Economical Steel Box Girder Bridges

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Composite steel box girder bridges have become an effective alternative to more conventional plate girder bridges and concrete girder bridges in North America in the past 25 years. At present a number of innovations taking place in Ontario and elsewhere have the potential of further enhancing the design, durability, constructibility and, by extension, the economics of steel box girder bridges. These innovations include reduction in the number of boxes, posttensioning of the concrete deck, and construction technology. These developments are by no means the end of opportunities to further enhance steel box girder technology. Combinations of steel boxes and concrete boxes, posttensioned decks, single box girder bridges, multiweb and multicell boxes are all design and construction options that have potential. For the evolution of steel box girder bridges to continue, researchers, code writers, and designers must continue to cooperate with owners and agencies to take advantage of current advances in modeling and analysis software, instrumentation and testing technology, and the current trend among designers and researchers to do more full-scale modeling, analysis, and testing of total bridge structures.

Steel box girder bridges have become an accepted method of constructing highway bridges. Significant design and construction advances have occurred in the past 25 years.

THE PAST

In 1965 the "Criteria for Design of Steel-Concrete Composite Box Girder Bridges" were incorporated into the AASHTO bridge specifications. These criteria were based on folded plate analysis methods and scale model testing completed at that time. Richard Fountain presented this background research to the Canadian Society for Civil Engineering in 1968. This research has provided the basis for current code provisions in the AASHTO specifications, Ontario Highway Bridge Design Code (OHBDC), and elsewhere for the past two or more decades.

At the conclusion of his paper, Fountain made the following comment: "By using the least number of boxes practical to support a cross section, it should be possible to obtain designs requiring the least amount of steel. Such designs will also require the least number of boxes to be fabricated and erected" (*1*). This statement is fundamental to the design principles presented in this paper.

THE PRESENT

There have been a few bridge projects recently for which the designers have reduced the number of boxes to support a given cross section. The result has been a corresponding increase in the number of design lanes per box. Some owners and agencies have encouraged this trend through the technical review and approvals process and through inclusion of provisions in current codes that allow designers to move outside the limitations of the original criteria published in 1965, providing they can demonstrate an appropriate methodology for analysis and design. Significant economic advantage can be achieved by reducing the number of boxes required to support a given cross section.

Bridges are complex structures. For this reason, designers have sometimes attempted to reduce their analytical models to simple or continuous beams to reduce the analytical effort. Design codes have reflected this approach. Typically a complex bridge deck is idealized as a beam or an orthotropic plate to assist designers by providing simplified methods of live load distribution. This has resulted in bridges that are difficult to calibrate to the actual design because many important characteristics of the superstructure are ignored in the analytical method. This situation is beginning to change.

Modeling bridges in three dimensions using elements designed to simulate the behavior of actual bridge components is particularly important for steel box girder bridges. Designers are gradually moving in this direction as desktop hardware and software with modeling capability becomes available in the design office. The result will be bridges with a more uniform strength reserve, that are more economical as a result. Researchers can begin to correlate structural responses based on testing to actual designs.

THE FUTURE

The history of box girder bridges in North America is relatively short. Many innovative and cost-effective steel box girder bridges have been constructed in the past 25 years. Future design innovations may include posttensioning of concrete decks, precast decks, multimaterial designs that integrate concrete and steel box girders, multicell/multiuse, and single box bridges. Construction innovations may include more incremental launching and lateral sliding.

Current codes in North America generally provide designers with a simplified method of live load distribution for bridges. This approach is rational for a large percentage of the smaller and less complex structures. However, it may be counterproductive in some respects by limiting designers to approximate solutions and simplified methods of analysis when a more direct method of analysis and design could result in a better understanding of bridge behavior and in significant economies. This is particularly true of larger and more complex structures.

