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ALTERNATIVE DESIGNS OF MULTI-SPAN CONTINUOUS CONCRETE GIRDER BRIDGES WITH SEMI-INTEGRAL ABUTMENTS

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Abstract: In recent years, it has become more and more evident that transportation infrastructure in the Greater Toronto Area (GTA) has not expanded cohesively with the growth of populations outside of the downtown core. The capacity of existing roadways has been reached and the need for new infrastructure to alleviate congestion is immediate. This paper presents a summary of alternative designs of a multi-span continuous CPCI semi-integral abutment bridge that would allow crossing of major roadways in developed areas for highway extensions. One benefit of the semi-integral abutment bridge is the shielding of major bridge components (e.g. bearings) against water leaks by the offset of conventional expansion joints to the outside of the structure envelope, decreasing the potential for significant future maintenance cost. This bridge type also allows for longitudinal movements through sliding bearings relieving the bridge structure from significant straining actions because of thermal and creep/shrinkage effects. To develop a durable, economical design, five alternative deck structures were evaluated; 7, Prestressed Concrete I-Girder, 7, Prestressed Concrete Box-Girder and two sizes of the recently developed NU I-Girder, two 7 girder alternatives and one 6 girder alternative. NU I-Girders are advantageous because they can accommodate longer span lengths while maintaining, or reducing, bridge depth. Investigation of the NU I-Girder Structural analysis was performed through finite element modelling and design was accomplished using the Canadian Highway Bridge Design Code, CHBDC. The optimal girder option was selected based on a comprehensive comparison criterion that considered cost and durability among various other comparison aspects.

1 INTRODUCTION

1.1 Background

Semi-integral abutment bridges were first introduced in Ontario in the 1960's. The first use was to replace an existing rigid frame bridge to allow widening of a roadway beneath it (Husain and Bagnariol, 1999).

A semi-integral abutment bridge can be single or multi-span and include a ballast wall which is a vertically bent extension of the continuous deck that covers the abutments (Burke, 2009). Expansion joints, normally found between the ballast wall/abutment and deck of a conventional bridge are now moved outside of the bridge superstructure envelope (Figure 1). This is the main benefit of a semi-integral abutment bridge as it largely decreases the possibility of corrosion of major bridge components such as girders and bearings (Burke, 2009). Protection of these components decreases the possibility of significant future maintenance cost and lack of structural integrity.

This bridge type differs from an integral abutment bridge as the exterior supports are not rigid; straining actions at the exterior support locations are minimized. Sliding bearings are used to allow longitudinal translation of the deck to absorb the effects of traffic loading and expansion and contraction due to thermal effects (Husain and Bagnariol, 1999).

Semi-integral abutment bridges have proven to be economical and durable. They have shown to have reliable performance and are generally simpler to construct. Where fully integral abutments are not suitable, semi-integral abutment bridges have become more in demand (Burke, 2009). Figures 2 and 3 show examples of semi-integral abutment bridges in Ontario, Canada.

