Ontario

Ministry of Transportation Ontario
Transportation Infrastructure Management Division
Standards and Contracts Branch
Structures Office

MTO Prestressed Concrete Girder Guidelines (DRAFT)

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Technical Report Documentation

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Abstract	The intent of these guidelines is to provide guidance to designers on prestressed concrete girder selection and design.
Key Words	NU girders; precast concrete; precast girders; girder selection
Distribution	Unrestricted technical audience.

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Published without prejudice as to the application of the findings.

Table of Contents

List of Figures

List of Tables

Appendices

1. Executive Summary

These guidelines were developed to create some guidance to bridge designers that is specific to bridge construction practices in Ontario using precast concrete girders. Precast solid slabs, hollow core slabs and box girders are used in Ontario successfully for shorter span bridges. The Nebraska University (NU) girder bridges began to be used in Ontario about a decade ago, and standard sections and drawings were developed in 2014. Guidelines existed from other jurisdictions, however they either had somewhat different girder shapes, or different practices in their industries which were not applicable to Ontario. Most notably were the differences in cement composition in various parts of North America that allowed higher concrete strengths to be used that were not applicable to Ontario.

MTO have been relying on Canadian industry design and fabrication practices and there was a need to create guidelines in line with Ontario practices and industry resources. The objective of this document, therefore, is to provide single source of information covering design, fabrication, and construction of Precast Concrete Girders in Ontario.

As these guidelines become dated, some of the values in this document may also become superseded by one of the aforementioned documents.

2. Introduction

The information for precast concrete girders contained in this document are supplementary and designers are required to perform the design in accordance with CHBDC and Exceptions to CHBDC contained in MTO's Structural Manual- Division 1 and other design requirements of MTO's Structural Manual including any Policy Memoranda that may exist.

2.1. Objectives

The objectives of these guidelines are to:

1) Encourage consistency in selection and design of precast concrete girders by providing design tables for efficient design,
2) Achieve constructible and cost-effective designs within limits established jointly with industry,
3) Improve the quality of precast concrete by starting with appropriate design decisions
4) Meet Operations Design Engineering Initiative (ODEI EN-11) recommendation of providing additional standardization for precast construction.

2.2. History of Precast Concrete Girders in Ontario

Prestressed concrete was first used in structural applications by French engineer Eugène Freyssinet in the early twentieth century. After learning about precompression of concrete using jacks, and the effects of creep and the need for high strength steel to overcome its effects, he designed the first pretensioned bridge (Iron Gate Dam Bridge) in 1936 and the first post-tensioned bridge (Luzancy Bridge) in 1941. It was not until the concepts of prestressing were explained and taught to engineers by Belgian professor Gustave Magnel that the technology would spread. The first prestressed bridge in North America was the Walnut Lane Bridge (1950) and the first in Canada was the Mosquito Creek Bridge (1952) in British Columbia. The first prestressed bridges in Ontario were the Cull Drain in Sarnia, Sussex Drive Bridges in Ottawa, Jock River bridge in Ottawa, and Crediton Bridge in Huron County, all from about 1955 and all using post-tensioning and precast girders. In 1957, Sydenham River Bridge and Dixon Road Underpass became the Department of Highways Ontario's (DHO) first post-tensioned bridges.

For the early uses, post-tensioning was used because hydraulic jacks reacting against the already hardened concrete was an easy and inexpensive means of imparting the prestessing force into the girder. It was not until adequate confidence in prestressing had been obtained in North America that the first pre-tensioned girders, and the necessary prestressing beds, were developed. In 1961, the first AASHTO girder standard shapes were used in Ontario for ONR Overhead at Earlton, Waverly Road Underpass and Holt Road Underpass. In 1963, the first side-by-side box girder bridges were built on highway 401 between Jane Street and Bathurst. These prestressed box girders did not see widespread use and the AASHTO girders dominated the Ontario landscape into the 1970's. CPCI girders began to be used in the early 1970's and would become the prominent girder by the late 1970's and early 1980's. The CPCI shape was slightly more efficient than the AASHTO shape. All these girders typically had concrete of 5000 psi (34.5 MPa) strength, and 4000 psi (27.6 MPa) at transfer.

From the 1960's to the 1980's, post-tensioned cast-in-place decks were common for longer spans but by the mid-1990's, CPCI girders came into favour and were used for exceedingly long spans, including spliced pre and post-tensioned CPCI 2300 girders. In the 2010's, the Nebraska University (NU) girders were first used on the Rt. Honourable Herb Grey Parkway. These girder shapes were designed to take advantage of higher strength concrete that had become available and had the ability to carry more prestressing strands. In 2014, MTO created standard NU shapes for Ontario, and in 2019, under Policy Memo BO-2019-03, MTO would eliminate the CPCI girder and exclusively use the NU girder. The progression of precast, prestessed I girder shapes is shown in Figure 1.

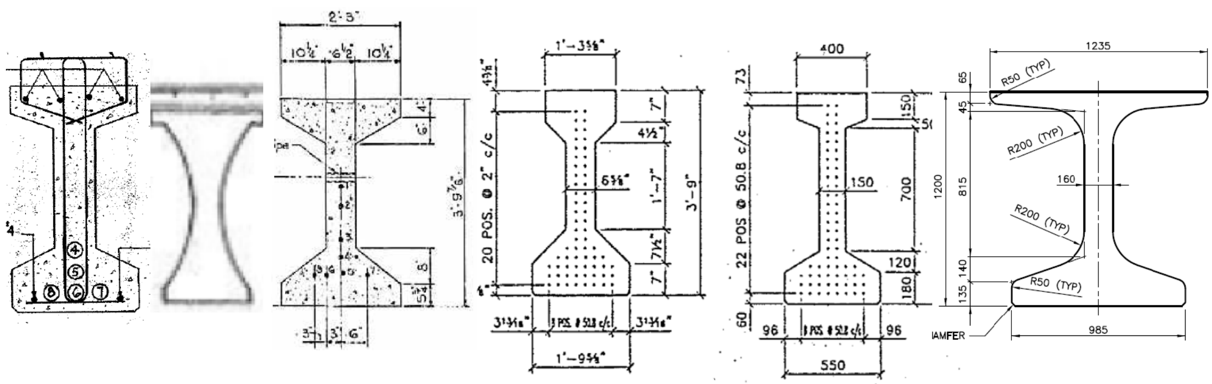


Figure 1 – Prestressed I Girder History in Ontario

2.3. Literature Review / References

Most guidance for designers came from supplier brochures which provided standard shapes and applicable span ranges. Pre-Con produced a Precast Concrete Bridge Design Manual and later, in 2004, a Precast Prestressed Bridge Components Technical Manual. The PCI Design Handbook, first published in 1978 and regularly updated, has also been used to guide designers in prestressed concrete.

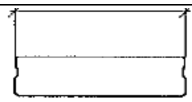
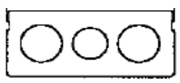
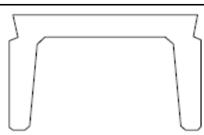
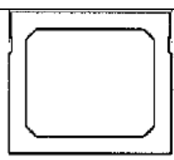
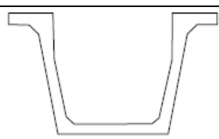
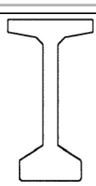
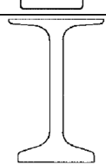
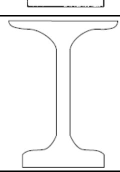
MTO's last major effort to identify design and mostly construction practices around prestressed concrete girders were published in the Prestressed Concrete Manual for Quality Assurance of Bridge Structures during Construction (Mihaljevic et al. 1998). These guidelines described the shop drawing and submission requirements at the time, along with fabrication practices and erection methods.

Alberta was the pioneer in the adaptation of NU girders in Canada. In 2018, the Alberta Infrastructure and Transportation released the NU Girder Design and Detailing Manual, with separate volumes for the manual and for design examples. These documents capture the Alberta experiences and current practices.

3. Girder Types and Applications

Several prestressed girder shapes have been developed, as shown in Table 1, although Ontario has generally only selected a few of these shapes to improve standardization and restrict the selections to the ones were MTO has had positive experiences.

Table 1 - Standard Prestressed Girder Shapes

Girder Type		Typical Depths (mm)	Typical Span Range (m)	Comment
	Solid Slab Girders (Planks)	300 to 500	5 to 16	Limited use in past. MTO considering for future.
	Hollow Slab Girders	400 to 600	9 to 17	Shall not be used on MTO projects due to concerns with cover & minimal weight savings from voids.
	Channel Girders (Inverted U)	700 to 1100	12 to 24	Has rarely been used in Ontario. Span range covered by other types.
	Box Girders	700 to 1200	16 to 36	Regularly used in Ontario.
	Trapezoidal Girders	1600 to 2200	34 to 44	Has not been used recently in Ontario. Span range covered by I-girders.
	CPCI Girders	900 to 2300	12 to 40	Typical girder for past 40 years. No longer in use.
	Bulb T Girders	1000 to 1800	16 to 42	Never adopted in Ontario
	NU Girders	900 to 2400	25 to 50	Current girder of choice in Ontario.

MTO abandoned the use of CPCI girders for new construction. Precast NU girders are used for the medium and longer span bridges, precast box girders placing side by side are used for medium span bridges and precast solid slab beams placing side by side are used for shorter span bridges.

3.1. NU Girders

NU girders have been developed specifically around the higher concrete strength that have become available. This results in a larger bottom flange in order to house more and larger prestressing strands of 15mm size. The bottom flange can hold 38 strands, while the web can carry two rows of strands, with 8 in the actual bottom flange at the low point.

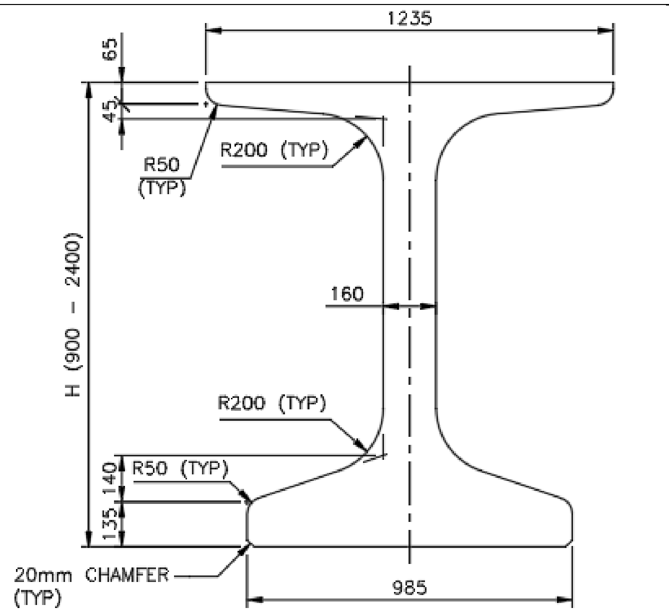
As opposed to the AASHTO or CPCI girders, all NU girder depths have the same top flange and same bottom flange. 160mm web thickness can accommodate two lines of strands when only pre-tensioning strands are used. NU girder design with combination of pre-tensioning and post-tensioning strands requires thicker web (185mm) to accommodate ducts for post-tensioning strands. The girders use modular forms with the top and bottom flanges being separate pieces with web formwork of differing heights inserted between them. All transitions are produced with radius between flange and web, allowing for better concrete placement.

Other advantages of the NU girder include a more even temperature distribution from casting up to transfer and for curing due to more uniform size of components, better stability during transportation and erection, less susceptibility to sweep under self-weight (1 in 1000 limit) and deck pour and less forming of deck slab (reduces formwork and labour).

The NU girders have a higher structural efficiency (I/A ratio) compared to CPCI girders. The more efficient NU shape results in shallower depth of girder compared to CPCI girders, or at the same depth, the NU shape can achieve slightly longer spans. The geometry and Standard Structural Drawings for NU girder used in Ontario is shown in

Table 2.

