



## Optimisation of large bridges

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### Abstract

Optimisation is a challenging term nowadays, as it may refer to various aspects of a large bridge, such as construction cost, ease of construction, future maintenance, architectural expression and CO<sub>2</sub> footprint. Using additional quantities may sometimes lead to construction simplicity — for example, by employing a larger number of identical elements instead of introducing individual variations. While the material itself has a direct cost and CO<sub>2</sub> footprint, there is also an indirect cost associated with carrying its weight, both in terms of construction expenses and environmental impact. This article addresses these issues through examples from existing large bridges, such as the Øresund Bridge (Denmark–Sweden) and the Great Belt East Bridge and West Bridge (Denmark). The review highlights the compromises made in these structures, each designed according to its own optimization objectives, and revisits their designs in light of the most recent knowledge regarding CO<sub>2</sub> impact.

**Keywords:** continuous girders, haunched girder, long span, link-slab, two-level superstructure, cantilever, light-weight material, jacking, carbon footprint

### 1 Introduction

The article discusses the most important design drivers related to the optimisation of quantities, cost, and CO<sub>2</sub> footprint in large bridges:

- Span versus direct support
- Very long spans
- Viaduct bridges: continuous girders versus simply supported spans, haunched versus constant depth
- Tall structures
- Integral design or separate structures?

Optimising quantity means ensuring that all bridge components are fully utilised in accordance with the applicable load and safety requirements. The construction cost of a bridge consists primarily of the material quantity, plus a secondary lump-sum portion; therefore, cost optimisation largely coincides with quantity optimisation. The CO<sub>2</sub>

footprint depends on the choice of bridge material but is otherwise proportional to the material quantity. Since the choice of material with respect to CO<sub>2</sub> reduction is not the focus of this article, the discussion will mainly consider the proportional relationship between CO<sub>2</sub> emissions and quantity.

Hence, the broader term optimisation used throughout this article will typically refer to all three aspects — quantity, cost, and CO<sub>2</sub> — which generally go hand in hand.



Figure 1. The Great Belt East Bridge [1], Denmark



The Great Belt East Bridge (figure 1), with its 1624 m suspended main span, will be used to illustrate the optimisation potential of a very long span. Viaduct bridges with beam behaviour across few or many supports will also be discussed, with the Great Belt West Bridge (figure 2) serving as a case study.



Figure 2. The Great Belt West Bridge, Denmark, under construction [2]

The optimisation of very tall structures will be exemplified by the Sutong Bridge pylons.

Should a structure be designed to accommodate all types of load effects within the same system, or is it preferable to separate structural functions and design individual elements for specific purposes? Each approach has its advantages and disadvantages.

A common theme among these optimisation challenges is the position (or distance) of the structural material relative to its foundation — whether it lies close to or far from its supports. As will be concluded, optimisation may ultimately be reduced to a single key question:

- *Why is the structural material located so far from its support, and can this be justified?*

## 2 Reference bridge projects

The article refers to the following bridge projects:

1. Great Belt East Bridge (Figure 1), [1]
2. Great Belt West Bridge (Figure 2), [2]
3. The Øresund Bridge (Figure 18), [3]
4. Hålogaland Bridge, Norway (Figure 4), [4]

5. Raftsundet Bridge, Norway (Figure 8), [5]
6. Canakkale Bridge, Turkey [6]
7. Messina Bridge (project), Italy [7]
8. Qatar-Bahrain Causeway (project) (figure 16) [8]
9. Sutong Bridge, China (figure 15) [9]

## 3 Direct support or span?

The very purpose of a bridge is to span across an obstacle, so for a bridge engineer, it feels almost counterintuitive to suggest otherwise. However, if the sole objective were to minimize CO<sub>2</sub> emissions, there is considerable truth in this notion. A cantilevered deck slab can transfer a truck load from the edge of a bridge to the centroid of the main girder, from where the girder carries it to the supports at either end. Yet, this efficiency comes at the cost of material quantity — a “bill” that must ultimately be paid. While slender structures, long spans, and large cantilevers are deeply embedded in the DNA of engineers and architects, it must be emphasized that the most efficient way to carry a load is, quite simply, to place a direct support beneath it.



Figure 3. Copenhagen Opera House

The large cantilevered roof of the Copenhagen Opera House (Figure 3) serves as an illustrative example. If slender columns had been introduced beneath the cantilever — for instance, at the quarter point measured from the tip — the bending moments could have been reduced to merely 10% of those in the fully cantilevered configuration.

## 4 Optimisation of a long span

What are the best optimisation strategies for a bridge with a very long span?

The three main objectives are:



- to reduce the span length
- to reduce the weight of the span
- to reduce the weight within the span where it matters most

#### 4.1 Reduce the span length

From Cable Supported Bridges [10] section 3.2.7 the following is stated about the size effect:

*“It is a well-known fact that the quantities to go into a structure of a given type will increase progressively with the span length due to the ever increasing contribution from the self weight of the structural members”*

The cable quantity for a major suspension bridge can be estimated as being proportional to the square of the main span length ( $L^2$ ), where  $L$  is the span. Since the main cables of large suspension bridges already represent a significant portion of the total cost, it is evident that reducing the span length—and thereby material quantities, CO<sub>2</sub> emissions, and construction costs—is of critical importance.



