HIGH EFFICIENCY CABLE SUPPORTED BRIDGES

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ABSTRACT

Cable-supported bridges are the preferred structural form for long span bridges as truss bridges are becoming more of a solution of the past. Out of the different cable-supported bridge types, the cable-stayed form is becoming increasingly popular due to its economy and constructability advantages. The rapid advancement of this relatively new structural form is also the result of advances in analytical capabilities, construction technology and advances in manufacturing. In the last decade or so the industry has gained a considerable level of cumulative design experience with respect to cable supported bridges due to the multitude of projects involving such bridges over this period. At the same time, the advances in high strength materials and computing efficiencies have provided opportunities as never before for design refinement and optimization of long span bridges. The high level of structural redundancy inherent in these bridges also makes them amenable to analytically driven design refinements and optimization. While the discussion provided in the present paper has applications for all cable-supported bridges, it is more focused on cable-stayed bridges.

Key Words: Cable-supported, bridges, efficiency, optimization, high-performance, superstructure

1. INTRODUCTION – WHERE TO BEGIN?

The primary function of a bridge is to provide a structure that supports a roadway over an obstacle. However, as bridges are prominent public structures they are often required to be more than mere utilitarian. The resulting complex value system and the difficulty of quantifying cost vs. some of the benefit categories make a concise definition of efficiency rather difficult. However, with the premise that a solution based on form-following-function design and suitably refined would be naturally appealing, the efficiency of a bridge can be defined in terms of its cost per unit area of the usable roadway surface. This conventional measure appears to be the most logical quantifiable parameter in attempting to compare the different options and choices. Improving efficiency then is the same as reducing the major cost parameters for a given bridge, namely, optimization of material usage, improving constructability and reducing schedule. In other words:

1. Optimization of the structural system
2. Exploiting high-strength, high-performance materials
3. Improving constructability

A rational approach to improving efficiency requires us to identify the areas that provide the most potential for design refinement. Following table lists major components and their attributes for a typical cable-stayed bridge in the 1000 to 1500-ft span range. It represents a hypothetical bridge with a steel composite superstructure carrying 2 traffic lanes in each direction with standard shoulders and median. It is also assumed that the cable arrangement is such that the superstructure is not required to provide torsional stiffness to the global system.

Table 1: Attributes of a Typical Cable-Stayed Bridge

Design Element	Typical Cost Breakdown		Environmental Loads Above Foundation Level	
	$$	% Total	% Weight Contribution	% Exposed Area
Cables	$10mi	10%	4%	10%
Towers	$20mi	20%	6%	30%
Superstructure	$55mi	55%	90%	60%
Foundations	$15mi	15%		
Total	$100mi	100%		

As evident from the table the superstructure represents:

1. The majority of the bridge cost, estimated at 55%

2. Source of vast majority of the governing foundation loads: 90% of gravity (DL) and inertial loads (Seismic) and 60% of the exposed area for wind.

Thus the superstructure is the first logical candidate for design optimization as this has the most potential for direct cost savings. The reductions in weight and exposed area would have consequences reaching far beyond the immediate local benefits. The rising costs of construction material and environmental factors associated with the production, transportation and the reuse (or disposal) of construction materials are additional good reasons to minimize their use by maximizing design efficiency

2. OPTIMIZATION OF THE SUPERSTRUCTURE – THE GLOBAL SYSTEM

The function of the bridge superstructure is to provide a riding surface for the live loads. The superstructure must be designed to provide sufficient strength capacity against the internal forces developed as a result of these live loads (acting on top of the other permanent and transient load combinations). It must also satisfy certain global stiffness requirements to guard against excessive deflections and certain serviceability stress limitations to ensure durability. There are some key differences in the manner in witch the design demands are generated in the superstructure of a cable-supported bridge as opposed to a conventional girder bridge that must be addressed before proceeding further.

2.1 Conventional Vs. Cable-Supported

Conventional bridge superstructures carry self-weight and external loads applied to the superstructure to the supports in shear by spanning directly between the bearing locations. This is true for both girder and truss systems. The internal forces within the superstructure, member strains, and the resulting deflections, are due to the loads carried in shear to the supports. If the longitudinal structural elements of a conventional bridge were replaced by higher strength steel members (and correspondingly smaller member sections), the internal forces developed within the longitudinal structural elements due to the design loads are virtually unchanged[1]. However, the deflections under the loads (and the vibration periods) will be larger due to smaller section modulus (and the reduction in stiffness). In summary, the use of higher strength steels in conventional bridges produces more flexible superstructures, resulting in for example, larger deflections and longer vibration periods, but the design strength requirements remain virtually unchanged.