Table 2 – NU Girder Geometry Used in Ontario

NU Girders Height (H)	MTO SSD numbers	Dimensions
900	SS 107-16	
1200	SS 107-17	
1400	SS 107-18	
1600	SS 107-19	
1800	SS 107-20	
1900	SS 107-21	
2000	SS 107-22	
2400	SS 107-23	

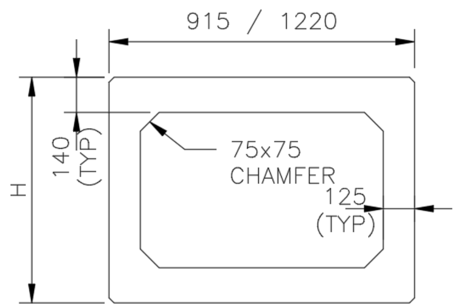
3.2. Box Girders

Precast box girders are available in standard sizes from B700 to B1000 with 100mm increment, although larger sizes have been used successfully. They are precasted in two standard widths 915mm and 1220mm. Non-standard widths were used in the past with the combination of standard widths to build the bridge deck but not desirable to avoid extra effort to build non-standard width boxes.

Construction of bridge deck with precast box girders side by side do not require formwork between the girders and the cast-in-place deck only requires one layer of reinforcement. Therefore, it requires reduced working days on site with compared to deck construction with spaced precast girders. However, the production of precast concrete box girder is more costly than precast concrete I-girders because two webs must be formed for each box girder. Moreover, the void in the box is formed with expanded polystyrene, which also adds to the cost. Since the box girders are used side by side, higher number of box girders are required than precast I girders with spacing for the same deck width.

Based on MTO experience and fabricator input, there is little benefit from deflecting (harping) the strands in the webs. For such shallow girders, the benefits of creating additional negative primary moments to counteract dead load are exceeded by the difficulty associated with deflecting strands. According to MTO policy, deflected strands shall not be used for boxes 900 mm deep or shallower. The bottom flange contains a single row of 15 mm strands, and each web contains a single line of strands, that are only deflected for the larger sizes. The standard sizes and Standard Structural Drawings for box girder used in Ontario is shown in Table 3,

Table 3 – Box Girder Geometry Used in Ontario

Box Girders Height (H)	MTO SSD numbers	Dimensions
700, 800, 900	SS 107-13	
1000	SS 107-14	

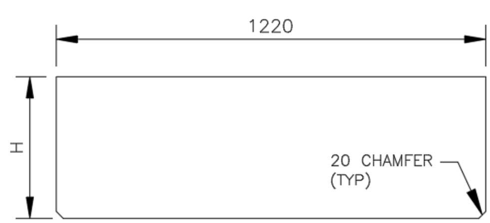
3.3. Hollow and Solid Precast Slabs

Hollow and solid precast slabs placed side by side and connected across the top using a cast-in-place concrete topping slab for deck construction provides an economical option for shorter span bridges. The 1220mm wide precast hollow and solid slab girders are produced according to the “S” series of girders. MTO is using solid slab girders with depths of 300 mm, 400 mm, or 500mm and hollow slab girders with depths of 400 mm, 500mm, or 600mm. Due to their shallow depth, these girders are fabricated with straight strands only.

Hollow slab girders have been used on occasion for bridges. The 1220 mm wide hollow slab typically contained three circular voids. However, they are not recommended for several reasons. The weight savings from the voids is not significant, and for the short spans that these are applicable for, the forces due to live load are significantly larger than from dead loads. Thus, the extra cost of forming the voids far exceeds the savings in weight. Also, the space between the voids and the exposed surfaces do not provide adequate space for the reinforcement to achieve concrete covers required by the CHBDC and the MTO Structural Manual. The standard sizes and Standard Structural Drawings for solid slab used in Ontario is shown in

Table 4.

Table 4– Solid Slab Geometry

Solid Slab Height (H)	MTO SSD numbers	Dimensions
300 - 500	SS 107-25	

3.4. Cross Sections for Precast Bridge Systems

The common and cost-effective solution for a prestressed girder superstructure is to use NU girders spaced at approximately 3 m on centre, with a thin deck slab (225 mm thick). This is typically built with a cast-in-place deck slab, or a cast-in-place deck slab cast over partial depth precast panels. The partial depth precast deck panels eliminate the need for formwork between girders, although formwork is still required for the cantilevers.

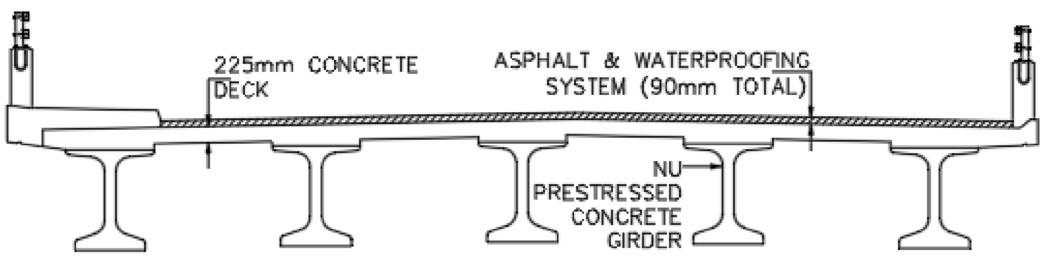


Figure 2 – Typical Deck Section with NU Girders

Box girders have been used in the past as separated girders, however there is minimal benefit compared to using an I-girder, so that practice has generally been discontinued. The typical application has been for side-by-side box girders, where there are several benefits. They have a better appearance from below, they completely replace deck formwork (except for the overhangs), and they result in a shallower superstructure compared to I-girder systems with spaced out girders, owing to more total strand across the bridge width. In this application, the boxes are connected at the top to transfer shear from one girder to the adjacent, ensuring equal vertical deflection on either side of the interface. This connection is routinely made with a 150 mm topping slab with single layer of reinforcement.

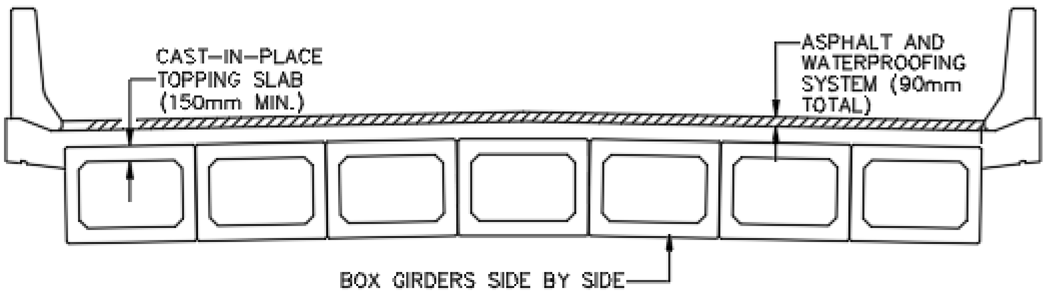


Figure 3 – Typical deck section using precast box girders with cast-in-place topping

In the last two decades, shear links have been designed directly between boxes with a thicker top slab (200mm) to directly carry wheel loads, as shown in Figure 4. Ontario is using this system to build bridges on low volume roads in the rural areas where ready mix concrete is not available. Due to the thicker and heavier box girder section, and the additional effort to form and then cast the shear links, the shear key option is significantly more expensive than using the topping slab and should only be used when concrete for the topping slab is not readily available. Analysis has shown that whether the boxes are connected for shear, or by a thick slab with two layers of reinforcement, there is little effect on the transverse live load distribution since this connection still has a small flexural stiffness compared to the box itself. Thus, a topping slab with two layers of reinforcement is not required.

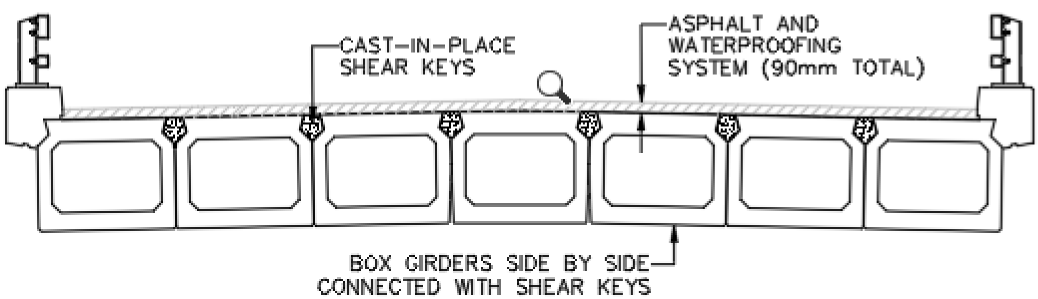


Figure 4 – Deck section with shear-connected non-standard side-by-side box girders

There has been some use of hollow core slabs in the past, always with a 150 mm topic slab. As described previously, they have several problems associated with them and should not be used for bridges.

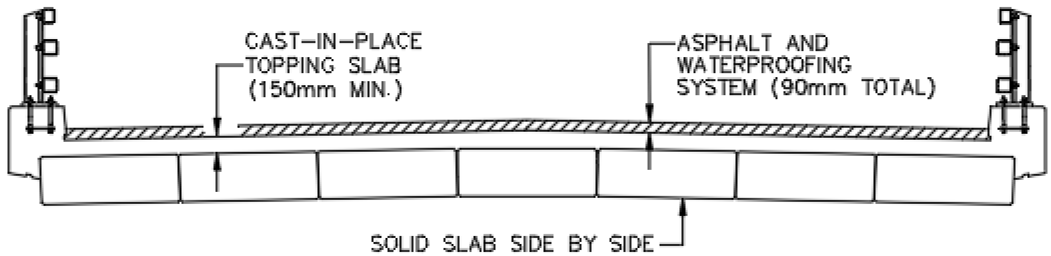


Figure 5 – Typical deck section using precast solid slabs with cast-in-place topping

3.5. Spacing of NU Girders for Optimum Construction Cost and Future Inspection

Although girder cost does increase with the size of the girder, the overall cost is dominated by the number of girders (i.e. total length of girder). Simply occupying a spot on the fabricator's casting bed is a large part of the cost. Fewer girders have a benefit of easing future inspection effort. The maximum depth of girder restricted by the available vertical clearance under the structure and the maximum 4.0m spacing of girders allows empirical method of deck design. Attempt to space the girders as large as possible up to 4.0m results optimum construction cost.

Tightly spaced girders make it physically harder to properly view between the girders. Consideration had been given to using side-by-side NU girders, which would be less costly than side-by-side boxes, however this consideration of accessing between the girders for inspection makes it undesirable. The minimum spacing between NU girders should be 1.8 m.

4. Design Considerations

For a given shape of precast prestressed girder, girder depth and girder spacing are related. For a given span, there is a range of feasible cross-sections. Since concrete girders have predetermined section sizes, the range of possibilities varies from deep girders spaced further apart, to shallower girders spaced closely together. Girder spacings of up to 4 m spacing result in cost-effective designs while still allowing for good constructability with respect to forming the deck slab and allow the deck reinforcing steel to be designed with the empirical method of deck slab design. At the other end of the spectrum, girders can be placed side-by-side to eliminate the need for the deck to be formed between girders. Due to tolerances in cambers, a thicker flange may be required.

For new bridges over rivers and in greenfield construction, girders should be spaced at least 3 m apart for economy and sustainability, as shown in Figure 6. The deck slab is a fixed cost whereas the cost of girders is related to the number of girder lines. Based on recent bridges, the girders are a sizable cost compared to the total cost of the deck slab. Therefore, reducing the lines of girders leads to overall economy of the cross-section, and a lesser use of materials. Considering only the girders and deck, the most economical NU girder bridge will be the one that uses the fewest girders, working within the transfer strength limit that allows a 24-hour production cycle.

For bridge replacements, the existing site constraints and grading often limit the cross-sectional depth of a new bridge. In those cases, it is more important to design a bridge within a prescribed depth than to reduce the girder lines, as shown in Figure 7. However, in no case shall NU girders be spaced more closely than 1.8 m to allow for removal of deck formwork and future inspection. MTO will consider options for having adjacent, or closer spaced, NU girders where there are net benefits, however this would require approval by the Head of the Structural Section.