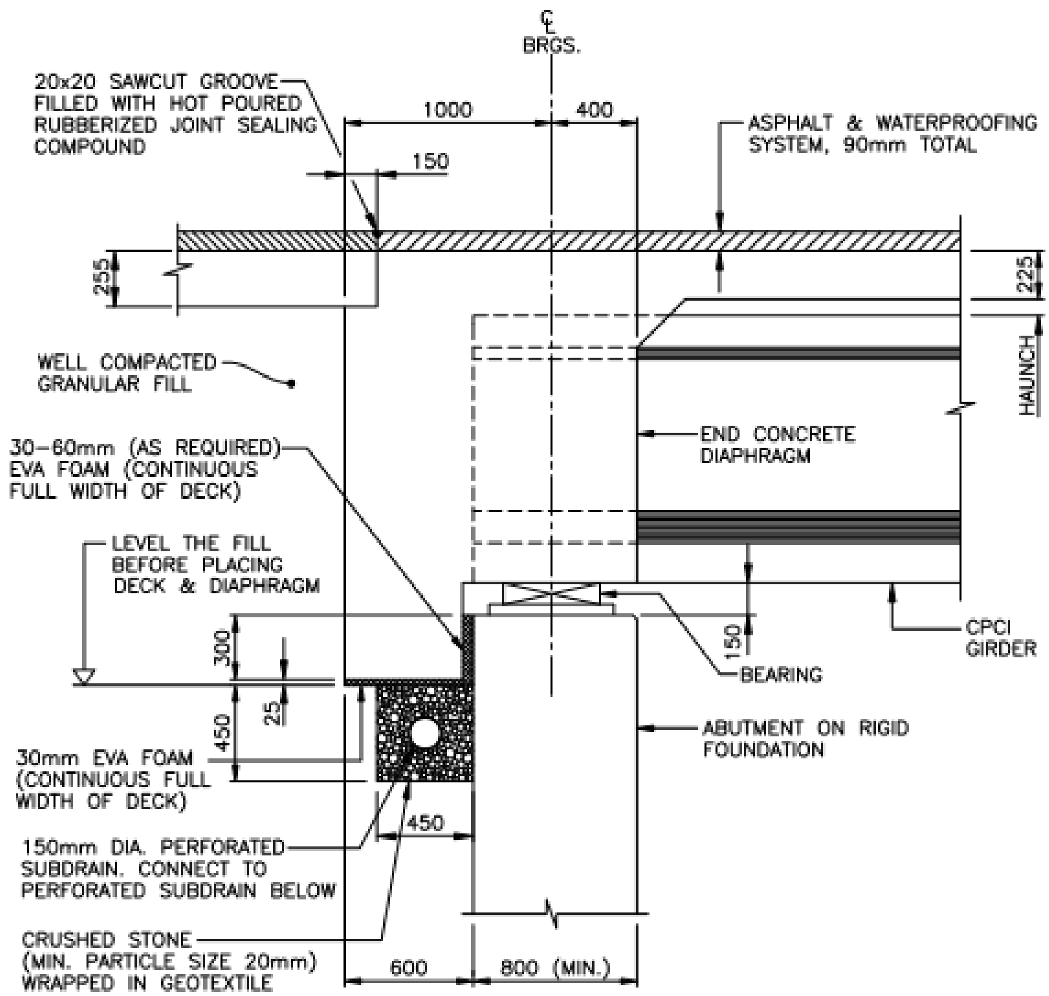


Figure 1: Semi-integral abutment end diaphragm detail (Husain and Bagnariol, 1999)

Figure 2: CPCI I-Girder semi-integral abutment bridge

Figure 3: End diaphragm of completed bridge

1.2 Problem Definition

In recent years, it has become evident that transportation infrastructure in the Greater Toronto Area (GTA) has not expanded cohesively with the growth of populations outside of the downtown core. The capacity of existing arterial roadways has been reached and the need for new infrastructure to alleviate congestion is immediate. Residential and commercial growth in areas north of the GTA has created congestion on the Don Valley Parkway (DVP), a major travel route connecting the northern and southern parts of Toronto, requiring an extension of Highway 404. The task at hand was to develop a multi-span continuous CPCI semi-integral abutment bridge design that will allow crossing of major roadways in developed areas for the highway extension.

1.3 Description of Bridge

Located in the north part of GTA, Ontario, near the town of Newmarket, this bridge is composed of two spans, both 23.5m in length, with a grade of 2.5% downward from North to South abutment (Figure 4). The bridge deck width is a total of 15m; 7.5m from bridge centerline to the centreline of the outermost girder (Figures 5 and 6). The bridge deck has a cross fall of 2% from bridge centreline downward towards the exterior barrier walls. A major roadway exists beneath the proposed bridge structure. The grade of the existing roadway is 1.75% downward from east to west. A minimum vertical clearance of 5.5m is required. All computations performed throughout the preliminary design of the multi-span continuous CPCI semi-integral abutment bridge were in accordance with the Canadian Highway Bridge Design Code, CHBDC (CSA, 2014).