With cable-supported bridges (cable-stayed and suspension), the main longitudinal spanning mechanism is provided by the cables and towers. The contribution of the longitudinal girders to the global stiffness is generally small[2]. With cable-supported bridges, the flexibility of the towers and the cable system determines the global structural stiffness (and thus the deformations due to loads and the vibration periods). The governing internal forces within the longitudinal structural elements and the superstructure deflections are induced due to the global deformations (resulting from tower & cable flexibility). Thus, if one were to substitute a higher strength steel (and correspondingly smaller sections) for longitudinal structural elements, the global stiffness would remain virtually unchanged, resulting in no appreciable change in the deflections or vibration

[1] Neglects the slight difference in dead load due to reduction in steel volume

[2] With suspension bridges in the longer span range, the torsional stiffness of the longitudinal superstructure is key to its wind stability. The span length and the structural configuration of the suspension bridge considered in this case study is outside of this range for most wind climatic conditions

periods. However, the design strength requirements are reduced in proportion to the reduction in member stiffness. The following table summarizes these key differences in behavior:

Table 2: The general trend of using higher strength materials in longitudinal superstructure elements

Design Parameter	Bridge type	
	Conventional	Cable-Supported
Global Flexibility	Increases	No change
Deflections	Increases	No change
Vibration Periods	Increases	No change
Internal Forces	No change	Decreases
Design Strength Demand	No change	Decreases

Table 2 illustrates that cable-supported bridge types that use HPS have further benefits in terms of structural response than what can be realized in conventional bridges. The use of HPS provides a means of increasing capacity without increasing stiffness (higher f_y/E) and can provide tremendous advantages for long-span bridges as it can be used to reduce the girder design demand. At the present time, this is a factor not widely recognized within the design community. This issue can be easily illustrated by examining the dependency of LL design moments on the stiffness of the superstructure cross section (represented as depth in Figure 1 below):

Figure 1 shows that the LL bending moment induced in the superstructure is reduced by 85% (a six fold reduction) as its structural depth is reduced from 10-ft to 4-ft. There are other design considerations such as cable (or hanger) spacing and design requirements for cable (or hanger) loss that can dictate the minimum structural depth of the longitudinal girders. However, in a practical design, the minimum depth is not controlled by global stiffness requirements such as LL deflections or vibration periods as one may suspect initially.

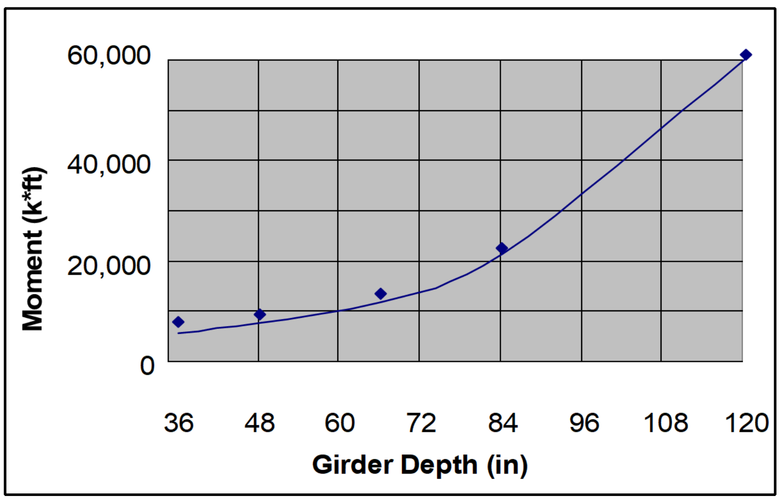


Figure 1: LL Design Moments Vs. Superstructure Girder Depth

2.2 Exploiting HPS Steels for Cable-Supported Bridge Applications

The development and wider availability of High-Strength High Performance Steels (HPS) has inspired a search to find effective methods of utilizing the high strength of these new materials. The benefits and the limitations of the use of HPS steel for conventional bridge types are well documented. However, the use of higher strength HPS for cable supported long span steel bridges is still emerging. The key differences in the behavior of cable-supported long span structures discussed above that make HPS an ideal material of application on these loner span structures. However these special advantages of HPS are yet to be fully recognized and exploited for these types of signature large scale structures. Further, these bridges also contain fabrication intensive elements such as cable anchorages and other similar elements. The fabrication-friendly properties of HPS can provide a significant advantage in producing more cost effective designs and improved bridge performance..