Spaced NU girders shall be used where possible because they are the most cost-effective precast concrete solution in most cases. In some locations where placing formwork is exceedingly challenging (e.g. construction over an active rail corridor), or where cast-in-place concrete is unavailable at site, the side-by-side box may be preferable. However, with a typical NU girder spacing of 3 m, the side-by-side box girder bridge requires 2.5 times as many girders as an equivalent NU girder bridge.

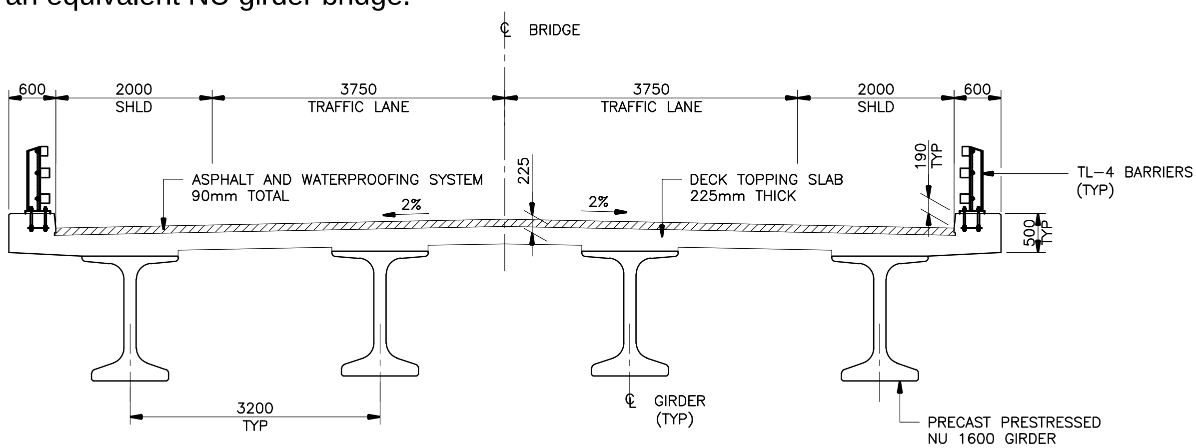


Figure 6– Typical deck section with NU girders at optimal spacing

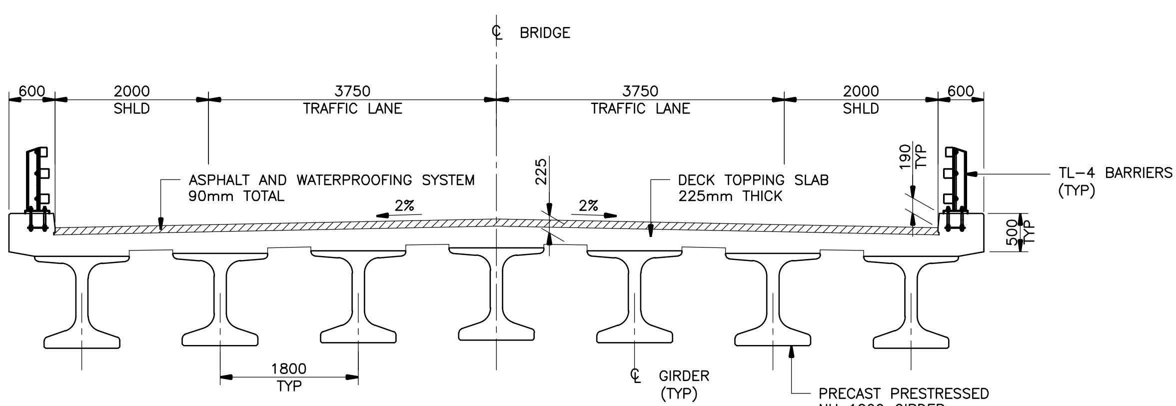


Figure 7 – Deck Section with NU girders of shallow depth, at minimum spacing

4.1. Concrete Stress Limits in Tension and Compression

Much of the flexural design of prestressed girders is for the Serviceability Limits States (SLS) requirements, with Ultimate Limit States (ULS) requirements done as a check. Shear design is completed at ULS. The CHBDC limits the SLS stresses in the girder at each stage of construction and service. When the girder is cast, the top surface is generally in tension, or close to tension, with the ends not aided by a compressive component of stress due to girder self-weight. The CHBDC Exceptions limit the tension to 0.6 f_{cri}. The bottom surface is subject to larger compressive forces and is limited to the 0.6 f'_{ci}.

In the 1980's, in the infancy of limit states design, prestressed concrete was treated almost as a different material than reinforced concrete and the thought was that there should be no tension in the concrete in service. The rationale being that if the complex process of prestressing was performed, then there would be a benefit to preventing cracking to enhance durability. It was also found that there was not a large cost differential in providing the additional prestressing to prevent this tension. With CHBDC, this was relaxed to allow tension, provided that the crack provisions were satisfied if the tensile strength of concrete was exceeded. Ontario took a conservative approach and continued to limit the concrete stress to half of the cracking strength. This was for additional conservatism due to the variability of the cracking strength, and the uncertainty of the various calculations including for differential shrinkage. As design methods have improved, the limit on tension has been increased to 75% of the cracking strength (0.75 f_{cr}).

Through discussion with CPCI and fabricators, MTO has standardised on the concrete strengths in Table 5. Most fabricators have developed a few mix designs that they are familiar with and meet the various requirements of the OPSS specifications. Although higher strengths have been used in other jurisdictions in North America, these are not applicable for Ontario. There are some differences in cement manufacturing in different parts of the continent and those cements are also better able to achieve the strength with less heat. The key factors for developing the standard concrete strengths is to balance strength with time (to keep production on a 24-hour cycle) while controlling heat. Precast box girders, with solid portions at their ends, have more massive concrete sections and thus require a lower target design strength to control the temperatures of these sections. Using a non-standard mix requires much greater effort for the fabricator to ensure compliance with specifications, so any higher strengths should not be specified unless approved. If the tabulated values are not required by design, lower values may be specified.

Table 5 - Standard Concrete Strengths

Type of Girder	f'_c (28 day cylinder strength)	f'_{ci} (Maximum initial transfer strength)
NU Girder	60 MPa	40 MPa
Box Girder	50 MPa	38 MPa
Solid Slab	50 MPa	38 MPa

4.2. Prestressing Steel Stress Limits

In some earlier codes, the stressing limit was set at 78% of the strand ultimate strength at jacking. There was some uncertainty in the way the clause was written as to whether this full force was transferred to the girder, or whether some nominal early strand relaxation or losses in the plant would be subtracted from it. CSA S6:19 was changed to clarify that a full 75% of the strand ultimate strength was to be transferred the girder at the time of release, and the fabricator would have to add a small amount depending on the magnitude of the losses prior to release at the specific plant. This change to 75% also resulted in a slightly lower prestress force being created, which added to the safety against strand rupture.

While CPCI girders were typically designed with 13 mm 7-wire low relaxation strands with an ultimate strength of 1860 MPa, both NU girders and box girders use 15 mm strand with an area of 140 mm^2 and the same ultimate strength of 1860 MPa.

4.3. Span-Spacing Charts

Design charts were developed to aid in proportioning of a bridge's cross-section with the limits described in previous sections.

4.3.1. Use of Charts to Determine the Girder Type, Size, and Spacing

The live load acting on a single girder is determined from the CSA S6:19 (CHBDC) simplified methods of analysis in Section 5.6. The live load fraction (F_T, F_S) varies by spacing but also by the width of the bridge, since narrower bridges, on average, see a higher live load fraction owing to the possibility of a greater proportion of the lanes being heavily loaded. The number of design lanes is taken from CHBDC Section 3.8, while the simplified equations in CHBDC Section 5.6 already include allowances for any number of lanes up until the maximum from Section 3.8. Generally, a bridge width of 9 to 12 m (corresponding to a 2-lane bridge or a narrow 3 lane bridge) usually produces the greatest live load effect.

The charts are also appropriate for estimating girders spans of integral abutment bridges. With integral abutments, the design engineer may have the impression that spans could be longer owing to a degree of rotational fixity at the abutments, but this is not the case. While moments due to superimposed dead loads and live loads are lower in end spans with integral abutments than in end spans with conventional abutments, the integral abutment connections also set up restraint effects. In the long-term, the bridge shortens due to creep and shrinkage, and the restraint of the shortening produces positive moment across the span.

Charts have been created for single span, two equal spans, and three spans with end span length of 90% of the main span. For non-optimal multi-span bridges, the tables may be marginally unconservative depending on the exact span ratios.

4.3.2. NU Girders

NU girders come in sizes ranging from NU900 to NU2400. Single span, two span and three span charts have been developed to estimate the maximum girder span which is achievable as a function of girder spacing and are provided in Figure 8, Figure 9 and Figure 10. They are developed for typical loads in Ontario bridges: a 225 mm deck slab, haunches as per CHBDC, a 90 mm asphalt wearing surface, concrete barriers (10 kN/m), and the CL-625-ONT live load. For exterior girders, the cantilever overhang is assumed to be no less than 40% of the girder spacing, and no greater than 60% of the girder spacing or 1.8 m. The charts also include the effect of thermal gradients at SLS1. Each chart is based extensive study of all possible bridge widths and lengths, achieved through analysis of designs produced by a computer program written to automate the design of over 2000 girder span and spacing permutations. Depending on the exact bridge width, and the number of design lanes, or if a refined analysis is done, the girder may be able to span slightly longer lengths, or have wider spacing, than shown.

Two-span and three-span charts assume a symmetric span arrangement. For a three-span bridge, the assumption is that approach spans are no greater than 90% of the main span. When that condition is met, the net positive moments in end spans will not be greater than the main span.

As the girders become more closely spaced with longer spans, the c/d ratio (neutral axis depth) increases. NU1600 through NU2400 spaced closely (spaced closer than 1.7 m) may have trouble meeting the c/d limit of 0.5 required by CHBDC. The c/d ratio should be determined by a strain compatibility analysis, to achieve an Mr equal to Mf. If the c/d ratio is greater than 0.5, it may be necessary to increase the compressive resistance of the slab. This can be accomplished by increasing the deck slab f'_c or adding compression reinforcement in the deck slab. Most practical designs, with girder spacing greater than 1.8 m, will satisfy the c/d requirement.