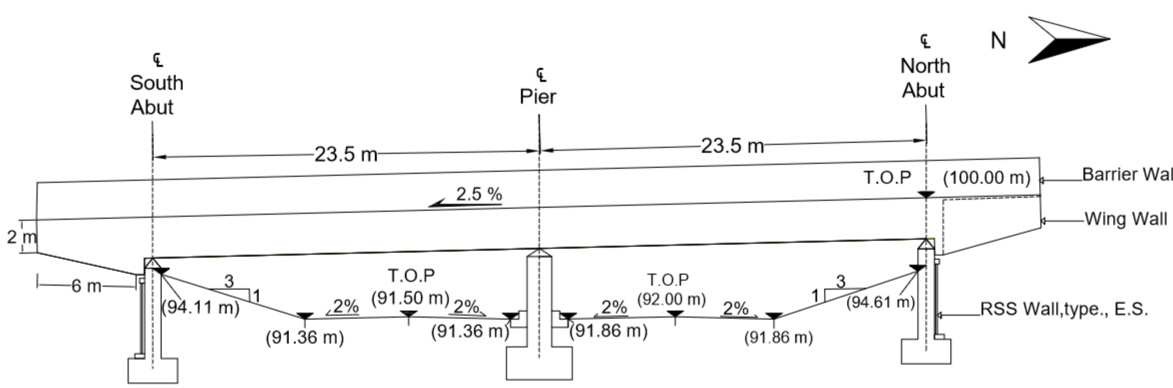


Figure 4: Bridge elevation

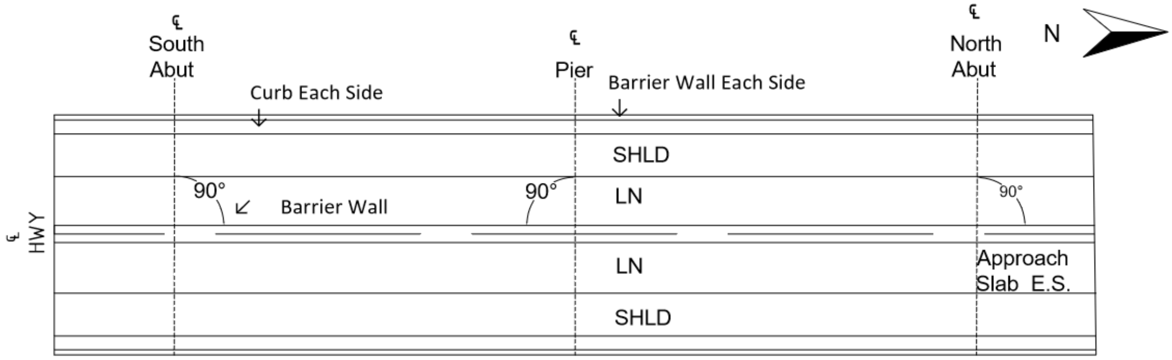
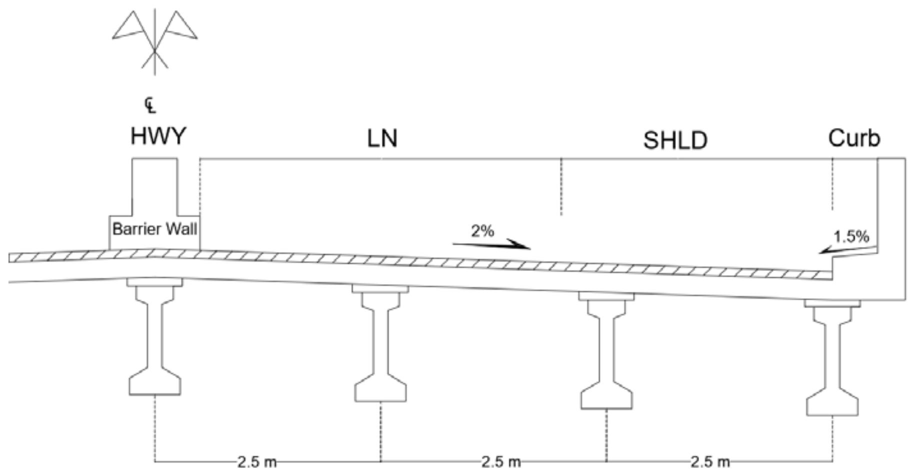
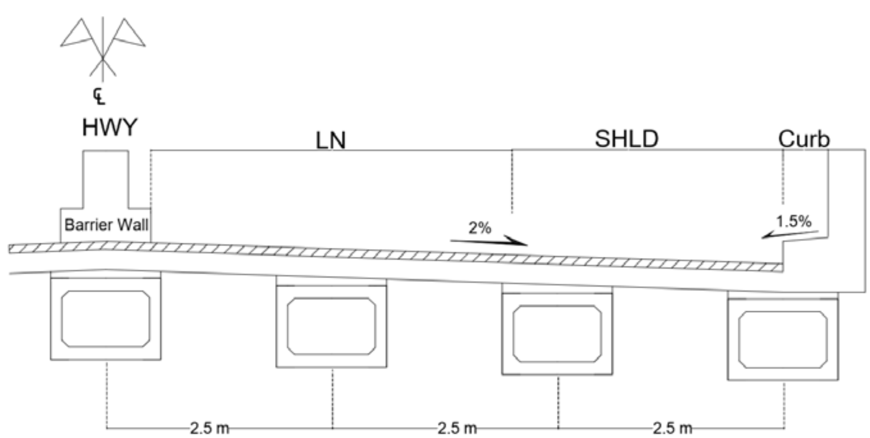