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Limiting criteria in current codes restrict the designer to a maximum number of design lanes per box. These criteria are defined by the original research and development on which the current provisions are based and have been used effectively for many years. These limiting criteria should be reviewed carefully to explore box girder configurations outside the limitations of the current code provisions. Further research and development in this area have the potential for increasing the cost-effectiveness of steel box girder bridges in the future.

DESIGN FOR ECONOMY

Fewer Boxes

Steel box girder bridges are inherently more efficient than conventional plate girders and reduce the amount of steel required for bending and shear by virtue of their torsional stiffness and enhanced live load distribution. A significant incidental benefit is the reduced transverse bending in the deck due to better load sharing between boxes and corresponding reductions in differential deflections.

Figure 1 demonstrates schematically how the torsional stiffness of a closed box girder section contributes to better live load distribution in a bridge superstructure.

Several current codes contain the following live load distribution provisions for composite steel box girder bridges. The provisions are in accordance with the 1989 AASHTO bridge specifications, Clause 10.39.2.1, and the 1983 Ontario Highway Bridge Design Code (OHBDC), Clause 3.7.1.4.2(b).

$$W_L = 0.1 + 1.7\,R + \frac{0.85}{N_w} \qquad 0.5 < R < 1.5 \tag{1}$$

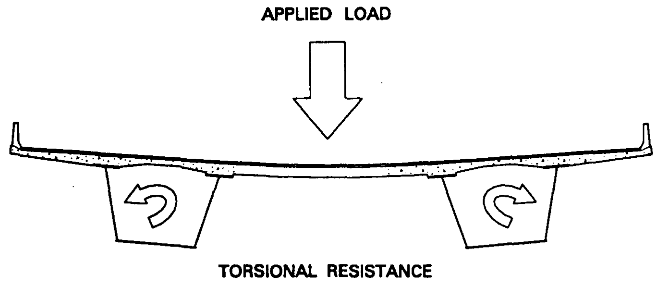

FIGURE 1 Live load distribution.

where

W_L = fraction of wheel load;

$R = \dfrac{N_w}{\text{number of box girders}}$;

N_w = number of design lanes;

N_w = $W_c/12$, reduced to nearest whole number; and

W_c = roadway width between curbs (ft).

As can be observed in Table 1, Ontario designers have typically designed bridges for one design lane per box. This is not accidental, given the limiting criteria for R.

On several recent projects listed in Table 1, there have been significant increases in the ratio of design lanes per box.

Figure 2 demonstrates how the total live load carried by a given cross section decreases as the number of boxes is reduced. In the process, the amount of steel in the bridge has been reduced. This is a result of the inherent torsional behavior that makes steel boxes more efficient for live load distribution, but, more significantly, it is a result of reducing the number of webs. Webs are typically underused in box girder bridges. Minimum thicknesses are required for fabrication and handling. Much of the web in a bridge cross section

TABLE 1 Bridge Design in Ontario

	SPAN (m/ft)	NUMBER OF LANES[a]	NUMBER OF GIRDERS	R
Communication Rd	58/190	2	2	1
Burnstown	76/249	2	2	1
Lacroix	46/151	4	4	1
Whirlpool	50/164	3	3	1
Brimley	51/167	3	3	1
Trenton	74/243	2	2	1
Burlington Bay Skyway	64/210	5	3	1.67
Hunt Club	100/328	3	1	3
Hwy 407	80/262	7	2	3.5
Humber 1	81/266	3+	2	1.5
Humber 2	81/266	5	2	2.5

[a] Design Lanes as defined by AASHTO (1989) and OHBDC (1983)