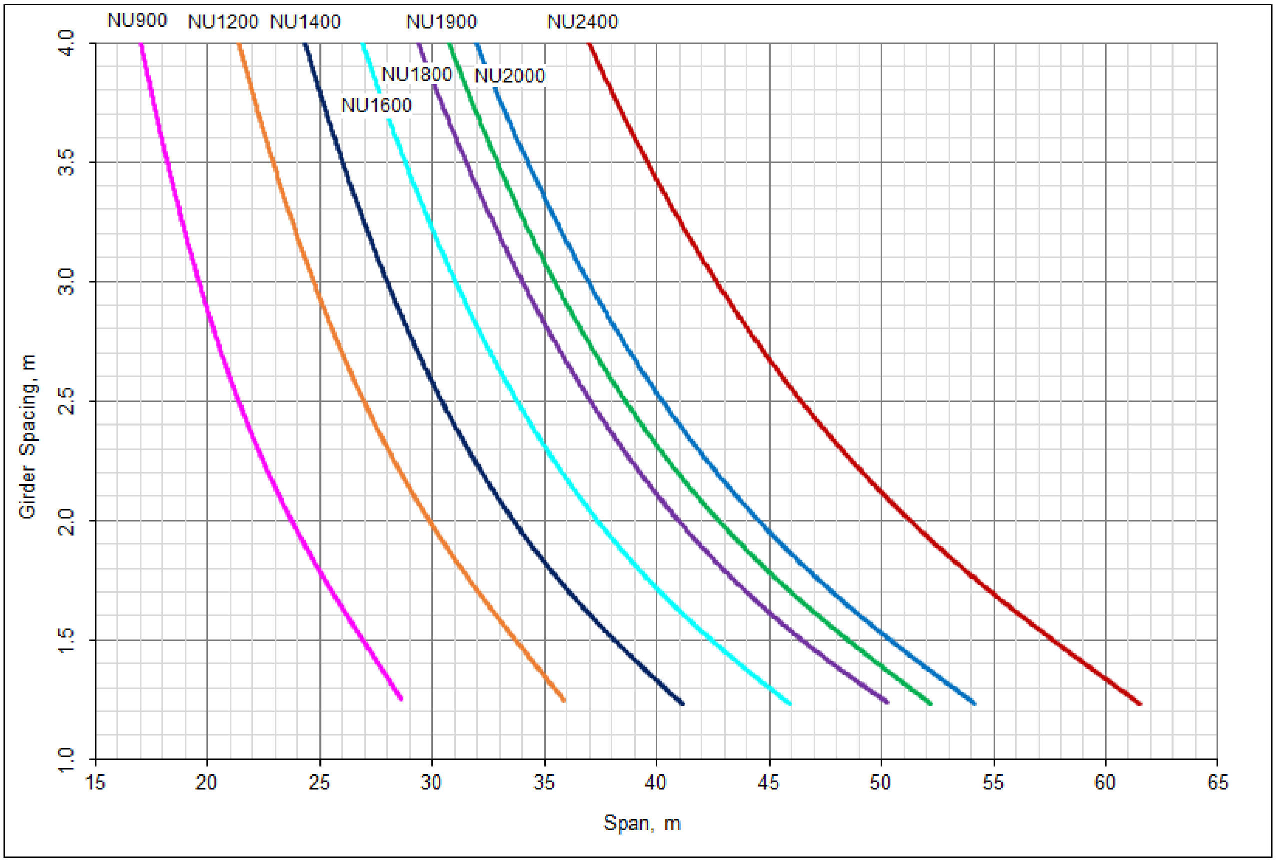


Figure 8 - NU Girder Span Spacing Chart for a Single, Simply-Supported Span

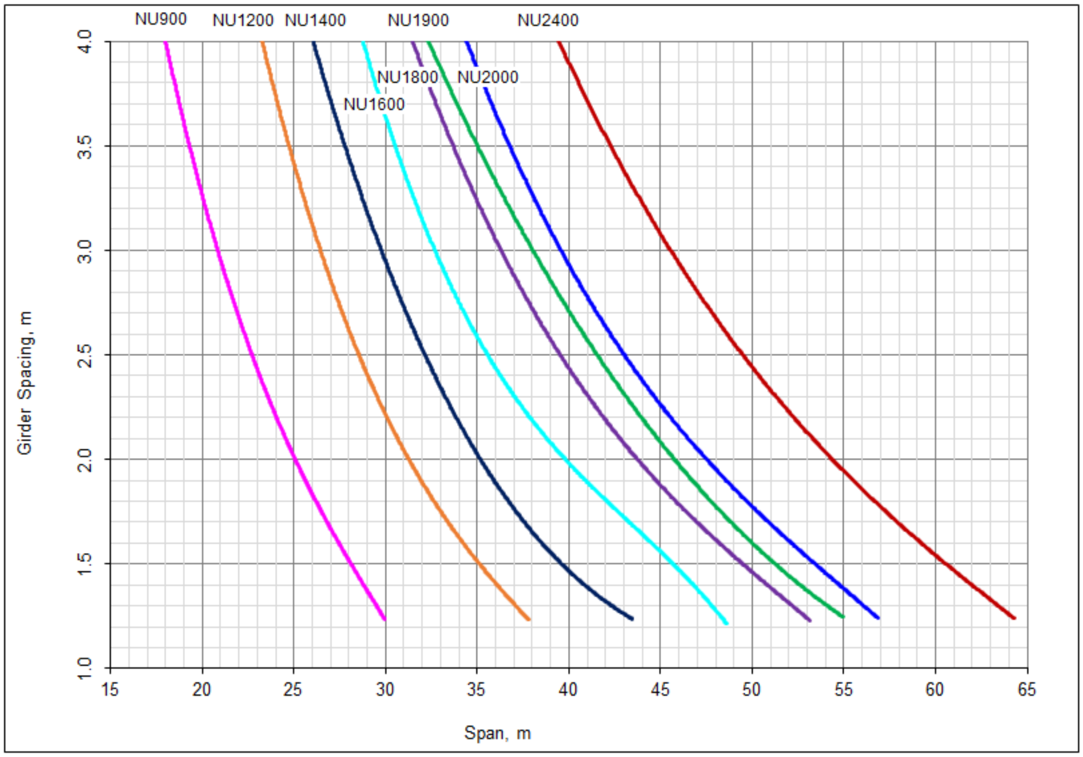


Figure 9 - NU Girder Span Spacing Chart for a Two (Equal) Span Bridge

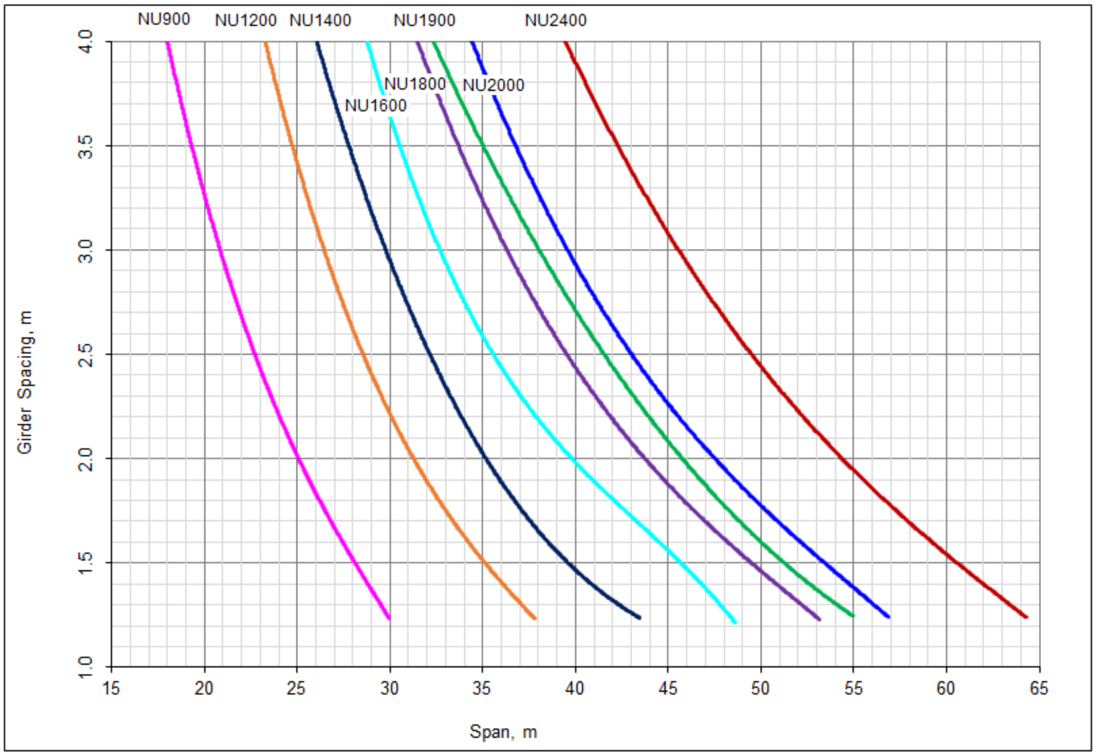


Figure 10 – NU Girder Span Spacing Chart for a Three Span Bridge (End Span 90% of Main Span)

4.3.3. Side-by-Side Box Girders

Side-by-side box girders span tables have been developed for single and 2 span (with equal spans) bridges, see Figure 11 and Figure 12. The boxes are 1220 mm wide and placed with a nominal gap of 10 mm to accommodate any tolerance, such as sweep, so that accumulation of these tolerances won't lead to a wider bridge. Since the spacing is set (essentially touching), the tables simply show the applicable ranges. Similar to the NU Girders, these charts were determined by analysing a wide assortment of bridge widths and design lanes, to determine the range of applicability – with some extremes eliminated. Generally, the live loading is higher for bridge width at the lower limits for the specific number of lanes show in CHBDC Section 3.8.2, while it is lower for bridge width closer to the higher limit. For example, a 14 m wide bridge and a 16.5 m wide bridge would both carry 4 design lanes, but the wider bridge would have 2 extra box girders, meaning the 16.5m wide bridge carries less load per girder, and a similar depth of box could span longer.

The charts assume a topping slab of 150 mm thickness placed on top of the boxes, along with the 90 mm asphalt and parapet walls (10kN/m) and 500 mm overhang.