Figure 5: Bridge plan



a) CPCI/NU I-Girder alternative



b) CPCI Box Girder alternative

Figure 6: Cross sections of bridge alternatives; (a) CPCI/NU I-Girder, (b) CPCI Box Girder

Three girder alternatives were evaluated to determine the optimal design both structurally and economically; CPCI 1200 I-Girder; CPCI B-900 Box Girder; 1200 NU I-Girder.

1.4 Design Considerations

Three main constraints governed the selected design of the proposed semi-integral abutment bridge. Firstly, a minimum vertical clearance of 5.5m was required for the bridge design as mentioned above. This sets a limitation on depth of girder alternatives based on elevation of existing roads. Secondly, the Canadian Highway Bridge Design Code set limitations and standards for specific design to the province of Ontario. For example, truck loading analysis was performed using the CL-625 Ontario truck as specified in section 3 (CSA, 2014).

Lastly, budget constraints were considered in design. The requirement of sufficient structural capacity was the main concern. Once it was determined that a given girder alternative performed adequately under the given loads, selection became based mostly on cost. In the cases where girders performed exceedingly well under the specified loading, the possibility of reducing girder size to save cost was explored.

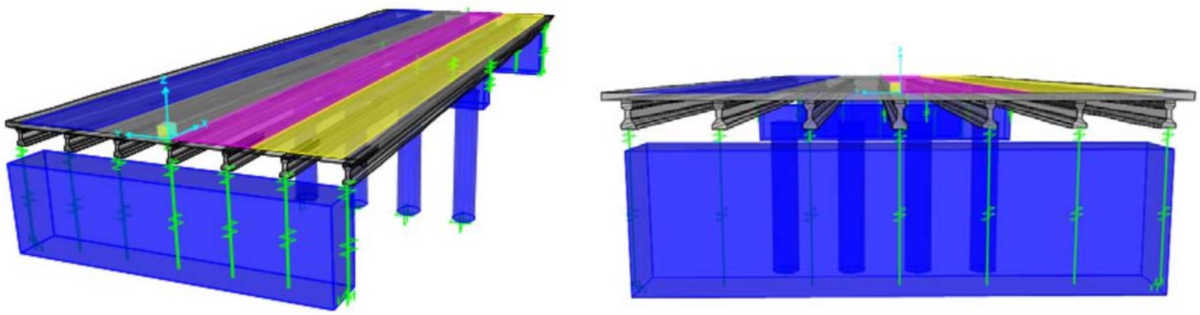


Figure 7: Rendered view of bridge

2 PRELIMINARY MODELING AND ANALYSIS

2.1 General

The preliminary analysis stage of design was carried out to evaluate the structural capacity and response of each girder alternative. SAP2000 software (SAP2000, 2018) was used to determine the maximum values of factored design moment and shear. Various loading cases/combinations were evaluated to ensure that the maximum possible response values were obtained. In addition, interior and exterior girder cases were evaluated individually as the response in each case was expected to be different. These results were used continuously throughout the design process to perform checks that would provide a safe bridge design. Analysis was performed following set guidelines and standards provided in the CHBDC 2014. Seismic activity was not considered in design as the suggested bridge location is not within an active seismic zone (CHBDC, 2014).

2.2 Dead and Live Loading and Loading Combinations

2.2.1 Dead Loads and Superimposed Dead Loads

Elements of the bridge superstructure included as dead loads were the ballast walls, wing walls (carried by ballast walls/superstructure), girders, deck slab, and haunch self-weight. The superimposed dead loads used to evaluate loading response included all non-structural bridge components such as the wearing surface (asphalt/waterproofing), and barrier walls. All dead and superimposed dead loads were divided evenly over 7 girders for preliminary analysis. Based on this, the only changing dead load for each alternative was the girder self-weight.

2.2.2 Soil Pressure – Backfill Loads and Compaction Surcharge

The backfill load and compaction surcharge were considered in structural analysis of the bridge. The intensity of the backfill load is increased by the depth. Hence, the backfill load was represented by a triangular distributed load. The magnitude of the backfill pressure was determined based on the angle of the internal friction of the soil given in the geo-technical report. According to the CHDBC, the compaction surcharge pressure affects the bending moment of the superstructure.