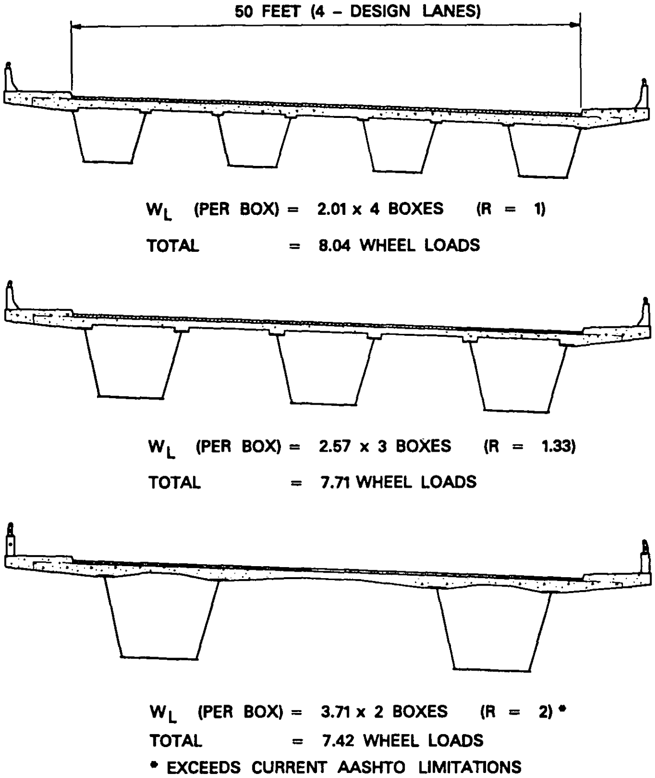

FIGURE 2 Total live load.

is required to provide a reasonable separation between the top and bottom flanges and occurs in areas of relatively low shear.

Redundancy and Fatigue

Redundancy becomes an issue in multispine bridges where the number of spines is reduced. Several factors alleviate this concern for box girder bridges:

- As the number of girders is decreased, the ratio of dead load to live load increases. This in turn reduces the stress ranges for fatigue and the potential for fatigue problems.
- Boxes consist of individual plate elements. It can be argued that as such, even single box girder bridges have multiple load paths and significant redistribution potential.
- Fatigue cracks in the web will typically commence in regions of high stress and move toward regions of lower stress. This has the effect of slowing rather than accelerating the propagation of these cracks.
- Sometimes the designer will choose to carry longitudinal bottom flange stiffeners continuously through positive moment regions for stability and strength, particularly during construction. This provides additional load paths should fatigue cracks develop in the flanges.
- The same arguments can be applied to longitudinal web stiffeners.
- Quality control continues to improve in fabricating plants. Fully automated welding, NDT methods, and girder building machines are increasing the quality of the finished product.

Unit Weight of Steel

Figure 3 represents the relationship between the unit weight of steel in a bridge relative to its longest span. These curves represent a data base of several hundred steel bridges constructed in Ontario between 1980 and 1990. Several observations can be made from this figure:

- Single-span bridges become inefficient compared to multispan bridges in the range of 40 m (130 ft).
- The unit weight of steel for multispan continuous bridges is linearly proportional to the span.
- The unit weight of steel is consistently reduced where the number of boxes is reduced ($R > 1.5$) on the basis of these data.

Figure 4 illustrates an actual bridge for which an alternative preliminary design was completed using two boxes instead of four. This bridge is a five-span structure with main spans of 72 m (236 ft) and an overall length of 319 m (1,047 ft). The alternative design reduced the total weight of steel in the structure by 500 tonnes (550 tons), or about 20 percent of the total.

Longer Transverse Deck Spans

Reducing the number of boxes increases the transverse deck spans. This could be considered a disadvantage. Several solutions to this apparent disadvantage could be considered by the designer:

- Increasing the deck thickness,
- Vaulting the deck soffit,
- Considering arching action, or
- Transverse posttensioning.

Increasing the deck thickness, within limits, can have certain benefits: increased stiffness, reduced cracking, increased mass, and better dynamics are examples. The cost of material and forming probably begin to offset these benefits for average deck thicknesses of more than 250 to 300 mm (10 to 12 in.).