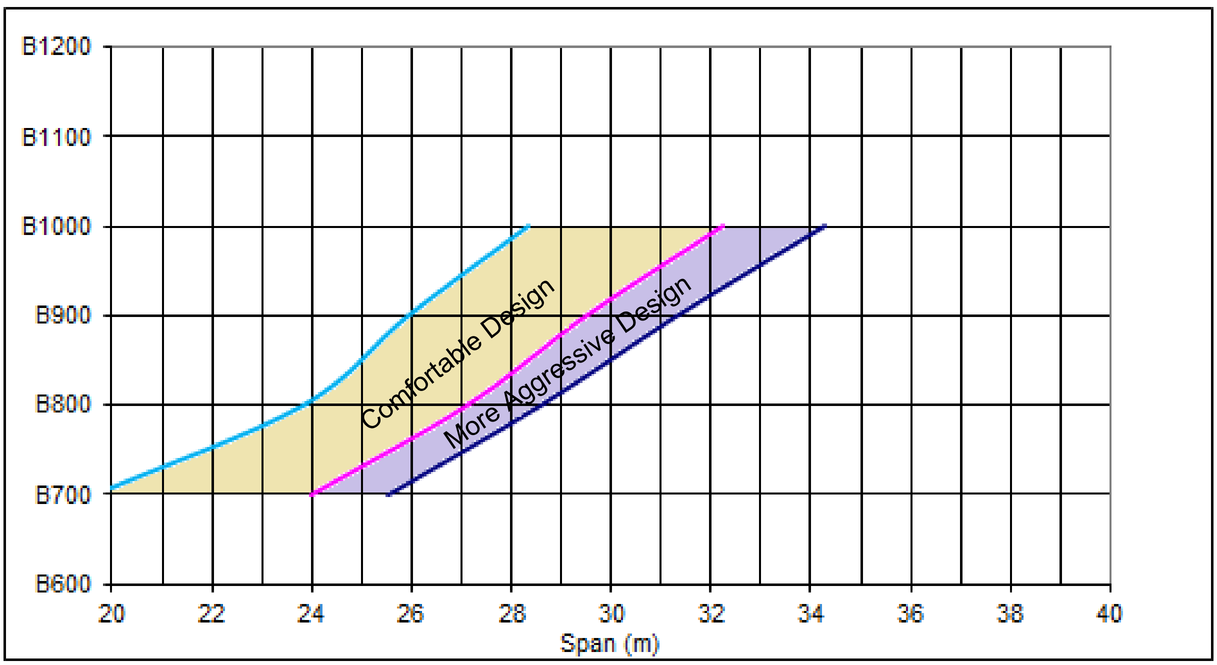


Figure 11 – Singe Span Side-by-Side Box Girder Span Applicability Chart

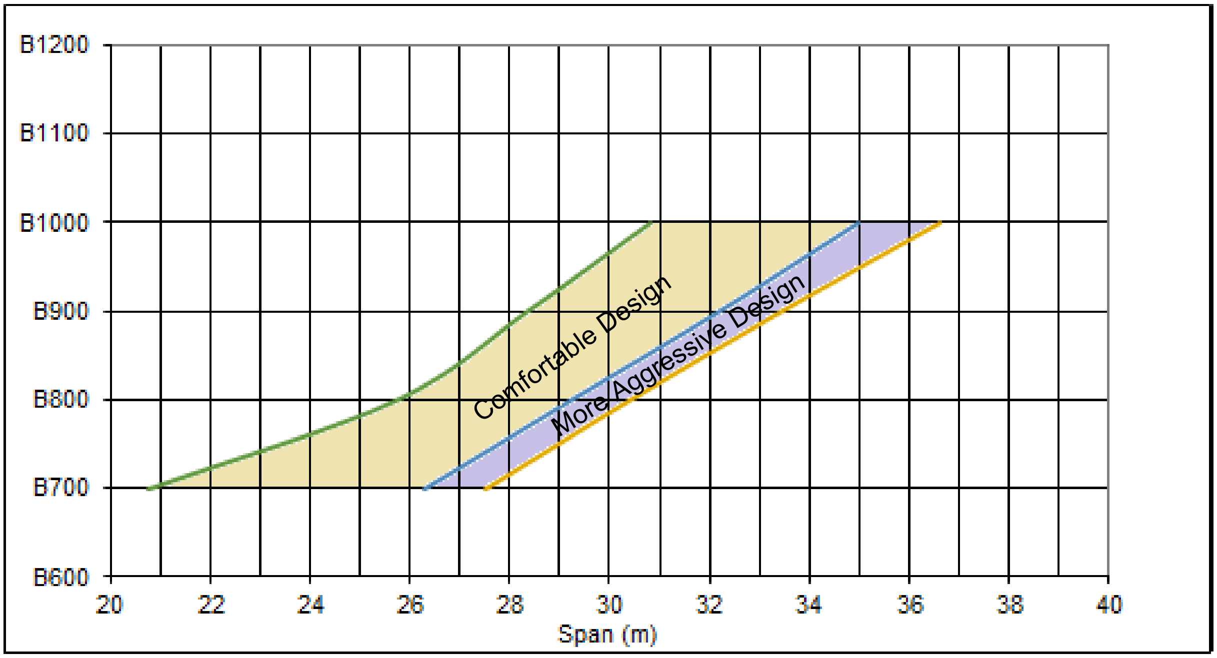


Figure 12 – Two (Equal) Span Side-by-Side Box Girder Span Applicability Chart

4.3.4. Solid Slabs

Deck construction with solid slabs side-by-side can use similar details as with box girders. Span chart for precast solid slabs with depth of 300 mm, 400 mm and 500 mm placed side-by-side is shown in Figure 13. This chart is developed considering 150 mm cast-in-place topping slab over the precast solid slabs. Precast solid slabs used 50 MPa concrete with 35 MPa transfer strength and the cast-in-place topping used 30 MPa concrete.

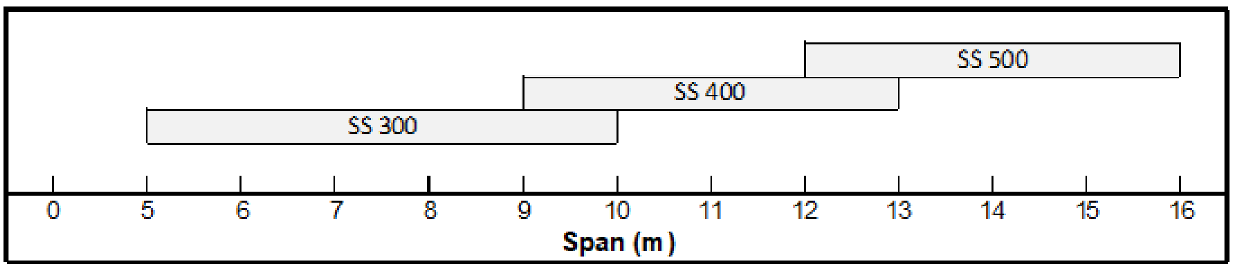


Figure 13 - Single Span Side-by-Side Solid Slab Span Applicability Chart

4.4. Detailed Design

4.4.1. Design Methodology

1. Design Criteria

Girder design shall follow the latest edition of CHBDC (CSA S6-19 at the time of this writing). It is also required to follow the Exceptions to the CHBDC, which is found in Division 1 of the Structural Manual. These 2 are mandated by Ontario Regulation 104/97, made under the *Public Transportation and Highway Improvement Act* (*PTHIA*).

For all MTO designs, it is also required to follow the MTO Structural Manual, as well as any policy memoranda that have been issued. The latest versions of these documents are available on the MTO Technical Publications website (https://www.library.mto.gov.on.ca/SydneyPLUS/TechPubs/Portal/tp/TechnicalPublications.aspx).

2. Load Effects

Loading and load factors on a bridge are taken from CHBDC Section 3. The first loading applied to the prestressed girder is the prestressing force. As the force is applied in the prestressing bed, the girder cambers upwards at the midspan and this imparts the self weight of the girder onto the girder. These loads are applied early in the life of the concrete, at roughly 16 to 18 hours age.

Once on location, in addition to the self weight, the girder is subject to loading from the deck and overhang formwork, construction loading and the weight of the wet deck slab (or topping slab). Once the deck hardens, the bridge will have been continuous or integral (if applicable) and act compositely between the deck and girders. At this point, additional dead loads of barrier or curb/railing and asphalt are applied.

The typical loading that governs for a prestressed girder bridge is highway loading, either truck load along with dynamic load allowance (DLA) or lane load (which can govern on longer spans), applied to the continuous bridge. CHBDC Section 5 can be used for regular bridges to determine the transverse live load distribution, to determine the fraction of a truck acting on a girder. In more complex cases, a structural model can be used, applying the correct number and locations of traffic lanes, to directly determine the forces carried by a girder. In some instances, differential settlement may occur, which will impose deformations and the resulting force effects from the time the girders are made continuous or integral, if applicable. Other loads maybe applicable in certain conditions and shall be applied as required. There are other forces from prestress losses and differential shrinkage which must be considered.

3. Material Properties

Material properties for concrete and reinforcing/prestressing steel are found in Section 8 of CHBDC. Weights of these materials can be found in Section 3.

4. Section Properties

Typically, the girders have 2 stages to consider. When being cast, the member is the naked girder section itself and it must resist all forces from when the girder is released from the stressing bed, stored in the yard, transported to site, erected, supported deck formwork, construction loads, and the wet concrete deck. The gross section properties of the NU girders can be found in Design Aid 7-1 of the Structural Manual. The gross section properties of concrete girders yield a somewhat more conservative result and should be used for new girder design. The thickness and weight of haunch should be considered, confirming any initial assumptions after the full prestress camber profile is calculated. The haunch can reduce tensile stresses by up to 0.75 MPa.

Once the deck has been cast and hardened, which typically involves casting pier and abutment diaphragms to make the structure continuous, if applicable. For this case, the girder and deck slab form a T-section, whose section properties can be determined from the parallel axis theorem. The nominal haunch depth can be included in determining the section properties and do provide a noticeable benefit. It can be confirmed after detailed calculations are done that the expected haunch will be similar in magnitude, as this depends on the prestress camber acquired and the roadway profile. In calculating the composite section properties, CHBDC Section 5 must be used to determine the effective flange width for determining the section properties. If the spans are short, the forces “do not have time” from the point of inflection to fully engage the flange. Also, the deck slab is typically cast with different concrete strength than the girders, so the section properties must consider the modular ratio between the 2 materials by transforming the deck into equivalent width concrete of same stiffness as the girder.

5. Prestress Strands

Both NU and Box girders use 15 mm strands with an area of 140 mm^2. Fabricators have standardized their operations to have a regular 50 mm grid on which the prestressing strands can be placed. In the past, some fabricators had accommodated draped strands at the hold-down points to have a 25 mm or 38 mm spacing, however that didn’t allow proper consolidation of the concrete and didn’t work with most hold down devices. Thus, minimum 50 mm strand spacing shall be used throughout the girder.

6. Initial Estimate of Prestress Losses

As soon as the jacking force is applied into the prestessing strands various prestressing losses start occurring. CHBDC classifies the pestressing losses into two groups at transfer and after transfer based on their time of occurrence.

Prestressing losses at transfer:

1. Relaxation of strand
2. Elastic shortening of girder

Prestressing losses after transfer:

1. Creep
2. Shrinkage
3. Relaxation of strand

Guidelines to estimate these losses are provided in the CHBDC Section 8.7.

7. Initial SLS Stress Checks

Prestress girder design involves checking concrete stresses in SLS during prestress transfer and in service to ensure SLS stresses are within allowable limits, which typically govern the girder design. Then the girder is checked for its flexural and shear capacity in ULS.

The stresses in the girder are accumulated over the life of the girder, with the first stresses on the naked girder and later stresses on the composite girder section ("cg" in Figure 14) . These stresses must be less than the allowable in the final condition, which has a limit on compression at the top and a limit on tension on the bottom, as shown in Figure 14. The strand force "F" is the unknown but can be estimated using the assumed prestress losses. The eccentricity, "e", can be assumed to be 90% of the girder height. The moments from Dead Load (DL), Superimposed Dead Load (SDL), and Live Load are obtained from the analysis (moment from differential settlement or other sources can be added). The stress due to differential shrinkage can be calculated during the detailed check but can be assumed to be 0.5 MPa at this stage. Typically, the tensile stress at the bottom fibre governs, so the Numbers of strands can be determined by summing the stresses in the bottom fibre so that the total equals the allowable tensile stress (0.75 f_{cr}), although a somewhat lower value should be targeted at this stage to account for some of the assumptions that have been made. With a preliminary estimate for the number of strands, the stress at the top fibre can be check against the allowable (0.45 f'_c under dead load and 0.60 f'_c under total load). The stress at transfer at the top (tension) and bottom (compression) fibres can also be checked using the first three stress diagrams of Figure 14.

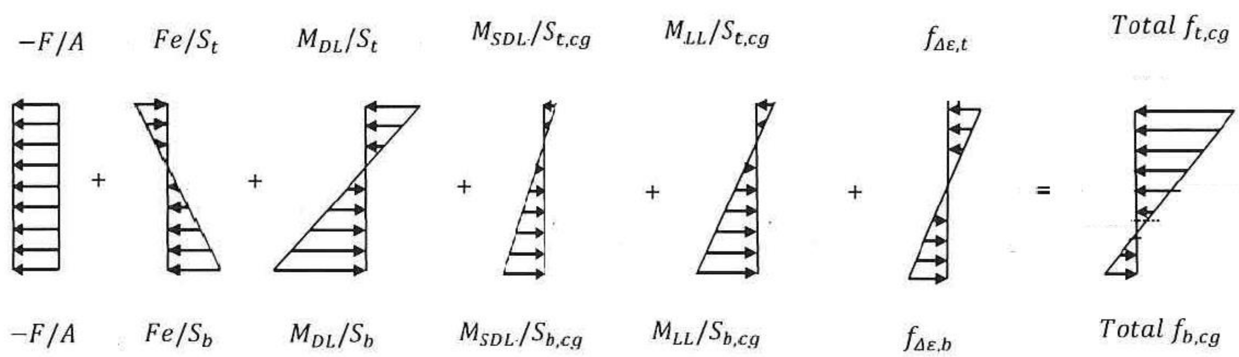


Figure 14 – Concrete Stresses in Girder at Midspan.

The next part of the design check is to ensure that the concrete stresses at the ends of the girders are adequate. At this location, the dead load is virtually zero, so the only stresses to consider are due to the prestress (the axial and bending components), as shown in Figure 15. It must be remembered that the prestress forces are higher at this stage than in service, as only some initial losses have occurred. Invariably with the lack of dead load moment assisting, the girder end would fail if the same arrangement as the midspan was used. There are two options to solve this problem, either:

a) the strands in the web can be draped so several of them are moved above the girder neutral axis at the ends – which reduces the overall eccentricity, or
b) some of the horizontal strands in the bottom flange can be debonded, so that they only transfer the force into the girder some distance from the end where the dead load stresses begin to increase.

The former cannot be done (is not economical) for shallow prestressed members. The latter is not allowed for bridges with expansion joints where water may enter the debonded strand leading to future corrosion.

