors obtained from the CHBDC in order to produce the most critical scenario of loading.

3.2.2 Live Load

A standard CL-625-ONT truck was utilized as per the CHBDC guidelines.

3.2.3 Soil Pressure

As mentioned, semi-integral bridges are held in place laterally and longitudinally mainly due to the soil pressure exerted on the ballast wall and wing wall. These forces directly influence the forces/moments present throughout the entire superstructure due to the moment connection to the ballast wall. Hence they were also included as part of the structural analysis of the bridge. The soil load was represented by a triangular distributed load increasing linearly with depth and the magnitude of the forces was determined based on the properties of the soil (angle of internal friction of 35^{o}). The CHBDC also prescribes an added load due to compaction of the soil near the surface.

3.2.4 Load Combinations

Through the use of structural analysis software, the applied factored bending moment and shear force diagrams were produced. These considered multiple combinations of maximum and minimum dead load and live load cases. For preliminary calculations only, load types and appropriate factors corresponded to the ultimate limit state (ULS-1). These combinations included:

Case 1: Maximum loading along both spans; minimum soil pressure
Case 2: Maximum loading on the first span, minimum on the second; maximum soil pressure
Case 3: Maximum loading on the first span, minimum on the second; minimum soil pressure

Soil pressure loads were minimized in Case 1 since its presence lowers the effects of super structure stresses. The bending moment and shear force diagrams corresponding to the outermost girder (that which experiences the greatest loads as per the aforementioned torsional effects) are summarized below.