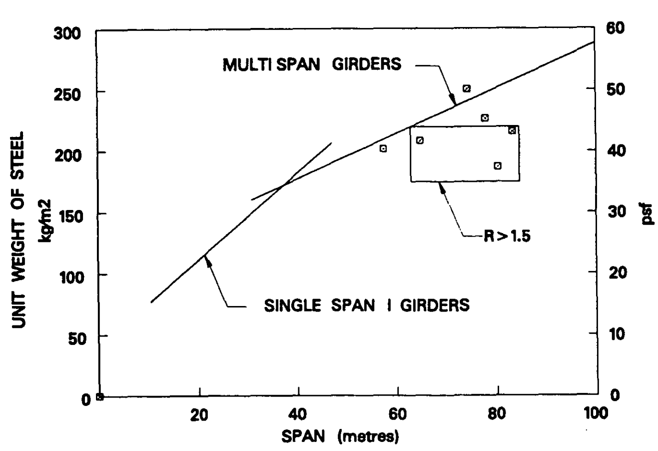

FIGURE 3 Unit weight of steel.

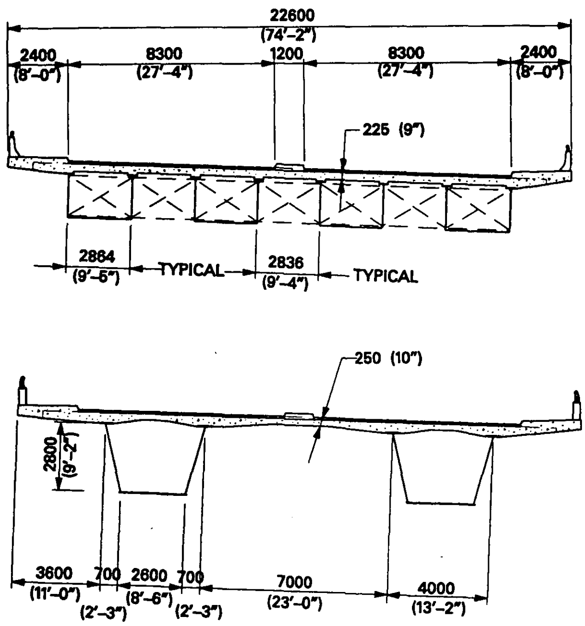

FIGURE 4 Longer deck spans: *top*, conventional cross section; *bottom*, alternative cross sections.

Vaulting the deck results in a thicker deck, which requires more concrete. Some benefits are associated with this practice, including enhanced durability and damping of dynamic effects.

Deck arching becomes more effective in resisting live loads when the deck thickness is increased. Deck arching has been adopted by some agencies (such as Ontario) when certain geometric criteria are met; it reduces significantly the amount of reinforcing steel needed.

Current code provisions have relatively stringent requirements for the maximum tensile stress in concrete decks. The result can be a significant amount of reinforcing steel for longer spans. Transverse posttensioning is a cost-effective way of controlling the maximum design tensile stress, providing the required strength and reducing the amount of normal reinforcement at the same time.

As the deck span increases between adjacent girders, the designers must consider the distribution of both live load and dead load effects for the following components and configuration of the design:

- Framing,
- Transverse deck bending,
- Girder torsion, and
- Bearing configuration.

The relative stiffness and interaction of these elements is fundamental to the behavior of a steel box girder bridge. If the designer wishes to consider single bearings, for example, the torsional effectiveness of the girders for live load distribution may be significantly reduced as the torsional fixity provided by double bearings is eliminated. This has the effect of redistributing torsional effects into the cross frames and the deck.

Deck Posttensioning

As a result of reducing the number of boxes, the transverse deck span can exceed the practical limits for normally reinforced concrete and transverse posttensioning becomes necessary. The transition to posttensioning appears to be necessary in the range of 5 to 6 m (16–20 ft).

The following parameters were used on a recent bridge using transverse posttensioning in Ontario: the clear span between boxes was 9.86 m (32 ft 4 in.); the deck thickness varied from 260 to 460 mm (10.24 to 20 in.). The transverse posttensioning consisted of four 15-mm (⅝ in.) strands at 700-mm (30-in.) centers.