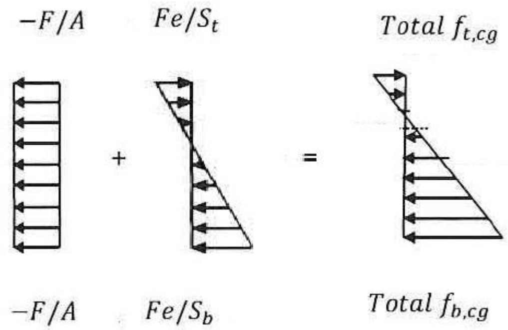


Figure 15 – Concrete Stresses in Girder at Ends.

If deflected (draped) strands are used, all strands in the web would normally be draped. Although it varies somewhat in each case, about 30% of the total number of strands, can be draped. The typical hold-down point for strands is at about 1/3 of the span. If debonding is used, no more than 35% of the total strands shall be debonded, and typically no more that 4 debonded at one location. There are typically multiple debonding locations spaced 1 to 2 metres apart.

8. Detailed Prestress Losses

In order to perform the detailed calculations for the prestress losses, the exact strands forces and resulting concrete stresses, must be known in order to perform the calculations for elastic shortening, relaxation, creep, shrinkage. The various prestress losses can be calculated in accordance with CHBDC Section 8.7.

9. Differential Shrinkage Stress Checks

Differential shrinkage stresses arise since the girders are cast at an earlier time than the concrete deck. The concrete deck then tries to shrink more than the girders, which places a compressive force onto the top flange of the girders. This force is resisted by the girder. Accounting for the eccentricity of this force from the composite neutral axis, the resulting moment puts a tensile stress into the bottom of the girder. It also results in a tensile stress in the deck, which must be considered for crack width calculations in negative moment regions of the bridge. This stress distribution can be seen in Figure 16.

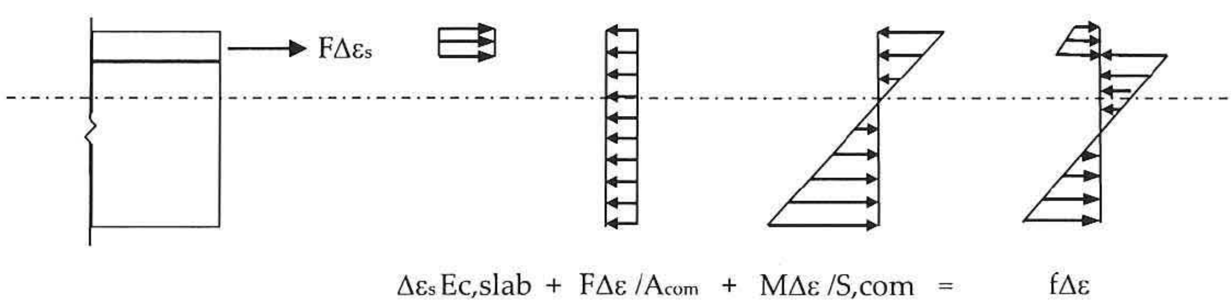


Figure 16 – Concrete Stresses Due to Differential Shrinkage.

10. Final SLS Stress Checks

Using the final prestress forces and losses, the stress checks can be done similar to the descriptions above. Typically, the stresses are checked at every 10^{th} span location, although sometimes more frequent may be required for longer spans. In the initial condition after transfer, often the hold down location become critical as the strand force is already low in the girder causing tension on top, while the self weight moment is not close to its maximum. In service, the tension on the bottom face at the midspan often governs. Other checks need to be done, using a similar stress accumulation methodology, in the concrete deck that did not receive any of the prestress force, but is still subject to tensile loads (near piers and integral abutments). The deck typically contains heavier reinforcement at 150 mm spacing at these negative moment locations.

11. ULS Flexural Check

After an SLS design has been completed, the ULS capacity must be checked in accordance with CHBDC Section 8.8 but is typically adequate. It is also required to confirm the location of the neutral axis to ensure the section is not over reinforced, and the section is checked against the cracking moment.

For integral and multi-span bridges, the negative moment capacity must also be checked. Typically, this governs at the section immediately beyond the girder, where there is no prestress effect. The moment is resisted in large part by the reinforcement in the deck, meaning that no adjustments are needed to the girder design.

12. ULS Shear Design

Shear design is completed in accordance with CHBDC Section 8.9. The shear resistance is obtained from 3 components, the concrete component (Vc), the steel (stirrup) component (Vs), and the prestress component (Vp). The prestress component depends on the prestress force and its vertical component. The concrete component depends on the concrete strains and reinforcement spacing and area. This is not typically designed for, and simply a value that is obtained from the girder flexural design. The component the designer has the most control over is the stirrup component. Stirrup reinforcement is added to compensate for the deficiency in shear capacity after consideration of Vp and Vc. Only 15M stirrups can be used as the bend radius limitations would not allow a larger bar to fit within the web. In high stress areas near the supports, the shear stress is relatively high and a close spacing can be expected. The spacing requirements of Clause 8.14.6 shall also apply. Although the CHBDC allows stirrup spacing of 600 mm or 0.75 dv in low stress areas, MTO practice has been to use 450 mm maximum spacing for girders 1400 mm deep and shallower.

After the shear stirrups are determined, checks must be made on the amount of longitudinal reinforcement on the tension and compression sides of the girder to ensure that the horizontal component of the Vc force can be carried by the longitudinal reinforcement.

13. Anchorage zone design

At the ends of a prestress concrete girder, there is a large concentration of forces being introduced into the girder. CHBDC Section 8.16.3 specifies requirements to limit and control the end splitting cracking that could occur. This typically involves several closely spaced stirrups near the girder end. Where minimal or no draped strands are used on NU girders, additional reinforcement may be required to prevent cracking near the web-bottom flange interface.

14. Hold-down

When draped strands are used in the prestress girder design, the location of the slope change of the draped strands is termed as their hold down points. The change in slope of the draped strands results in a hold-down force that must be resisted, as shown in Figure 17. Based on fabricator plant designs, the maximum hold-down force that can be accommodated is 80 kN. Typically, this translates into 4 to 6 strands being held down at one point. The slope of the strand is also limited to 1:6 to allow for the strand through the bulkhead to be properly stressed.

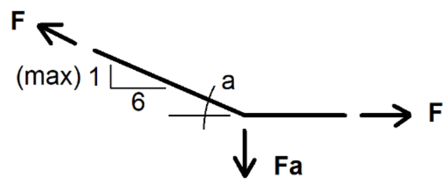


Figure 17 – Hold-Down Forces

15. Camber

Deflection and camber must be considered in establishing the screed elevations. The deflections are required to set the screed elevations. This is the elevation that the concrete finishing screed machine rails are to be set so that when all loads are placed the road surface will be at the desired elevation. The screed elevation is the final roadway profile, plus the deflection of the naked girder due to loads (wet slab), plus the deflection due to loads on composite girder (superimposed dead loads of barrier/railing/curb and asphalt), see Figure 18. There is also an effect on these deflections from prestress losses and differential shrinkage, as well as from formwork loads that are applied to a naked girder but removed from a composite bridge, that are not described and are often small but should be considered.

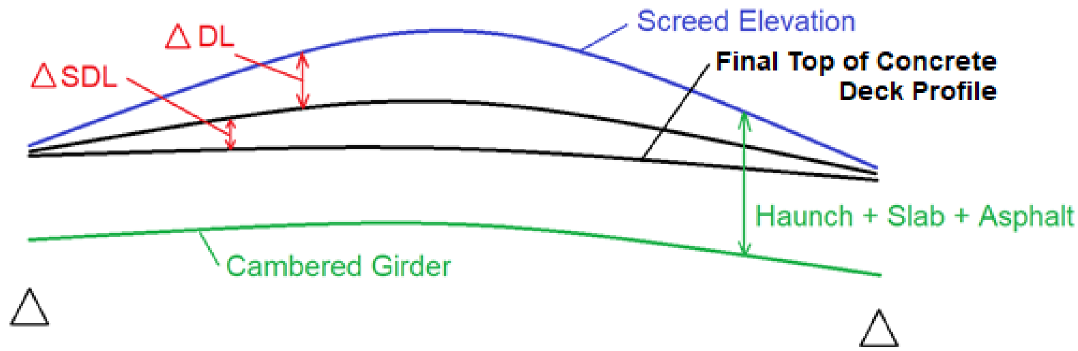


Figure 18 – Girder Screed and Camber

The difference between the screed elevation and the cambered prestressed girder gives the height of the haunch (plus a constant value of the deck slab) along the length of the bridge, see Figure 18. This, of course, is based on some uncertainty due to the creep calculations and the variability of the actual girders and is adjusted in the field during construction to set the haunch heights, which is the more precise value. This haunch height estimate/calculation is needed for several reasons. Due to the crossfall of the road, and the width of the girder flange, the haunch will be somewhat less on one side of the girder. This low side haunch should not be less than 0 mm, see Figure 19. Normally the standard haunches (which are applicable at the supports) in the Structural Manual (50mm for shallow girders and 75 mm for deeper girders) is adequate to maintain a haunch on the low side, although these may be increased if required. Also, this haunch height (at centre-line of girder) is used to determine the projection of the stirrup above the top of girder. The stirrup must extend at least 25 mm above the bottom mat of reinforcement and the stirrup must be at least 100 mm below the top surface of the deck, as shown in Figure 20. The stirrup projection can change along the length of a girder, if required.

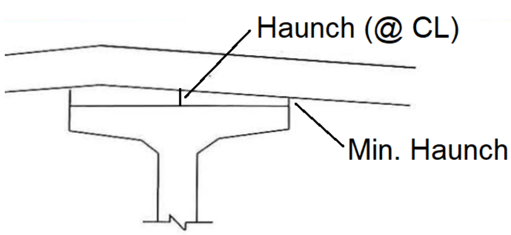


Figure 19 – Haunch Height

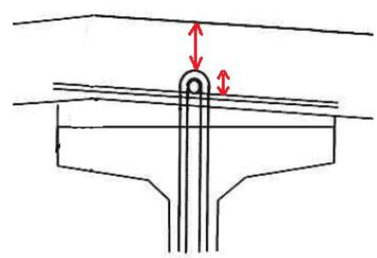


Figure 20 – Stirrup Projection

This camber and deflection information is also needed to ensure that the girder sits flat against the bearing. The prestress creates an upward deflection and thus end rotation of the girder. The girder self weight and wet concrete create a deflection and rotation in the opposite direction. Finally, due to the highway profile, the girder will have a rigid body rotation (somewhat matching the grade), with direction depending on whether the girder is sloping upwards or downwards. Considering these factors, the underside of the girder will not exactly sit flat and be in full contact with the bearing, see Figure 21. If the difference is minimal, it could be taken up by the rotation of the bearing (more compression on one side than the other). Otherwise, the girder must be undercut either at the back side or front side in order for the bearing to sit flat and true, as shown in Figure 22. If undercut is required, it is added to the formwork as a beveled plate, which should have a minimum dimension of 6 mm at the thin end. The maximum undercut allowed is 18 mm, as it will eat into the concrete cover. If a larger undercut is required, then instead of an undercut, a bevelled steel plate is embedded on the bottom flange to account for the girder end slope.

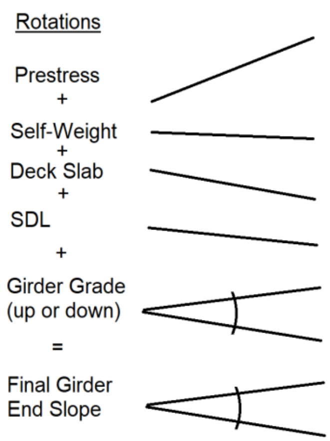


Figure 21 – Girder End Rotation

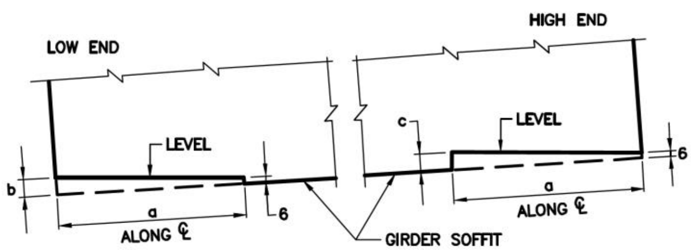


Figure 22 – Girder Undercut

Each of the deflections and rotations described shall have factors applied as per CHBDC and the Structural Manual to account for long term effects.