The potential use of HPS steels in cable-supported long-span bridges can be categorized into two major areas:

1. Specialty items such as cable anchorages and/or other high stress areas

2. Superstructure framing of steel-composite cable-supported bridges

Currently, the use of HPS on cable-supported long-span bridges is limited to specialty components or high stress areas (Item 1 above), and is relatively narrow in scope. These special applications are exemplified by the Charles River Bridge (Boston, MA, USA) where Grade 70 HPS was used in cable anchorages and in the steel-composite section of the tower[3]. The primary reason for the use of Grade 70 HPS in this application was the reduction in plate thicknesses for cable anchorage components and steel inner core of the composite tower design. The reduction in plate thickness was found to be beneficial for ease of fabrication, especially the rolling of tube sections used for stay cable anchors. Also, thinner plates reduced the lift weights of the tower core sections. The use of higher strength resulted in a proportional reduction in the tonnage (for example 100 HPS would save 50% of material over Grade 50) as the details were developed in such a way that local plate stability did not govern the design.

As demonstrated later using a case-study, the bridge superstructure design (Item 2 above) offers a promising application of HPS steel. For transverse elements such as floor beams, advantages would be similar to the use of HPS on conventional bridges. The transverse span, the depth of the floor beams and deflection criteria would dictate the economics of using HPS for the transverse elements. On the other hand, ad outlined before, the main longitudinal girders of cable-supported bridges behave quite differently from those in conventional bridges where a reduction is girder stiffness would in-turn result in a reduction in design demand. It is these differences that provide considerable opportunities to take advantage of higher strengths. Key issues that govern the overall design of the longitudinal elements of a bridge superstructure can be categorized as:

1. Strength: Providing sufficient strength to resist load effects produced within the superstructure in the longitudinal direction

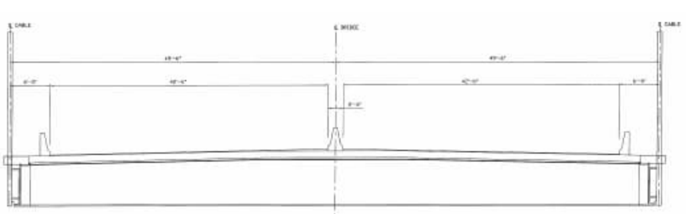
2. Serviceability & Stability: Assessments based on limiting deflections and/or modal frequencies

3. Other Considerations: Includes fatigue (and effects of cable/hanger loss or exchange for cable-supported bridges)

Fatigue does not typically govern the longitudinal girder design. The stresses due to cable/hanger loss can be controlled by using a hanger/cable spacing suited to the selected girder depth.

2.3 Parametric Study

To illustrate the potential for design optimization of cable-supported bridge superstructures using HPS, the cable-stayed and suspension bridges shown in Figures 2 and 3 were used in this case study. They have a main span length of 1250-ft with two symmetrical back spans of 550-ft each. The superstructure cross section consists of two traffic lanes with inner and outer shoulders in each direction. The superstructure framing consists of two longitudinal girders and a series of floor beams with a composite concrete deck (Figure 2). The center-to-center distance between the longitudinal edge girders is 92-ft, and the total weight of the superstructure was taken as 25 kips/ft.



[3] Zakim I ... International (IABSE) Vol 13, number 13.

Figure 2: Superstructure Cross Section
The bridges were designed for HS-25 live load and impact with a distribution factor of 2.5 traffic lanes (corresponding to 6 lanes shifted and applied at 75%). The two bridge layouts are shown in Figure 2.