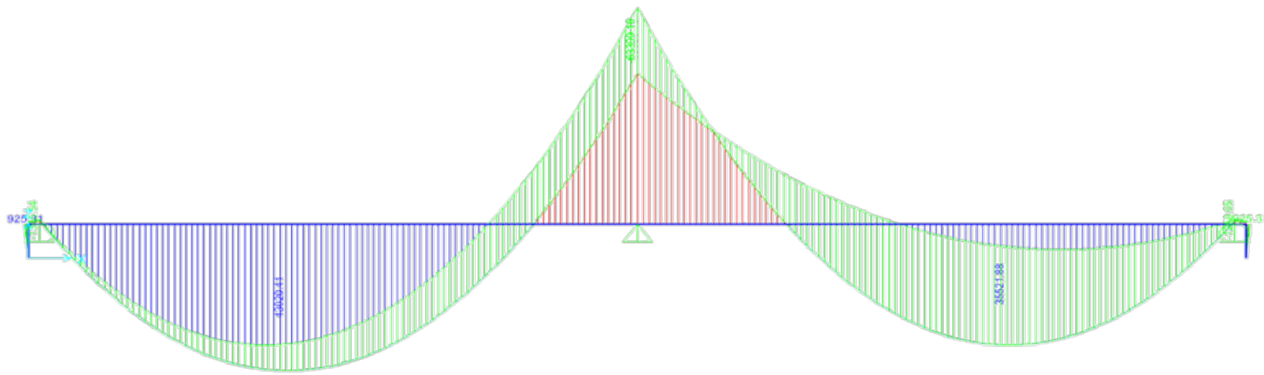


Figure 5: Bending moment envelop (ULS-1) for the exterior girder

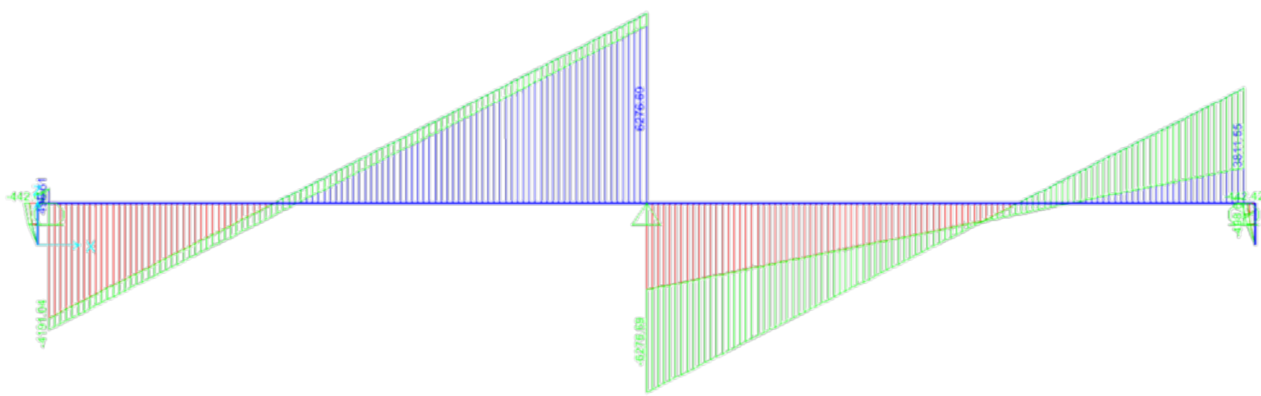


Figure 6: Shear force envelop (ULS-1) for the exterior girder

4. PRELIMINARY DESIGN

4.1 Flexural Design

Assumptions for flexural design include (i) full composite action between the reinforced concrete deck and steel girders; (ii) concrete does not contribute in resisting flexural tension; and (iii) sufficient longitudinal, transverse and bearing stiffeners were provided to prevent local buckling of the elements and lateral torsional buckling of the girder in the negative moment region.

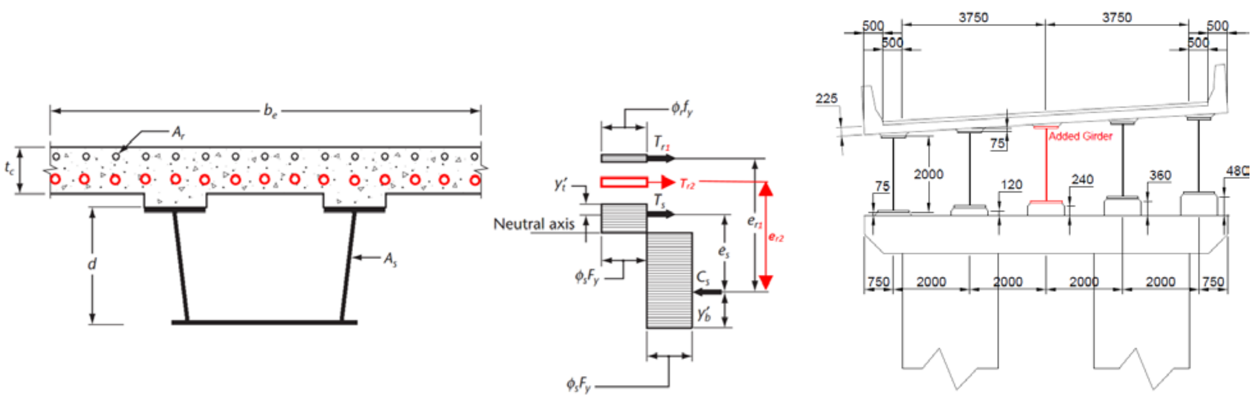
The positive and negative resisting moments of the composite section were determined following the above listed assumptions and the CHBDC guidelines. The results are summarized in Tables 1 and 2. As shown in the tables, under the column called “Original”, neither the I-girder nor the box girder had sufficient flexural strength to resist the negative factored moment. This resulted from the limited girder depth of only 2.0 m that is required to satisfy the minimum vertical clearance for vehicular traffic flowing underneath the bridge. This depth represents a span-to-depth ratio of around 25 which is considered quite challenging to satisfy with conventional steel girder superstructures.

4.1.1 Sensitivity Design Process

In order to determine the most suitable and economical alternative to increase the flexural resistance in the negative moment region using the limited girder depth, a sensitivity design process was performed. Through this analysis, several options were considered to increase the negative moment resistance of the section. At the end of this section a table is included summarizing how each option had an effect on altering the factored applied moments and factored resisting moments.