Figure 4. Hålogaland Bridge, Norway  
(COWI/DW Architecture)

For example, the Hålogaland Bridge (Figure 4) was initially proposed with a suspended main span exceeding 1,400 m, with towers located on either side of the fjord. In the final design, the span was reduced to 1,145 m (approximately 82% of the original length). Despite the additional cost of constructing offshore tower foundations, this optimization led to a net reduction in total project cost.

#### 4.2 Reduce the span weight

The second most important thing is to reduce the weight of the suspended span. The longer the span,

the more crucial this obligation becomes. Some examples to illustrate the potential.

##### 4.2.1 Main cable steel grade

It is evident that using the highest available grade of cable steel is an option worth examining. However, for practical reasons, suspension bridge cables are often made from slightly lower-grade steel. For the Messina Bridge, increasing the cable steel grade from 1,860 MPa to 1,960 MPa resulted in a 9% reduction in cable quantity [7]. The 1,960 MPa grade now represents the current industry standard, as used for example in the Çanakkale Bridge in Turkey [6].

By comparison, the Great Belt Bridge employs 1,570 MPa grade wires. Hypothetically upgrading these to 1,960 MPa would yield a significant cost reduction — estimated at approximately €5 million.

##### 4.2.2 Inspection walkways and wind fairings

Along the outer edges of a suspension bridge, inspection walkways are often provided. For large-span suspension bridges, to ensure optimal aeroelastic performance, including high critical wind speed resistance (flutter) the bridge deck will require a tailored geometric shape. This can be achieved by shaping the main structure to meet wind demands.

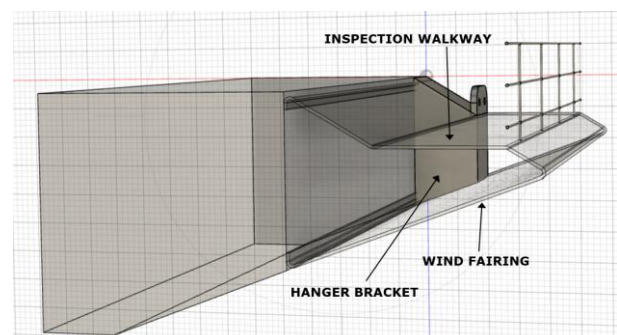


Figure 5. Inspection walkway and wind fairing

An alternative option involves a composite glass-fibre sandwich deck (Figure 5). This sandwich plate weighs about 15 kg/m<sup>2</sup>. The total saving amounts to €68 million for a 3,000m suspended span case study, refer to [11].



#### 4.2.3 Perforated steel trough

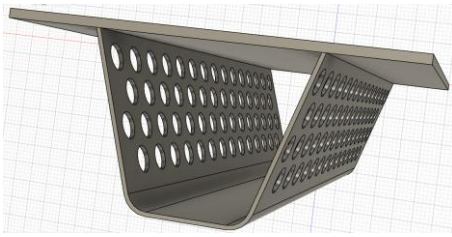


Figure 6. Perforated steel trough

The orthotropic steel deck offers numerous well-established advantages with few drawbacks. The closed troughs are a crucial component of these benefits; however, the webs may provide more shear strength than necessary. The solution is to perforate the webs—reducing, for example, one-third of the web material—without significantly affecting the overall performance. Including the cost of punching these holes estimated at €2 – 4 million, the total cost saving amounts to €15 million for the 3000m suspension span case study [11]

#### 4.2.4 Central clamp



Figure 7. Great Belt Bridge, central clamp

Some bridges feature central clamps that connect the lowest point of the cable to the deck at midspan (figure 7). These provide various benefits, but a significant drawback for a long, suspended span is that they introduce additional loads into the deck structure, necessitating extra steel material, and at the mid span where weight limitation is of utmost importance. For the Great Belt Bridge, this additional steel amounts to approximately 260 tons. While the direct cost of this may be around €2 million, if a similar quantity were required at the midpoint of a 1,624m span, the resulting indirect (additional) would be €6 million (rough estimate).

#### 4.3 Reduce the weight (within the span) where it matters most



Figure 8. Raftsundet Bridge, Norway [6]  
 (photo: broer.no)

The Raftsundet Bridge (Figure 8) features a 298 m haunched girder span, with a very shallow section at mid-span compensated by significantly deeper sections at the piers. Not only do the visible dimensions vary substantially, but the slab and wall thicknesses also change — being thinnest at mid-span and increasing progressively towards the piers.

This arrangement is intuitively logical: the majority of material is concentrated near the supports, where bending moments and shear forces are greatest, while as little material as possible is placed at mid-span, where these effects are smaller. The concept exemplifies efficient material use in structural design and will be further illustrated in the following sections on beam behaviour.