The cross section of this bridge is illustrated in Figure 5. Figure 6 is a schematic representation of the posttensioning layout, staging, profiles, and type. This deck was posttensioned using a combination of strand and threadbar to accommodate geometric and staging constraints.

Table 2 gives some of the key data used for this posttensioning design. The amount of reinforcing steel compares favorably to Ontario deck designs, which average 30 to 40 kg/m^2 (6 to 8 lb/ft^2) based on the empirical deck design provisions of the OHBDC, and transverse deck spans that vary from 2 to 4 m (6 to 12 ft). Posttensioning the deck can have significant

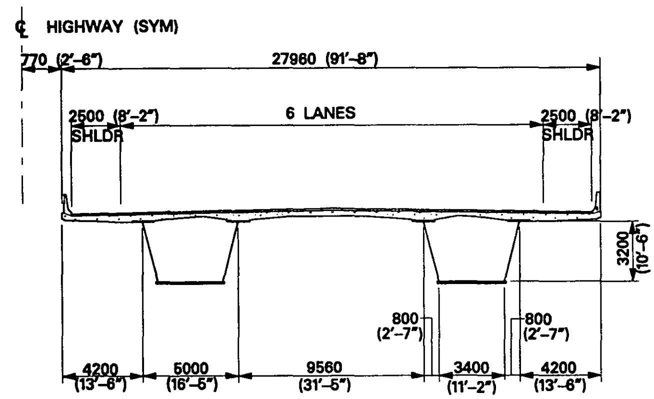

FIGURE 5 Posttensioned deck, Highway 407.

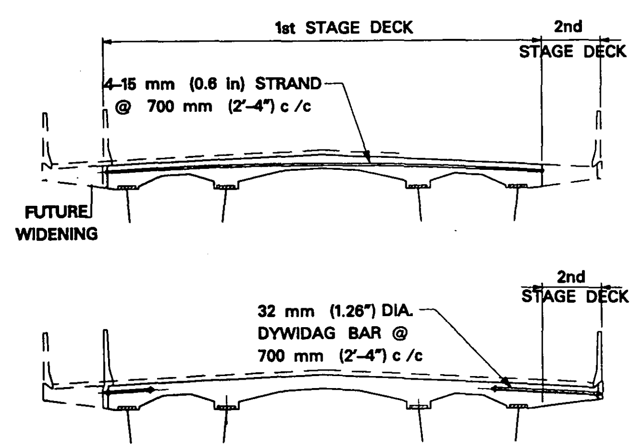

FIGURE 6 Posttensioning profiles: *top*, strand; *bottom*, threadbar.

TABLE 2 Key Data Used for Posttensioning Design

Datum	Value
Class of concrete	40 MPa (5800 psi)
Ultimate tensile strength of transverse strand	1860 MPA (270 ksi)
Ultimate tensile strength of transverse threadbar	1030 MPa (150 ksi)
Average transverse prestress in concrete deck	1.49 to 3.18 MPa (216 to 462 psi)
Maximum design tensile stress at SLS II[a]	1.52 MPa (220 psi)
Maximum design compression stress at SLS II[a]	15.98 MPa (2,320 psi)
Total weight of posttensioning steel	9.2 kg/m^2 (1.88 psf)
Total weight of reinforcing steel	46.0 kg/m^2 (9.42 psf)

[a]SLS II refers to Serviceability Limit State II as defined by the 1983 OHBDC. This limit state limits deformations and cracking.

advantages relative to the amount of reinforcing required for longer transverse spans.

These decks responses, and in particular the maximum design tensile stress, conform to the ASSHTO criteria of 1.57 MPa (228 psi) for severe exposure conditions. The amount of transverse reinforcing steel required for this design consisted of 20M bars at 225 mm (No. 6 bars at 8.5 in.) top and bottom.