4.1.1.1 Concrete Slab Reinforcement; Alternative 1

Increasing the amount of reinforcement in the slab appeared to be the most ideal and simple solution to generate a larger moment of resistance in the negative moment region. The top and bottom reinforcement in the slab was increased from 25M to 35M and the spacing reduced from 150 mm to 100 mm. Even with the maximum permissible amount of reinforcement, the moment resistance could not be increased sufficiently to resist the factored negative moment.



a) Alternative 1 b) Alternative 3

Figure 7: Alternatives investigated to improve flexural capacity

4.1.1.2 Variable Girder Depth; Alternative 2

The second alternative studied was the parabolic increase of the girder depth at the pier location. The maximum depth that could be implemented was 2.15 m at the mid-span of the bridge due to the clearance height constraint. However, even though increasing the girder depth did significantly increase the resisting moment, the increase in stiffness at the negative moment region that came from increasing the depth, led to even higher values of factored negative moment at that mid-span location, rendering the option unviable. For this alternative to be feasible the parabolic increase of the depth would have to be significant to allow for a greater increase of the resistance force over the factored force, however as mentioned, the height clearance is a governing factor in the design and therefore this alternative could not be accommodated.

4.1.1.3 Additional Girders; Alternative 3

The third alternative considered was the addition of another girder to the bridge cross section. Thereby reducing the factored moment on each girder while maintaining the same moment of resistance (Refer to Figure 7). However, this method was not viable since the construction and material cost drastically increased with the added premium of having an extra girder. Moreover, the further the girder is placed away from the centerline of the bridge, the greater the effect of the torsional forces on the girders. Hence the alternative was not satisfactory neither from an economical nor structural point of view. Furthermore, this option was not feasible for the Box-girder alternative since three box girder will not fit in a 9.5 m wide deck.

4.1.1.4 Girder Cross-Section Dimensions; Alternative 4

Ultimately, increasing the flange thickness proved to be the most economical way to increase the negative moment of resistance within allowable limits. Ratios on the flange width to thickness were generated to find the optimum dimensions that will generate the greatest resistance utilizing the least amount of material. Naturally, increasing the thickness of the bottom flange had a greater effect on the moment of resistance than increasing the thickness of the top flange, this is due to the fact that the neutral axis (of the plastic section) is closer to the top flange than to the bottom flange because of the composite action between the slab and the girders (which drives the neutral axis up). The new design ensured that positive and negative factored moments were less than the moment of resistance, while optimizing the cost of each girder.

4.1.1.5 Summary of Options

Tables 1 and 2 summarize the impacts of the investigated options in improving the negative bending moment capacity of the girders.

Table 1: Summary of applied and resisting moments (kN·m) for Option “B”; Box-girder

		Original		Alternative 1		Alternative 4	
		Box Girder 1	Box Girder 2	Box Girder 1	Box Girder 2	Box Girder 1	Box Girder 2
Positive Moment (kN·m)	Factored	41143.67	16297.26	41143.67	16297.26	43020.41	25272.42
	Resisting	57991.81	57991.81	56370.90	56370.90	85765.13	85765.13
Negative Moment (kN·m)	Factored	-60207.56	-24000.19	-60207.56	-24000.19	-63399.10	-25272.42
	Resisting	-43752.89	-43752.89	-55911.80	-55911.80	-66872.69	-66872.69

Table 2: Summary of applied and resisting moments (kN·m) for Option “A”; I-girder

		Original		Alternative 1		Alternative 3		Alternative 4	
		I-Girder 1	I-Girder 2	I-Girder 1	I-Girder 2	I-Girder 1	I-Girder 2	I-Girder 1	I-Girder 2
Positive Moment (kN·m)	Factored	22629.86	21497.13	22629.86	21497.13	18103.89	17197.70	23950.82	22751.97
	Resisting	31150.50	31730.29	32066.75	30797.49	29842.81	29672.34	46354.90	47055.93
Negative Moment (kN·m)	Factored	-33440.03	-30679.72	-33440.03	-30679.72	-26752.02	-24543.78	-35683.86	-32738.33
	Resisting	-23164.54	-23534.47	-30198.81	-30698.90	-22000.15	-22000.15	-37611.58	-37991.53

4.2 Shear Design

Assumptions for the shear design are the following (i) only the web contributes in resisting shear forces; and (ii) sufficient stiffeners will be provided to prevent web crippling or local buckling of the web. From the shear force diagrams, shown in Figure 6, the maximum shear forces on the girders, which occur at the support locations, were compared to the factored shear resistance of the girder. The values are summarized in Table 3. As shown, all of the girders meet the shear strength requirements.