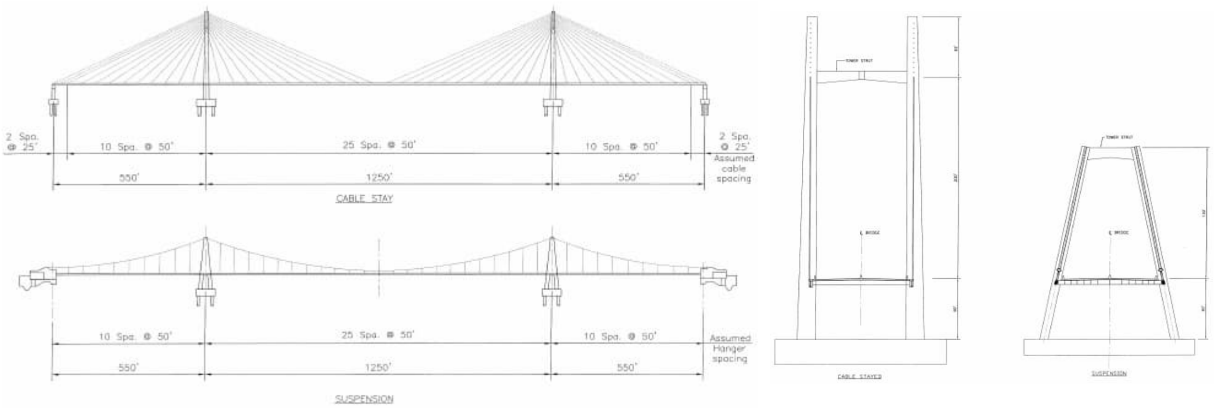
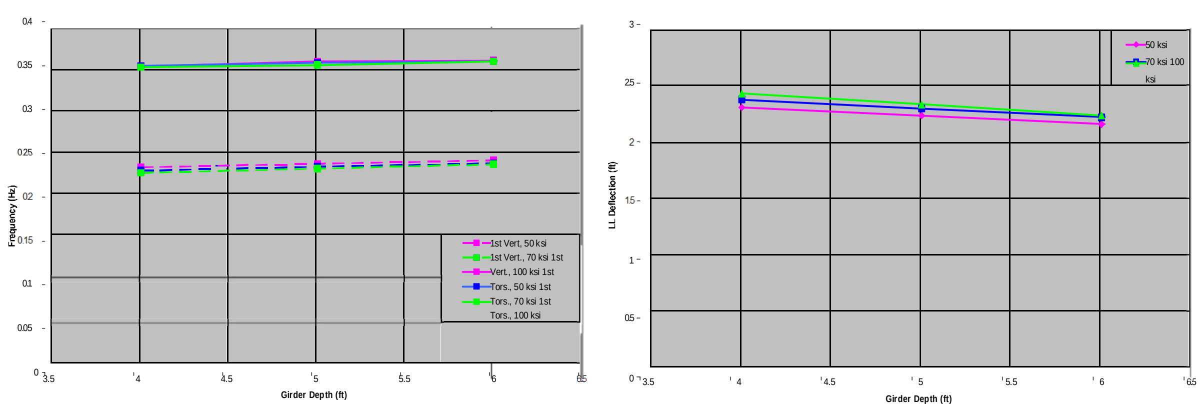


Figure 3: Case Study Bridge Layouts

The cable-stayed bridge is based on a typical H-tower configuration, and the suspension bridge is based on an A-tower configuration to increase the global torsional stiffness and to achieve the same range of torsional frequencies as the cable-stayed (Figure 3).

The case study bridges were analyzed and the superstructure longitudinal girders were designed using different steel grades. The DL was assumed unchanged and new live load envelopes, deflections and dynamic characteristics were obtained for each variation considered. As girder strength steels enable the use of shallower girder sections, the girder depths were also varied to take the best advantage of the higher strength grades. The longitudinal girder was designed along the length of the bridge to meet the AASHTO Group I load combination of 1.3DL+2.17*(LL+I). The resulting girder section properties were reinserted into the analysis model to obtain new live load demands. This process was repeated until the design was sufficiently refined.