In some cases, depending on the relative transverse spans, cross-frame stiffness and box girder proportions, transverse posttensioning effects can result in a significant redistribution of bearing reactions and axial forces in crossframes. The designer must account for these effects in the analysis and design.

Bracing, Framing, and Diaphragms

Several different elements of the framing and bracing systems are effective in distributing live and dead loads through the structure:

- Cross frames between boxes,
- Diaphragms inside and outside the boxes,
- Upper lateral bracing inside the boxes, and
- Distortion bracing within the boxes.

It is imperative that the designer take full account of the proposed bracing configurations and the relative stiffness of each for the distribution of both dead and live loads within the structure. In many cases dead load governs the design—particularly for larger spans, bigger boxes, and wide cross sections. Staged placement of the deck concrete can produce very different responses within the steel girder and bracing systems than the responses predicted by simultaneous placement of the deck. This factor becomes important as the spans increase over 60 m (200 ft) in conjunction with transverse eccentricity of the dead load relative to the shear center of the boxes.

One bracing option and associated details are shown schematically in Figure 7. This continuous transverse framing system is carried into each box and occurs at quarter points.

Nonprismatic Girders

Intermediate- and long-span bridge girders with the same approximate shape as the bending moment diagram use material

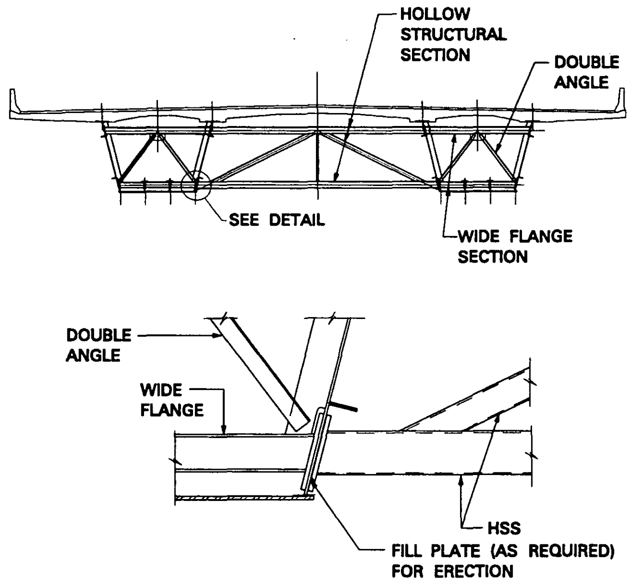

FIGURE 7 Crossframe (*top*) and detail (*bottom*).

most efficiently. In most cases it is possible to reduce, and in some cases even eliminate, changes in flange-plate thickness. This has the benefit of reducing fabrication costs associated with shop splices.

In addition to the above, haunched girders typically are reduced in depth in regions where there is reduced shear with associated material savings in the web.

Fountain originally predicted that a span-to-depth ratio (*L*/*d*) for box girders in the range of 25 would result in the most economical design. He also noted that box girders would conform to current AASHTO provisions for live load deflection with a span-to-depth ratio as high as 40. Typically in Ontario, noncomposite span-to-depth ratios are in the order of 30 to 32 and composite ratios are in the order of 28. Deeper girders have been used when incremental launching is proposed, to control cantilever deflections when necessary.

A nonprismatic girder can be proportioned in such a way as to require a single thickness of bottom flange throughout the entire bridge. This minimizes the need for bottom flange shop splices at thickness changes. For example, a three-span bridge with spans of 32 × 46 × 32 m (105 × 151 × 105 ft) was designed recently with a continuous 16-mm (⅝-in.) bottom flange plate throughout the entire bridge. The span-to-depth ratio at the piers was 27 and at midspan, 42. This resulted in a very economical steel box.

Use of a nonprismatic girder usually results in material savings when the design is properly executed. With the use of modern digitally controlled cutting machines there is little, if any, cost penalty in fabricating webs with variable depths. Webs are frequently cut to a profile for curvature or camber in any event.