Table 3: Summary of applied and resisting shear (kN) for Options “A”; I-girders and “B”; Box-girder

		Original		Alternative 4	Original		Alternative 4
		Box Girder 1	Box Girder 2	Box Girder 1	I-Girder 1	I-Girder 2	I-Girder 1
Positive Shear (kN)	Factored	5961.53	2426.24	6276.69	3296.26	3050.79	3516.92
	Resisting	10328.95	10328.95	10328.95	5164.50	5164.50	5164.50
Negative Shear (kN)	Factored	5961.53	2426.24	6276.69	3296.26	3050.79	3516.92
	Resisting	10328.95	10328.95	10328.95	5164.50	5164.50	5164.50

5. GIRDER OPTION COMPARISON

Once both options proved to be feasible, a number of variables were taken into consideration for the comparison of the two alternatives. Figure 8 shows a breakdown of the cost for each option, which was one of the main factors taken into consideration.

Figure 8: Cost estimate/breakdown

Also, as show in Table 4, each of the variables considered was given a weight percentage. The option with the highest total was chosen for the final design.

Table 4: Comparison of girder options

Determining Factors	Option A (I-Girder)	Option B (Box-Girder)
Performance/Durability (35)	**25 Points** • Performs well under shear and bending but not torsion. • Greater surface area exposed, higher chance of corrosion	**35 Points** • Exceptional in resisting shear, bending and torsion • Lower surface area exposed to weathering
Construction Impact (10)	**5 Points** • Higher risk to the installation crew due to the laborious installation process of the bracing • Simpler connections	**7 Points** • Less likely to fail due to torsion for unrestrained/unshored construction • Faster installation, bracing already installed inside the box itself
Aesthetic Appeal (15)	**5 Points** • Not aesthetically pleasing, due to exposed diagonal braces • Cluttered finish	**15 Points** • Aesthetically pleasing since bracing is hidden inside box • Smooth finish
Inspection Difficulty (5)	**5 Points** • Easy accessibility to all aspects of the girder • Flanges and web clearly visible	**2 Points** • Difficult to inspect as half the girder is enclosed • Must be inspected prior to installation
Overall Cost	**30 Points** $ 12,837,660	**35 Points** $ 11,437,188
	70 Points	**94 Points**

6. DETAILED ANALYSIS OF SELECTED ALTERNATIVE

6.1 General

For the detailed design phase, a 3D-FEM analysis utilizing “CSiBridge” was conducted for the box girder option, which was the one selected for implementation based on the above mentioned criteria. Shown in Figure 9 is the 3D isometric view of the bridge.

6.2 Detailed Analysis and Design

The shear, bending moment and deformations diagrams for all the load cases/combinations for the dead load, live load, temperature load, wind load, braking load and centrifugal loads imposed on the bridge were determined using the 3D-FEM analysis. Figure 10 shows an example of the output results obtained from this analysis.