(a) 1st Vertical Modal Frequencies

(b) LL Deflections

Figure 4: Variation of Global Stiffness with Structural Depth of Girders (Cable-Stayed Bridge)

It can be seen that the girder depth and the material grade (and thus the superstructure stiffness) does not have a significant influence of the global bridge stiffness. It must be noted that the magnitude of the LL deflections and dynamic characteristics are dependent on the tower stiffness selected, but the general trend is the same. Figure 6 shows variation in girder steel requirements for the cable stayed bridge option with varying girder depth and grade of steel

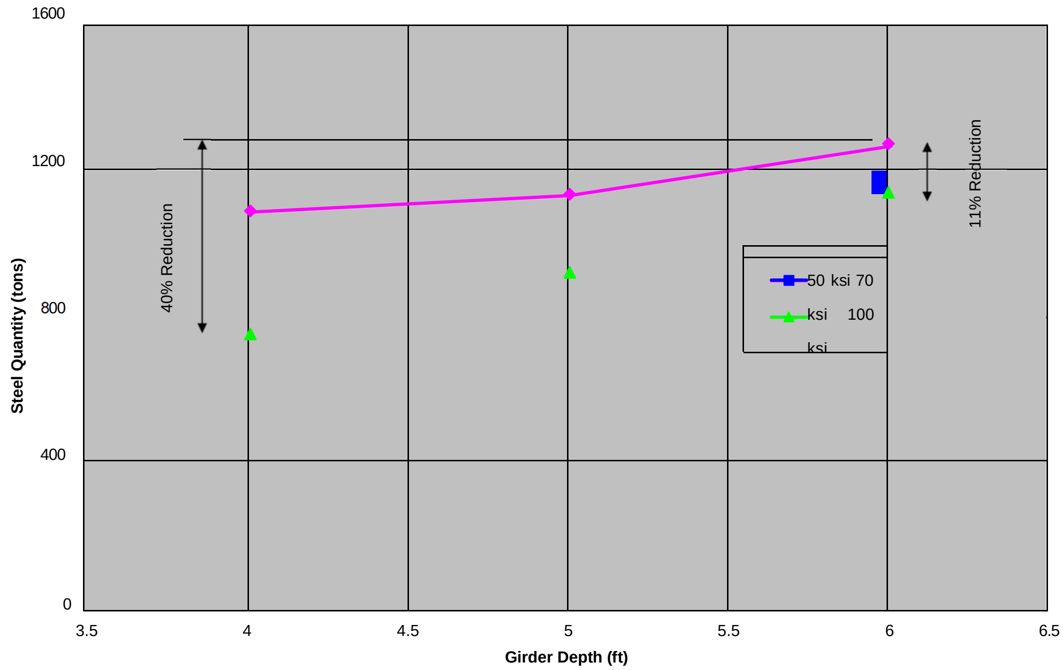


Figure 5: Variation in Girder Steel Requirement with Material Grade and Girder Depth

The total steel quantity in the longitudinal girder shown in Figure 5 must be considered theoretical quantities. For example, the use of grade 50 steel and 48" girder depth may result in excessively large flanges may not be the best choice in a real life design.

3. LOCAL SUPERSTRUCTURE DESIGN & DETAILING

Conventional steel framing for composite superstructures consist of longitudinal edge girders, transverse floor beams and a central longitudinal strut beam that support the longitudinal CIP joint between the pre-cast slab panels. The pre-cast slab is conventionally designed to span between floor beams. The key factors that define the structural system are the cable spacing, floor beam spacing and the thickness of the deck slab. These key parameters can be examined systematically to enhance the structural efficiency of the superstructure. The slab thickness and the floor beam spacing is selected considering the stability of the lab under axial loads generated within the cable-stayed superstructure. The span to thickness is normally kept below 17, giving a floor beam spacing of 16-ft for an 11-inch thick slab.