16. Bearings

With NU girders, typically laminated elastomeric bearings are used. Their design, size and height, need to be adequate for the loads, movements, and rotation they will see.

17. Cropping Corners on Skewed Bridges

When bridges are constructed on the skew, it is preferable to leave the girder ends square, as that requires the least amount of effort by the fabricator. When the skew becomes larger, the flange tip may be too close to the ballast wall (or back of the integral abutment) and needs to be cropped along the skew (see Figure 23). At piers, the girders can typically be kept square, although this depends on the skew and the pier cap width.

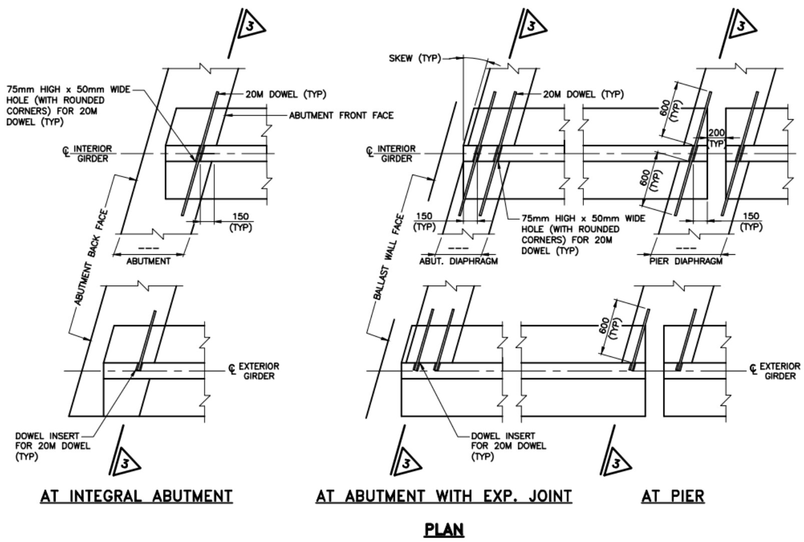


Figure 23 – Cropping Corners of Flanges on Skewed Bridges

5. Construction Considerations

5.1. Fabrication

5.1.1. Precast Operations

Fabrication of prestessed bridge girders is a major operation conducted in a facility that has made significant investments in plant and stressing equipment. Production can be indoors or outdoors, see Figure 24 to Figure 28. The stressing bed is a level surface that cannot be altered without great difficulty. The stressing bed is typically long enough to cast one girder, although they can cast several in one line if the girders are shorter. Solid abutments, against which the tensioning force is applied, are placed at each end of the stressing bed.

Figure 24 – Precast Plant

Figure 25 – Precast Plant Interior

Figure 26 – Stressing Bed

Figure 27 – Stressing Abutment

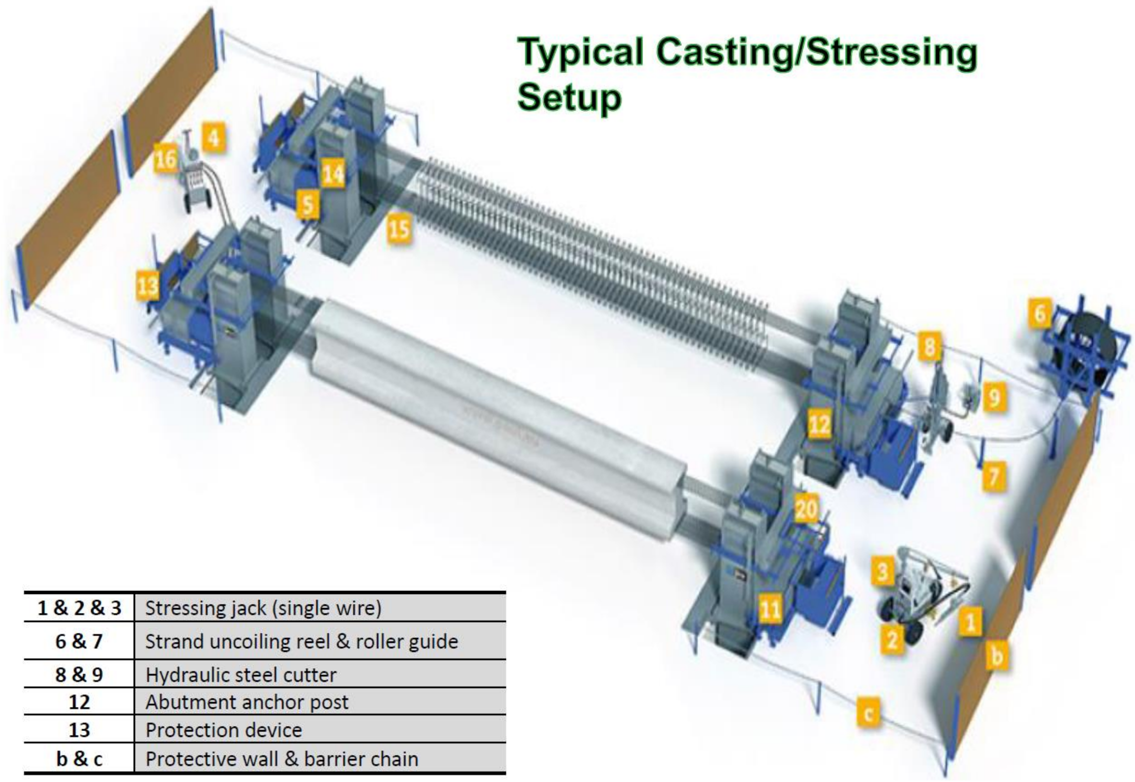


Figure 28 – Typical Casting Setup (image from ask Bo)

The reinforcement for the NU girder is installed in the bed, along with the stressing strands which are placed through steel bulkheads to properly position the strands. Draped strands are always installed into their draped alignment through the hold-downs in their correct position, and stressed in that position with low friction hold-downs. Then the side forms are brought in to complete the formwork. After the concrete has been poured and reached the calculated transfer strength, the strands are cut, the forms separated and the girders are moved to a curing enclosure, see Figure 29 toFigure 32. As can be seen, the forms for many fabricators have a middle segment that can be changed depending on girder depth.

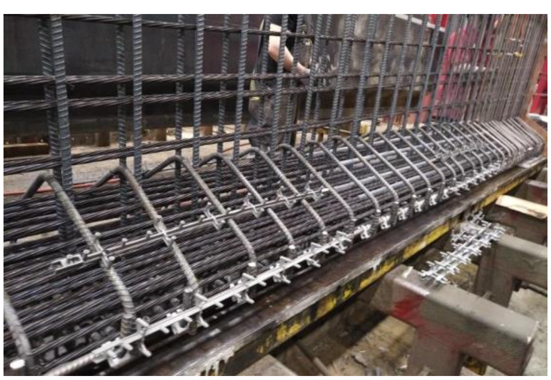


Figure 29 – NU Rebar Cage

Figure 30 – NU Girder Forms

Figure 31 – NU Bulkhead

Figure 32 – NU Form Removal

For box girders, the fabrication process is similar, with part of the rebar cage and strands being installed, expanded polystyrene void added, remainder of the rebar cage added and then the formwork brought in, see Figure 33 and Figure 34.

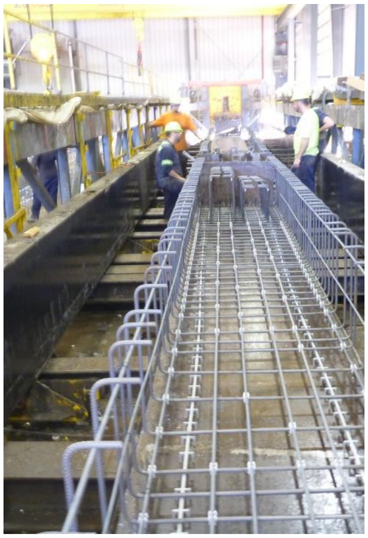


Figure 33 – Box Rebar Cage

Figure 34 – Box Girder Void

The stressing operation is typically done with single strand jacks in a predetermined sequence. The jack pressure and elongation are measured to ensure the proper stress it put into the girders. Steel wedges are used against the strands and tapered holes in the stressing abutment to lock the strands after stressing. After the concrete has gained the required transfer strength, the strands are torch cut to release the girder from the stressing abutment. The length of strand over this relatively short length (girder to abutment) does not retain adequate energy to cause a massive rupture, and the torch cutting heats the strand to lose its modulus of elasticity, elongate, as another mechanism that insures a high energy rupture does not occur. This is still a careful operation that must be done slowly in proper sequence. See Figure 35 to Figure 36 .

Figure 35 – Stressing Jack

Figure 36 – Pressure Monitoring

The largest cost in the production of a precast girder is labour, followed by reinforcement, followed by the concrete material cost. By extension, the space in the stressing bed has a significant cost as all labour is geared towards producing 1 girder per day. If a girder has to spend an additional day in a stressing bed to achieve a higher concrete strength, all the available labour would sit idle. In terms of reinforcement cost, adding straight strand is easier than adding reinforcing steel as it is the same operation as is done for the neighbouring strands, while any other reinforcement requires more labour to install.

5.1.2. Hold Down Locations

To create the draped strand profile, strands are held down using a low friction device, usually comprised of rollers. The strands are place through the hold down device and stressed. Some fabricators had deflected the strands to their final hold-down position after some initial tensioning was applied straight, however this is not the common. Stressing typically occurs from one end. For straight strands, the force in the strand will be the same in each end. As a draped strand is tensioned, the hold down will incline a small degree as the strands attempt to take the shortest route between the bulkheads. Since the hold down may introduce some friction which may cause loss of stress at the dead end, the fabricator must confirm that the stress losses are not excessive. This can be done by doing a pull-off test at the far end, to determine the force required to lift the strand wedges from the surface, and this force must match the input stress from the stressing side. A load cell can also be used at the dead end, or in the final portion beyond the final hold-down, the elongation is measured between 2 marked points to ensure that the elongation is appropriate for the expected strand force.

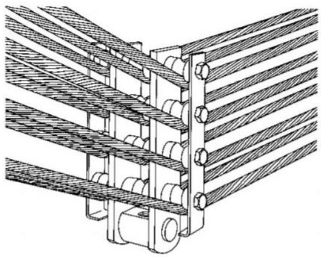


Figure 37 – Hold Down Device

Figure 38 – Hold Downs

5.1.3. Debonding

The purpose of debonding strands is to not impart the stressing force into the girder at the ends. Since all stressing must be done from the stressing abutments, plastic sleeves are placed around the tendons for a certain length so that the strand slips and only begins to transfer the stress into the girder at the end of this sleeve. It had become more noticeable with the higher strength strands that the traditional soft sheathing did not perform adequately. As the strand is stressed, due to Poisson ratio, the strand diameter decreases. Under the hydrostatic pressure of the concrete pour, the flexible sheathing would compress around the strand. When the strand was cut, the end portion would have zero longitudinal stress, which would cause the strand diameter to return to its original value. This resulted in radial outwards forces on the concrete that caused some cracking, and also created high friction which prevented much of the debonding from occurring. This was known as the Hoyer effect. A Michigan DOT study showed that the use of rigid tubing would solve this problem. See Figure 39 and Figure 40

Figure 39 – Debonded Strand

Figure 40 – Soft vs. Rigid Sheathing

5.1.4. Temperature Limits

Maximum temperature for precast concrete girders is 70 degrees C. Limits on the temperature were determined based on studies on Delayed Ettringite Formation (DEF) in Ontario concretes.