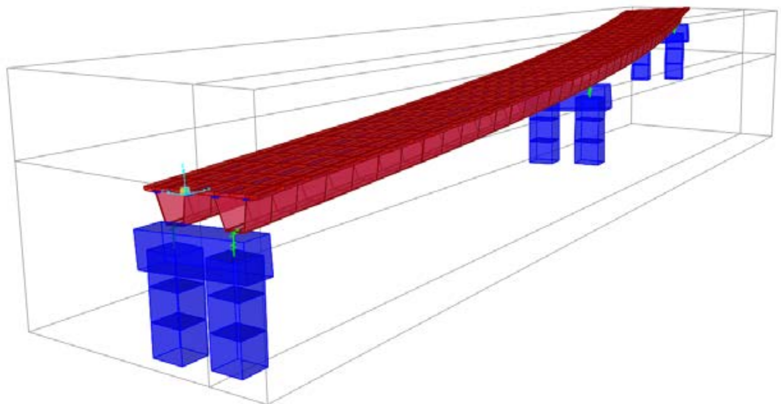


Figure 9: View of the 3D-FEM of the selected alternative: Option B; Box Girder

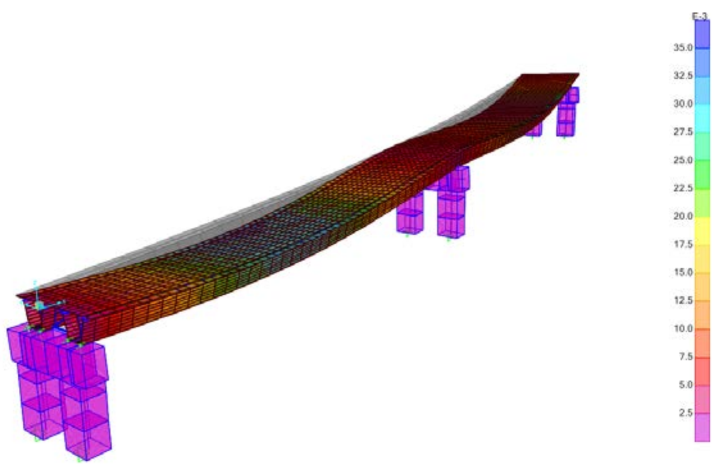


Figure 10: view from the 3D-FEM of the deflected shape for ULS-3 combination (cm)

The factored applied forces obtained from the 3D-FEM analysis were compared to the factored resistance of the section and the final design was optimized to meet the requirements in the most economical way utilizing adjustment for flange thickness as previously evaluated. See figure below for final dimensions of the box girder section.

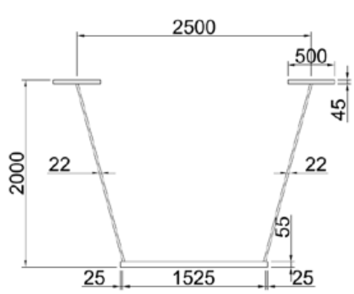


Figure 11: Final girder cross-section dimensions (mm)

7. CONCLUSION

The subject project involves the analysis and design of a two span semi-integral abutment bridge with continuous curved steel girders. The bridge will be located north of the GTA and will be a portion of the development that will see the connection of a major road way with a 400 series highway. Preliminary design discussions included two girder options, the I-girder and the box-girder. In order to come to a verdict on the design to use going forward into the detailed design process, many factors were taken into consideration. Once both alternatives proved to be feasible, the two options were compared and only one was chosen for 3D-FEM and optimization.

Listed below are the major aspects that support the selection of the bridge type and the corresponding options:

- Semi-Integral bridges provide an extremely durable alternative since they shift the expansion joints outside the superstructure itself. Thereby, protecting vulnerable bearing components that are costly to repair and maintain and get easily damaged from the deleterious effects of deck drainage. Semi-integral bridges feature several other advantages and their use is expected to burgeon further in upcoming years;
- The minimum vertical clearance required for the vehicular traffic flowing underneath a bridge may limit the girder depth and consequently, govern the design of the bridge girders. For the subject project, a sensitivity analysis/design phase was dedicated to investigate the optimum design option to satisfy such requirement. A refined optimization of the girder cross section dimensions, including the thicknesses of the girder flanges, proved to be the most suitable solution from both design and construction aspects; and
- In regards to girder selection, the box girder option enables for reduced construction costs due to the fact that it has a much greater torsional stiffness when compared to the I-girder and therefore it's more stable not only when the entire deck acts as one composite unit but also during the erection and staged construction phase, which is crucial in curved bridges. Further, the box girders possess more favourable aesthetic features than those of the I-girders and is less exposed to weathering and deterioration.

References

AASHTO. 2003. Guide Specifications for Horizontally Curved Steel Girder Highway Bridges, American Association of State Highway and Transportation Officials, Washington, DC, USA

Burke Jr. M.P. 2009. *Integral and Semi-Integral Bridges*, Wiley-Blackwell A John Wiley & Sons, Ltd., Publication, Chichester, West Sussex, United Kingdom.

CHBDC (2006). CAN/CSA–S6; Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, ON, Canada.

Davidson, J.S., Abdalla, R.S. and Madhavan, M. (2004). Stability of Curved Bridges During Construction, *UTCA Report 03228.*

Heins C.P and Firmage D.A. 1979. *Design Of Modern Steel Highway Bridges*, John Wiley & Sons, Chichester, United Kingdom.

Topkaya, C. (2003). Behavior of Curved Steel Trapezoidal Box girders during Construction, *Engineering Structures*, Vol. 26, 721-733.