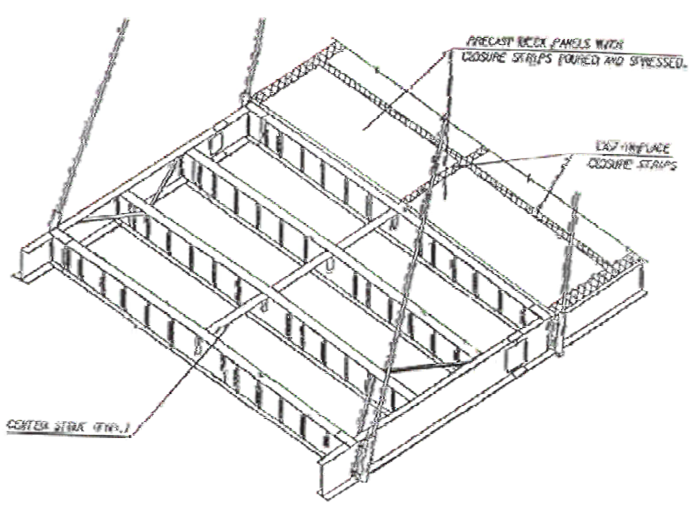
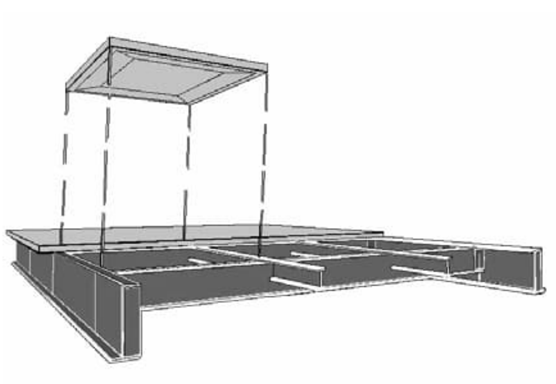


Figure 6: Conventional Superstructure Layout (Steel-Composite)

The concrete slab is longitudinally post-tensioned so the service level tensile stresses at the top of slab do not exceed a given limit (normally $3\sqrt{f'_c}$) under DL and LL+**I**. This limitation of tensile slab stresses is a key issue as it could be related to the maintainability of the deck slab and the useful life expectancy of the bridge.

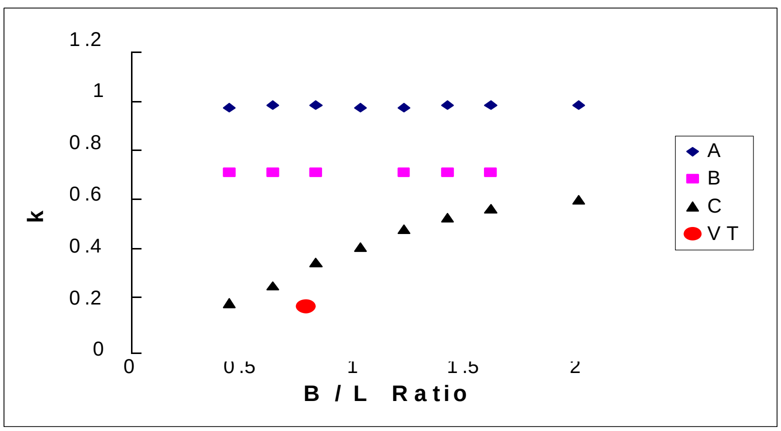


The following changes to the conventional design can be used to increase the floor beam spacing to the 25-ft range, reduce the effective slab thickness (the weight) by about 15% and significantly improve the service level tensile stresses at the top of the deck slab.

1. Use of heavier strut beams that can provide support to the slab between floor beams
2. Variable-thickness, two way spanning deck slab

Figure 7: Optimized Steel-Composite Superstructure

Two the two-way spanning pre-cast slab panels between the grid formed by the edge girders, floor beams and the longitudinal strut beams are considerably more stable than the conventional one-way spanning slabs as seen from Figure 8.



A) One-way simple span
B) One-way continuous
C) Two-way continuous
VT) Variable Thickness

Figure 8: Effective span length parameter k for different support conditions

It can be seen that the effective span parameter k for a two-way slab is considerably lower than for a continuous slab for width/span ratios (B/L) < 1 and that the variable thickness deck slab (VT) has even superior stability. The two-way action also significantly improves the live load carrying capacity of the deck slab. The structural efficiency of the deck slab is further enhanced by using a variable thickness configuration. The variable thickness is obtained by using a bottom pan made to shape. The variable thickness deck panels improve the structural efficiency by:

i. Providing a minimum weight design
ii. Slab is thickened over the areas of negative moments and higher shear demand due to live loads (thereby reducing the concrete shear stresses and tensile stresses on the top surface of the slab to acceptable levels)
iii. The longitudinal and transverse post-tensioning over the thickened negative moment areas of the slab also produces an eccentric pre-compression moment that increase the level of pre-compression at the top of slab, significantly increasing the effectiveness of the post-tensioning in controlling tensile stresses due to live load