Trapezoidal Section

The typical trapezoidal section used in many box girder bridges has the benefit of allowing the designer to provide the same amount of material required in the bottom flange for bending in a thicker plate. This increases the stability of the wider bottom flanges and reduces the need for longitudinal stiffening.

Typically the neutral axis goes up in a trapezoidal section and makes the bottom flange more efficient. This does not necessarily mitigate against the efficiency of the top flange because most of the load in the top flange is dead load due to composite action of the deck for live loads.

In addition, the trapezoidal shape is inherently more stable for fabrication and erection and reduces the fatigue problems arising from secondary vibrations.

Software

There is good news for the designers of larger and more complex steel box girder bridges. Software exists that enables the designer to model, analyze, and design for the specific responses of a complex box girder bridge of any configuration. If the designer had to list modeling, analysis, and design tools in order of importance, accurate bridge modeling would probably be the most important tool. The value of a software package that provides the designer with the tools for building a realistic yet practical model of his bridge for analysis cannot be overestimated.

DESIGN FOR CONSTRUCTION

Erection Methods

Designers should work closely with contractors for construction design. Specialist contracting experience exists that, if properly considered during design, can be effectively incorporated into the contract documents without eliminating effective competition, and can reduce construction costs.

Conventional Techniques

More and more structures are being constructed in urban areas where access is increasingly difficult because of built-up conditions and adjacent infrastructure. These conditions make the erection of steel girders more attractive where field sections can be placed quickly at night with limited impact on the traveling public. By contrast, falsework construction for concrete bridges can be expensive, time-consuming, and in many cases disruptive.

Reducing the number of boxes will generally result in fewer field sections. Fewer field sections means fewer field splices. Offsetting this advantage is the fact that the field sections will generally increase in weight. Field sections should be sized as a function of local fabricator and contractor capability. Fabricators and contractors normally prefer to fabricate sections that are a maximum of 45 tonnes (50 tons) but are capable of increasingly large sections, in some cases up to 105 tonnes (115 tons) or more.

Incremental Launching

Reducing the number of boxes is a direct advantage for steel bridges that must be incrementally launched.

In some cases, new infrastructure projects for "green field" locations are subject to increasingly onerous environmental restrictions. In other cases, existing transportation functions or site topography impose significant limiting criteria for conventional construction techniques. Mobilization of large cranes and delivery of field sections to environmentally sensitive or functionally difficult locations are specifically prohibited by owners and other agencies. Steel boxes are ideally suited for incrementally launching given their relatively light weight and inherent torsional stability (particularly on curved bridges), their reduced self-weight relative to concrete, and the fact that they can be fabricated off-site and off the critical path for construction.

One of the advantages to incremental launching of steel box girder bridges is that in most cases they can be launched over longer spans without the expensive intermediate piers and bents required for incrementally launched concrete bridges. In addition, steel boxes require very limited additional design effort or material to be added to the girder section for launching. Incrementally launched concrete structures must be designed specifically for the launching conditions because of the dead weight associated with concrete cantilevers. Concrete structures likewise are very sensitive to dimensional tolerances and elastic/inelastic deformations. Steel structures are forgiving and can experience large elastic deformations without distress. During launching, steel box girders are typically braced at the top flange, resulting in a quasiclosed section that is torsionally relatively flexible. This arrangement provides torsional as well as flexural flexibility during launching that assists in redistribution of launching reactions between webs.

Sliding: A Case Study

Steel bridges provide the opportunity for innovative construction techniques. The opportunities sometimes increase as the number of boxes is reduced.

The city of Trenton in Ontario, Canada, requested consultants to submit a technical proposal for replacing an existing swing bridge. The bridge spanned a busy recreational waterway on a major highway feeding the central business district of a popular tourist town. The owner specified several constraints:

- The new bridge was to be a fixed-span high-level crossing to replace the existing swing span, which was becoming more and more expensive to operate and maintain.
- The new bridge was to be located in the same position as the existing structure to avoid any property acquisition on the main street through town.