DEF is a serious long-term durability issue. Ettringite forms within air bubbles and decreases the effectiveness of the air entrainment, and creates expansive forces which lead to ongoing cracking.

Temperature limits can be managed by managing the initial concrete temperatures and managing the heat gain through the hydration process. For the former, ice water can be used in lieu of water. Heat is controlled by using supplementary cementing materials to reduce the cement content. Liquid hydrogen has also been used to cool the hydrating concrete. These methods are often used to achieve the concrete strengths specified in Table 5, depending on the time of year production occurs.

Temperature is highest at the most massive parts of the girder, which is the bottom flange of NU girders and the solid ends of box girders. They are monitored using thermocouples for internal temperature and differential temperature between the surface and core, the latter which is limited to 20 degrees C, see Figure 41 and Figure 42.

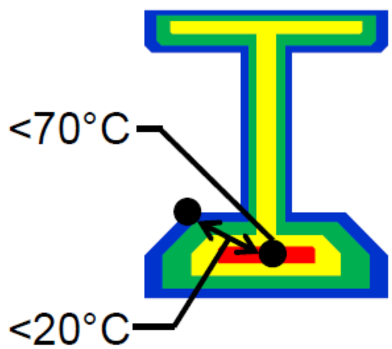


Figure 41 – High Temperature Areas

Figure 42 – Surface Temperature

5.1.5. Curing

After the girders are removed from the beds, curing continues for 4 days after casting in accordance with OPSS specifications. This is done with a sprinkler system or by moist curing in a fog mist chamber, with adequate misting to cover the entire length of the girder. Wet burlap can be used for smaller members. After the curing, the girders are stored outside until ready to be delivered to the site. See Figure 43 and Figure 44.

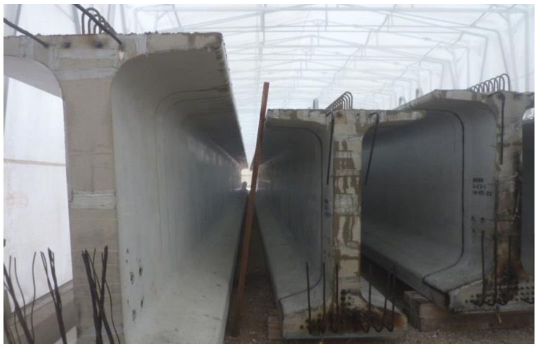


Figure 43 – Moist Curing

Figure 44 – Girder Storage

5.2. Transportation and Erection

Girders are stored in the fabricators yard and transported to site when required. They are transported one at a time for most typical girders. They are placed on the truck with a rear dolly carrying the back half of the girder, see Figure 45. This dolly has the ability to steer at low speeds as the girder leaves the yard and arrives at site. Often, the girder is classified as an oversize load, and often also overweight, and required permit from MTO to travel to site. Generally, the height is not a concern, nor is the width – it is purely a length consideration. MTO has annual permits for vehicle and load up to 25 m, project permits for vehicle and load up to 37.75 m, and single trip permit to 45.75 m, see Figure 47. For longer girders, temporary top strands are added for stability during transportation, with the strand subsequently removed and the holes later filled with grout.

Figure 45 – Transportation

Figure 46 – Girder Erection

Lifting of the girder is done through lifting loops near the ends, for all lifting in the plant and on site. Since the girder relies on the self weight to create positive moment at midspan to counteract the prestress forces, the lifting locations must be near the ends to maintain these moments. The girders are usually lifted directly from the truck to the bridge supporting elements.

When placed, NU girders require temporary bracing to ensure their stability before releasing from the crane, as shown in Figure 48. Minimum bracing is provided in the Contract documents, but responsibility for supporting the girders rests with the Contractors – with the precaster typically providing the details. For box girders, whose width is greater than their height, they are stable without such braces. The bracing is attached to the ends of the girders and to the support, typically with cables or struts. More recently, it has been found that bracing is also required 5m into the span to prevent vibration and fluttering of the girders under truck gust or wind gust loadings. The contractor is responsible for any additional bracing needed with the installation of deck formwork, see Figure 49.

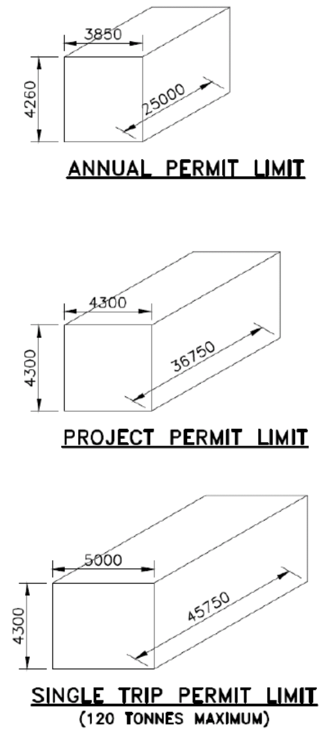


Figure 47 – MTO Permit Sizes

Figure 48 – Girder Temporary Bracing

Figure 49 – Deck Formwork

5.3. Girder Camber and Early-Age Creep

The prestressing moment applied to a girder always exceeds the moment due to the self-weight of the girder, resulting in an upwards hogging of the girder. Precast girders are particularly sensitive to early-age creep because the prestressing is applied to the girder at a very early age, typically assumed to be 0.75 days for design purposes. The upwards hogging of a girder depends on the prestressing force, the force eccentricity and the dead load acting on the girder at the time of initial transfer. Typically, the dead load is a smaller component and the girders hog upwards. Furthermore, it is not possible to introduce a camber into the girder because the precast beds are always dead level, and it is not practical to ‘weigh them down’ in storage to prevent upwards creep, although some adjustments may be necessary if one girder is observed to behave differently than its neighbours. As the concrete gains strength and stiffness with time, the rate of creep decreases.

Delays in girder erection can increase the camber of a girder over time. Girder erection may occasionally be delayed by up to 8 months due to unforeseen circumstances, and one of the concerns is increasing hogging deflections. If hogging progresses, it may reduce the haunch towards the centre of span and may affect the ability of a deck to be cast to the correct screed elevations, or cause the stirrups to project too close to the top surface of the deck. If these delays occur, the girders should have some weight added to prevent this increasing camber from occurring.

6. References

1) Bridge Girders Technical Guide, Precast Concrete Girders and Beams, Regional Specification for AB/MB/SK, Armtec, 2012
2) Con Force Alberta - Technical Information Bridge, 2004
3) CPCI Design Manual, 5th Edition, Precast and Prestressed Concrete CPCI, 2017
4) NU Girder Bridge Design and Detailing Manual, Volume 1 Manual, Alberta, August 2018.
5) NU Girder Bridge Design and Detailing Manual, Volume 2 Design Examples, Alberta, August 2018.
6) Prestressed Concrete Manual for Quality Assurance of Bridge Structures during Construction, MTO, Mihaljevic et al., 1998.

Appendix A: Construction Issues and Repairs

1) Concrete Placement and Consolidation

The mixing of concrete, placement of concrete and consolidation are critical to making a durable girder. Most of the defects discovered on the girder can be attributed to the construction process. As such, the quality assurance requirements of the specifications are critical to follow.

Figure 50 – Casting Concrete, Outdoor and Indoor

2) Physical Defects

A prestressed girder can contain a significant amount of concrete and can be over 2m deep. Cold joints, an indication of improper vibration and consolidation between multiple batches of concrete, can also contain consolidation problems such as voids. The NU girder bottom flange, and the underside of prestressed box girders, are horizontal (or near horizonal) surfaces that are especially vulnerable to improper vibration. These surfaces can have honeycombing or void problems. See Figure 51 toFigure 54.

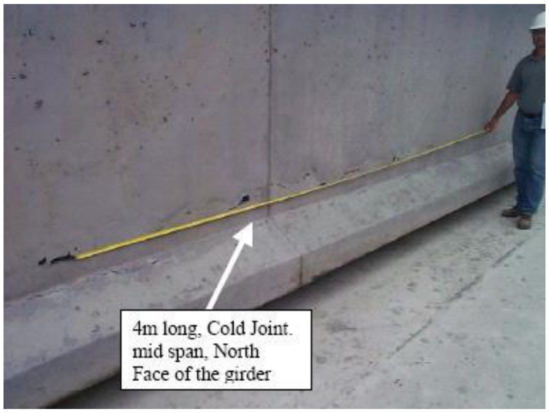


Figure 51 – Cold Joint

Figure 52 – Cold Joint Void

Figure 53 – NU Girder Voids

Figure 54 – Box Girder Soffit Void

Other defects that can occur are spalling and cracking. Spalling can occur due to improper releasing of forms, or due to physical damage due to impact. The thin top flanges of NU girders are susceptible to the latter. Spalling may also be from bursting forces relating to the transfer of force to the girder and/or the end reinforcement to resist it. Cracking can occur from the tensile stresses in the girder ends due to transferring the loads. They tend to be lower with draped strands and well spaced debonded strands. Controlling the sequence of strand release, along with proper stirrup reinforcement helps resists these cracks. If the cracks are contained within the future cast-in-place diaphragm or abutment (when Integral), there they typically pose no durability concern.

Figure 55 – Bottom Flange Spall

Figure 56 – Top Flange Spall

Figure 57 – Web Spall

Figure 58 – Web Cracking

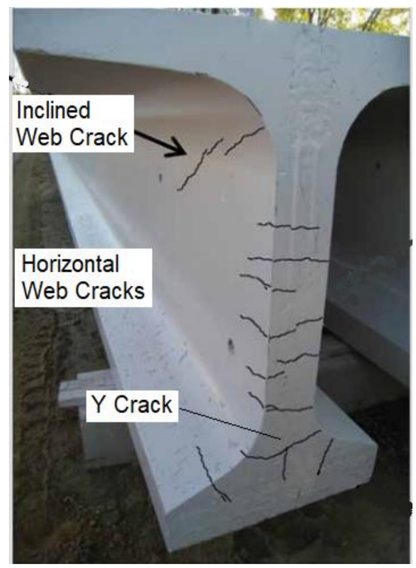


Figure 59 – End Cracking Pattern

3) Temperature

Temperature is checked during hydration using thermocouples. The two concerns are absolute temperature at the core of the precast member, and differential temperature between the surface and the inside. The temperature control could include use of ice, supplementary cementing material, scheduling concrete pours away form the hottest time of day, use of liquid nitrogen for formal thermal control plans with cooling pipes and temperature modelling. The use of these methods depends on the time of year and the size of the member being cast. Aside from the planning, monitoring the curing enclosures for breaches and ensuring temperature control equipment is working ensures the temperature requirements are met. See Figure 60 and Figure 61.

Figure 60 – Cooling Pipes

Figure 61 – Curing enclosure

4) Curing

Besides curing temperature, moist curing is also essential in creating a durable concrete. The main concerns are that the proper moist curing time is achieved, and the moist curing functions as required. These concerns are similar to regular cast-in-place concrete and included burlap not thoroughly wetted or not covering all surfaces, broken vapour barrier, inadequate sprinkler reach. For members in fog mist curing chambers, insufficient misting may not reach the full length of the member, and members spaced too close may not allow proper moisture to reach the surface.

Figure 62 – Long Mist Chanber

Figure 63 – Box Girders Close Toghether

5) Dimensional Checks and Sweep

Various dimensional check must be done on the girders to ensure they are fabricated in accordance with the specifications. The include squareness at ends, sweep, camber, and projecting reinforcement spacing. These can be checked using simple tape measure, string line and level.

Sweep has been less of a problem with NU girders, with their larger weak axis moment of inertia, but could still be a concern. Improper deck forming and work platform installation could alter the sweep and it should be measured before installation. For box girders, excessive sweep would mean that adjacent boxes would not be close enough together. Excessive sweep could be an indication of eccentricity in the prestressing, differential shrinkage and creep across the cross section or thermal gradients when measured, see Figure 64 toFigure 66.

Figure 64 – Girder Measurement

Figure 65 – Measuring Sweep

Figure 66 – Girder Sweep