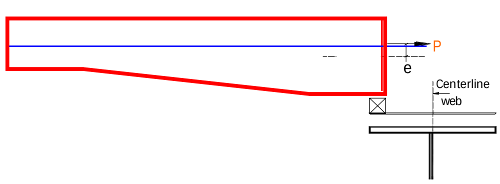
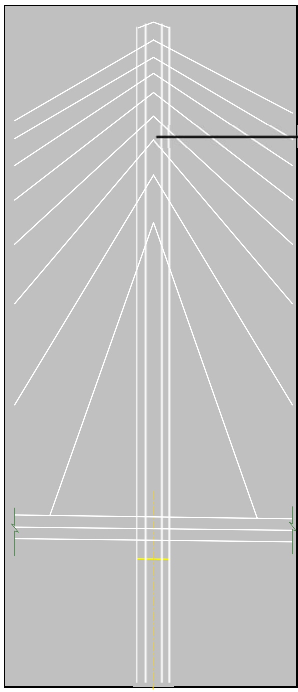


Figure 9: Variable thickness pre-cast slab panels
This allows a slab design that eliminates tensile stresses under LL+I without increasing deck post-tensioning requirements, a significant improvement in deck slab serviceability and maintainability.

4. IMPROVING CONSTRUCTABILITY – TOWER CABLE ANCHORAGES

A conventional tower and cable layout shown in Figure 10 involves cable penetrations that extend a relatively large portion of the upper tower length and transverse tower leg post-tensioning to contain the tensile stresses induced by anchoring of the cables.



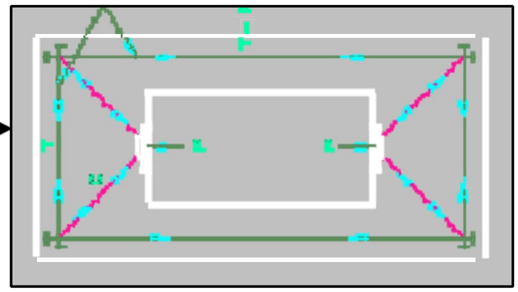


Figure 10: Conventional Tower & Cable Anchorage Details

It is possible to use a compact cable anchorage weldment as shown in Figure 11 that not only allows cables to be anchored at a half the regular spacing improves tower constructability significantly by:

1. Eliminating interruptions to tower construction due to cable penetrations and tower post-tensioning

2. Eliminating complex geometry adjustments in the field by use of shop-fabricated cable anchorage

The key feature of the newly developed steel cable anchorage is that it resolves tensions from the cables from the opposite sides of the tower as shear in a girder web without producing any tension. Due to the compactness of the cable anchorage (mounted at the top of the tower), cable angles to the horizontal are increased there by improving efficiency of the cable system as well.

Figure 11: Compact Tower Cable Anchorage

4. CONCLUSIONS

1. Long-span cable-supported bridges offer engineers excellent opportunities to design more efficient structures through the use of HPS steel.

    a. HPS may be used to improve the performance and constructability of special highly stressed details such as cable anchorages. This has been shown effective in previous projects including the Charles River Bridge in Boston, Massachusetts. When detailed appropriately, higher strength steel results in a proportional decrease in plate sizes for these special elements, resulting in easier fabrication and lighter sections.

    b. The inherent structural behavior of long-span cable-supported bridges offers the engineer the opportunity to take advantage of the higher strength of HPS steel, without the consequences that have been experienced in conventional girder HPS bridges, such as fatigue concerns, increased deflections, and increased vibration period. Using smaller sections, the bending moments in the main longitudinal edge girders are reduced. This, coupled with the higher strength of HPS steel, results in a combination that offers excellent potential for cost savings and improved constructability for long-span steel bridges.

2. The use of two-way acting variable thickness pre-cast deck panels can be used to reduce the weight of the deck slab, increase floor-beam spacing significantly and improve the service level tensile stresses at the top of the deck slab

3. The compact tower cable anchorage developed facilitates a significant improvement on tower constructability while minimizing the size of the steel assembly.

4. The design refinements discussed presently could be used produce:

    a. 15% reduction in the weight of the overall superstructure

    b. 40% reduction in the use of superstructure structural steel

    c. 20% reduction in the exposed are of the superstructure

    d. 75% reduction in the weight of the tower steel anchorage in comparison to a conventional tower box-type anchorage.