2.2.3 Live Load

Based on the location of the bridge being designed, the standard CL-625-ONT truck was used along with a dynamic load allowance of 0.25 as described in the CHBDC (CHBDC, 2014). The span lengths of 23.5m allowed for all five truck axles to be applied on the bridge. Different live load response was expected for the interior and exterior girders. To measure this difference, as prescribed in the CHBDC, truck load fractions (F_T) were considered.

2.2.4 Load Combinations

To ensure that maximum superstructure response would be returned from the SAP2000 software, each individual dead load was factored using the appropriate ultimate limit state according to CHBDC requirements. Presented below are the bending moment and shear force diagrams returned from analysis of the 1200 NU I-Girder alternative, for the interior girder case, at ULS-1.

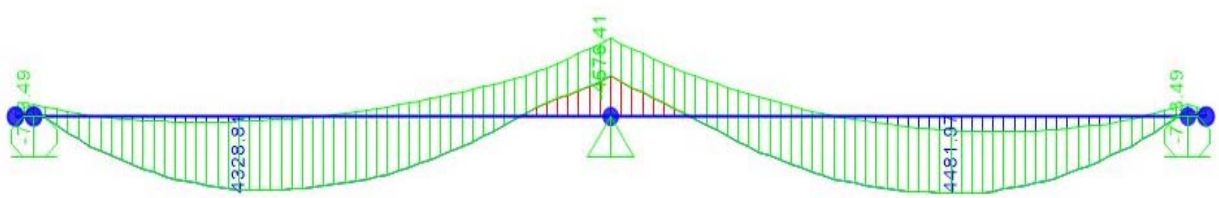


Figure 8: Bending moment envelope for 1200 NU I-Girder, interior girder case, at ULS-1 (kN·m)

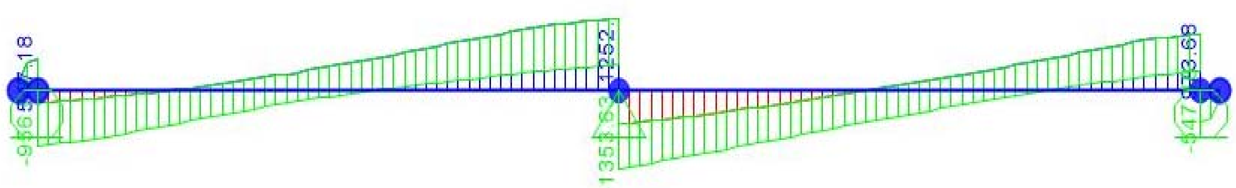


Figure 9: Shear envelope for 1200 NU I-Girder, interior girder case, at ULS-1 (kN)

3 PRELIMINARY DESIGN

To determine the most suitable girder alternative for design, critical design checks were performed. All girders proved to be suitable for design, especially the NU I-Girder alternative. Analysis showed that the NU I-Girders had much higher structural capacity so further alternatives were explored such as, the use of 7, 900 NU I-Girders instead of 7, CPCI I-Girders and the use of 6 instead of 7, 900 NU I-Girders.

3.1 Flexural Design

Using factored design moments, the resisting moments and the corresponding required prestressing strands for the girder, and rebars for the deck slab, were determined. In all cases, positive interior/exterior and negative interior/exterior, resisting moments exceeded design moments and the girders were deemed to be adequate for design. In all cases, the resisting moment was at least 20% greater than the cracking moment.

3.2 Shear Design

The maximum contribution of concrete and reinforcement resisting shear was checked against the maximum factored design shear force for each alternative. It was assumed that prestressing reinforcements did not contribute to shear resistance at this stage. All calculated shear resistances, V_r, were compared to their corresponding factored shear force, V_f; in all cases the design was determined to be safe, $V_r \geq V_f$.

3.3 Further Investigated Alternatives

In the preceding preliminary design checks and calculations, it was discovered that the 1200 NU I-Girder alternative produced much higher resisting moments in flexural design and shear resistance in shear design. Also, NU I-Girders are expected to have much higher negative moment capacities due to their wider bottom flange that allows an increased number of prestressing strands (Morcous and Akhnoukh, 2007); analysis results proved this to be true. Because of this, two further NU I-Girder alternatives were explored.