• The new bridge was to be constructed without any impact on existing traffic operations through the central business district.

• The new bridge was to be constructed without any impact on existing recreational boat traffic.

The solution consisted of a high-level fixed three-span box girder bridge constructed in a temporary location adjacent to the existing bridge. Construction was scheduled so the new bridge could be built around the seasonal operation of the existing swing span and then at off-peak hours be slid into its final position in the location of the existing bridge. The final bridge was 184 m (518 ft) long, carried two lanes of traffic and two sidewalks, and weighed 3000 tonnes (3,300 tons). It was slid into its final location in 3 hr 20 min.

Steel box girder technology provided the solution to construction over a busy waterway without interruption to recreational traffic and resulted in a structure light enough to be relocated easily in a short time.

DESIGN FOR REHABILITATION

How Long Do Bridges Live?

Several factors should increase the durability, lifespan, and feasibility of rehabilitation of steel girder bridges with a reduced number of boxes and longer posttensioned transverse deck spans. These include

- Increased thickness of deck,
- Controlled cracking resulting from posttensioning,
- Increased capacity to carry live load under partial deck demolition, and
- Staged deck replacement.

Recent statistics published by the Organization for Economic Cooperation and Development (*2*) on the average lifespan of bridges in Europe indicate that bridges are usually replaced for functional reasons before they reach their structural life expectancy. Life expectancies for a sample population of steel and concrete bridges using an a posteriori estimation of bridge life were reported as follows:

Bridge Material	*Estimated Life (years)*
Steel	63
Reinforced concrete	59
Prestressed concrete	33

This study indicates that steel bridges can be expected to last at least as long as concrete bridges. The bridge designer should be sensitive to measures that make the bridge more durable in the first place and that facilitate future rehabilitation.

What Components Die First?

Recent analysis of National Bridge Inventory (NBI) data in the United States (*3*) has indicated that the most common source of deficiencies in prestressed bridges is the concrete deck. This is no doubt true of steel bridges as well.

One of the significant advantages of a slab-on-girder type of structure is the fact that the deck can be replaced relatively simply compared to a concrete box girder bridge. Replacement can often be accomplished incrementally under live traffic conditions.

Other components of the superstructure can easily be replaced or repaired on steel girder bridges to extend their service life and increasing load-carrying capacity where required.

What To Do?

On the evidence that deck components are often the most common cause of deficiencies, two options could be considered: (a) design decks to last longer in the first instance, and (b) design decks as throwaway components to be replaced when they have reached the end of their service life.

Slightly thicker concrete decks associated with longer transverse spans that are transversely posttensioned should last longer than thin decks using conventional reinforced concrete.

Steel structures that can be designed in the manner described can be very attractive in the long term when total lifecycle costs are considered, especially if they are capable of significantly reducing negative impacts on existing traffic functions at the same time.

CONCLUSIONS

Reduce the number of boxes wherever possible in steel box girder bridges. This design principle will result in more efficient box girder bridges for both design and construction.

• The amount of steel can be reduced. This results in reducing fabricating cost, which is typically labor-intensive.

• Posttensioned decks are a cost-effective solution to longer transverse deck spans. Conventional posttensioning in the transverse direction can be introduced to enhance the strength and durability of longer transverse deck spans.

• Construction technology such as incremental launching is cost-effective in many structures. A reduced number of boxes lends itself well to this technique. Launching is a proven solution where site constraints mitigate against conventional erection techniques. Owners and designers should address the feasibility of incremental launching on a site-specific basis to satisfy specific environmental or infrastructure constraints.

Reducing the number of boxes in a bridge cross section will, in some cases, result in increased analytical effort. This could well be an advantage, as the increased understanding of steel box girder structures will result in more economical structures and further design developments that will further advance the state of the art for steel box girder bridges. This benefit should more than offset the additional design effort.

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