3.3.1 7, 900 NU I-Girder; Investigated Alternative 1

Analysis of the 1200 NU I-Girder alternative clearly identified that this type of girder provides an advantage in terms of structural capability. This alternative seemed to be the best however, upon investigation of girder cost (more details are discussed in section 4.1) it was found that the cost heavily outweighed the structural benefit; the CPCI I-Girder and Box-Girder alternatives could meet the same structural requirements at a much cheaper cost. It was then thought to decrease the NU I-Girder size, to decrease cost, and perform the preliminary design analysis again. At this stage, one change in the design was made; the girder self-weight decreased (Design Aid, 2010) which in turn decreased the dead loads applied to the bridge. Flexural design and shear design requirements were met for this alternative.

3.3.2 6, 900 NU I-Girder; Investigated Alternative 2

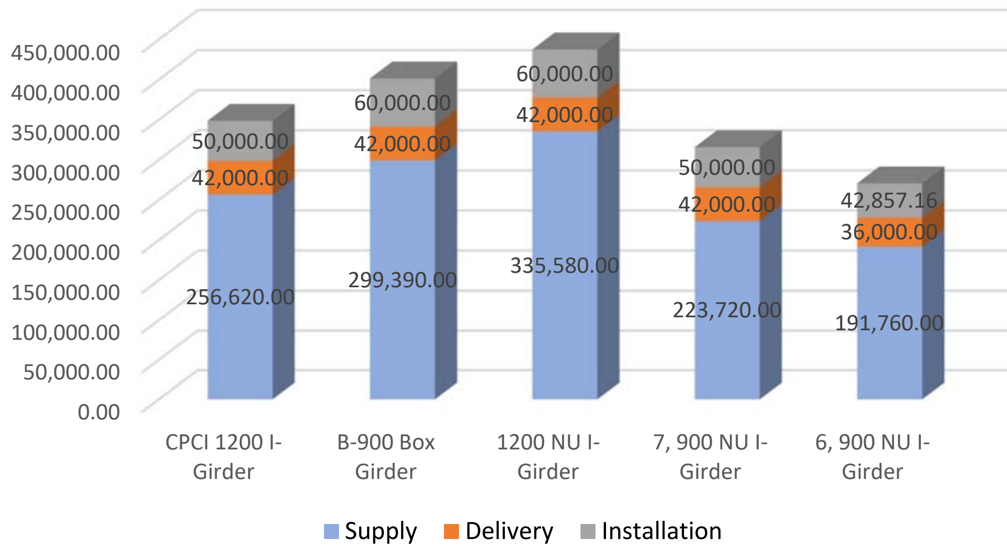
In identifying that the reduced NU I-Girder size was still capable of carrying the dead and live loads applied to the bridge a fifth alternative, second investigated alternative, was considered. This alternative was to decrease the number of girders and maintain the 900 NU I-Girder size. Two important changes were made prior to performing analysis for this option; dead and superimposed dead loads were divided amongst 6, rather than 7, girders; the deck slab thickness was increased by 50mm to provide adequate slab capacity for both shear and moment. Factored design moments and shear per girder increased significantly in this alternative (when compared to the 7, 900 NU I-Girder option) however, calculated negative and positive interior/exterior resisting moment and shear still proved to be sufficient.

4 COMPARISON OF DESIGN ALTERNATIVES

4.1 Direct Cost Summary

A summary of the direct costs associated with each investigated girder alternative has been provided in Figure 10. This is the most significant factor in choosing the favoured design alternative as all projects are subjected to budgets that restrict and dictate design.

Figure 10: Direct cost comparison of investigated alternatives



4.2 Qualitative and Quantitative Comparison of Design Alternatives

In addition to direct cost, five other factors were considered in deciding the recommended girder alternative; performance/durability; aesthetics; constructability; inspect-ability; and environmental impact. To evaluate each of these factors, a point system was used in which each factor was assigned a weight out of 100. A brief explanation for the points given to each alternative has been provided following Table 1.

Table 1: Comparison of quantitative and qualitative aspects of girder alternatives

	CPCI 1200 I-Girder	CPCI B-900 Box Girder	1200 NU I-Girder	900 NU I-Girder*	900 NU I-Girder**
Direct Cost (35)	27 Points $348,620.00	24 Points $401,390.00	21 Points $457,580.00	30 Points $315,720.00	35 Points $270,617.16
Performance/Durability (35)	35 Points	25 Points	35 Points	35 Points	35 Points
Aesthetics (10)	3 Points	10 Points	5 Points	5 Points	5 Points
Constructability (10)	5 Points	5 Points	3 Points	3 Points	5 Points
Inspect-ability (5)	5 Points	2 Points	5 Points	5 Points	5 Points
Environmental Impact (5)	5 Points	2 Points	5 Points	5 Points	5 Points
Total Points	80 Points	68 Points	74 Points	83 Points	90 Points

*900 NU I-Girder alternative with 7 girders.
**900 NU I-Girder alternative with 6 girders.

4.2.1 Performance/Durability

Durability impacts sustainability, therefore it is critical that the girder selected will give the longest service life, without compromising cost. The primary cause of bridge deterioration affecting durability is corrosion. In the case of the CPCI 1200 I-Girder and the 1200/900 NU I-Girders, the open geometry allows water to runoff exterior girder faces making it less likely to corrode because of accumulation of standing water and salt. The NU I-Girders have a higher ranking than the CPCI I-Girder as they have higher resisting moments and shears than other alternatives. The void in the CPCI B-900 Box Girder is typically filled with a cardboard material; when water enters the void, it soaks the cardboard, promoting corrosion due to standing water. This makes the Box Girder the least favourable for performance/durability.

4.2.2 Aesthetic Appeal

The bridge location and purpose suggest that it is likely to be frequently travelled. This makes aesthetics another factor to be considered in girder selection. In general, Box-Girders are more aesthetically pleasing than I-Girders due to their closed geometry and uniform symmetry. While this is the case for the average CPCI I-Girder with a small bottom flange width, the NU I-Girder's large bottom flange width makes the spacing between girders look slightly, but not entirely, similar to that of a Box-Girder from beneath the bridge. The scoring for aesthetic appeal reflected this.

4.2.3 Construction Impacts

The impact of construction on travel times/flow of vehicular traffic in areas surrounding the bridge plan site is significant from the cost aspect. The girder type does not have a large impact on the time of installation however, a girder with a larger flange width may be more difficult to install. Centering the flange over the bearing plate and alignment may require higher acuracy and time. For this reason, box girders are viewed as the better alternative in this respect.

4.2.4 Ease of Inspection

The ability for a girder to be inspected plays a significant role in the life cycle cost as difficulty can incur higher inspection costs and have an impact on maintenance/repair schedules. Open geometry of the CPCI I-Girder and NU I-Girder alternatives allow for easy inspection. Box Girders' closed geometry makes maintenance more difficult/costly which results in decreased frequency of inspection.

4.2.5 Environmental Impact

Recyclability plays a major role in sustainability and evaluating the environmental impacts of each girder alternative. The recyclability of each alternative is directly related to the amount of material that can be re-used at the end of the alternative service life. For example, the recycable steel depends on the amount that has corroded over time. As box girders are more likely to corrode from standing water it is more likely that more material would be salvagable in the case of CPCI/NU I-Girders at the end of their service life.

5 CONCLUSION

The subject project involves the analysis and design of a multi-span continuous semi-integral abutment bridge with four girder alternatives; CPCI 1200 I-Girder, CPCI B-900 Box Girder, 1200 NU I-Girder and 900 NU I-Girder. The bridge is to be built in the GTA to support increased traffic demands resulting from increased residential and commercial development North of the GTA. Each alternative was checked to satisfy the minimum vertical clearance for the traffic underneath the bridge, the superstructure compliance with the CHBDC requirements and the adequacy of the substructure/foundations system.

A detailed comparison was conducted for the four girder options to evaluate each alternative from different aspects including direct costs, performance/durability, aesthetics, constructability, inspect-ability, and environmental impact. After careful consideration and weighing of all criteria, the 6, 900 NU I-Girder alternative was determined to be the best.

The most significant factors leading to this decision were:

- The direct cost of the 900 NU I-Girder was found to be lower than all other girder alternatives. In the end, this made the 6, 900 NU I-Girder option the cheapest. This was the most significant selection criteria.
- Although all other girder alternatives meet the vertical clearance requirement, of minimum 5.5m, set for the bridge design the decreased girder depth of the 900 NU I-Girder allows for an even higher vertical clearance. Aesthetically, a decreased bridge depth is beneficial. Also, decreased

girder depth allows for the potential of increased depth of other bridge superstructure components should this be necessary in terms of structural capacity.

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