### 5 Viaduct bridges

In this section, two objectives for reducing material quantities will be examined:

- Variable-height versus constant-height sections
- Continuous versus simply supported spans

All explanations refer to the bending moment diagram of a fixed-end beam (Figure 9).

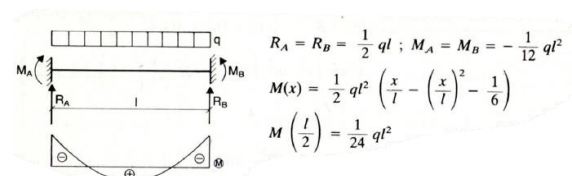


Figure 9. Fixed end beam moment



It is observed that the bending moment at the supports (“hogging moment”) is approximately twice the bending moment at mid-span (“sagging moment”). The points of contraflexure are located at roughly  $x = 0.21 L$ .

It is also worth noting that the sum of the bending moment at the support and the bending moment at mid-span numerically equals

$$M = \frac{1}{12} qL^2 + \frac{1}{24} qL^2 = \frac{1}{8} qL^2$$

which is the same as the sagging moment at mid-span of a simply supported beam (Figure 10).

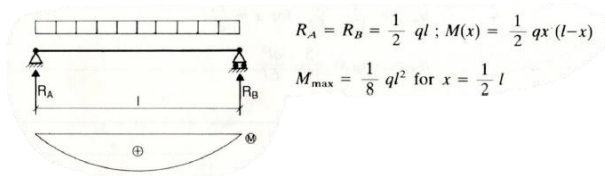


Figure 10. Simply supported beam moment

A beam of length  $L$  with a single support at its midpoint will experience a hogging moment of

$$M = \frac{1}{8} qL^2,$$

which is numerically the same as the bending moment at mid-span of a simply supported beam.

Multiple-span viaduct bridges are hyperstatic (statically indeterminate) structures. This gives the designer the freedom to achieve a desired bending moment distribution along the span. The bending moment curve should be designed so that the natural cross-sectional capacities of the bridge girder are fully utilised at every section.

For the dead load, the extremes occur either with zero moments at the supports (simply supported) or with zero moments at mid-span. In contrast, the live load bending moments depend on the structural response and cannot be directly controlled by the designer.

The built-in bending moments of a continuous viaduct bridge also depend on the construction method, unless additional interventions are carried out. Construction on scaffolding supported from the ground (rare for large bridges) produces fixed-end bending moments. Floating installation of full

spans placed on pier supports at either end produces sagging moments at mid-span (Figure 10). If the bridge girders are placed on the piers with the two halves in perfect balance, the result is zero moment at mid-span and full hogging moments at the piers. Additional interventions can, however, modify the natural bending curve to the desired distribution; this method is referred to as “jacking” in this article.

### 5.1 Variable or constant height section?

For haunched girder spans, such as the Raftsundet Bridge, both the weight and the beam stiffness increase toward the piers, further enhancing the tendency for hogging moments at the supports to increase while mid-span moments decrease.

Since the sagging moments over the spans act over a longer length than the hogging moments at the piers, the optimisation strategy is often to increase the hogging moment at the piers—i.e., over a relatively short length—thereby reducing the sagging moment over a longer portion of the span. This approach leads to a total material and cost saving.

### 5.2 Great Belt West Bridge

The tender for the Great Belt West Bridge included two options: either constant-depth bridge girders side by side for road and rail traffic, or a two-level superstructure similar to the Øresund Bridge.

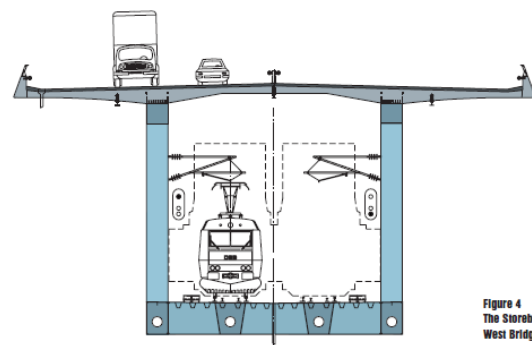


Figure 11. Great Belt West Bridge tender

The winning contractor, however, proposed a haunched girder concept that was well-suited to the floating installation crane Svanen (Figure 2). The support conditions during installation closely



mimicked those of the final state, which is normally very favourable for the overall economy of the bridge.

Hunched girders are capable of spanning longer than constant-depth girders, as the additional weight is concentrated near the supports, rather than at mid-span, where it would be structurally disadvantageous (as explained in Section 4.3). This deliberate choice resulted in a shallow mid-span section, with relatively little moment capacity; however, the acting moments at mid-span are similarly small, so the section is fully utilised. The deep sections at the piers carry the majority of the load and are also 100% utilised and optimised.

In conclusion, there is no “waste” of structural material: the material is placed where it matters most, achieving maximum efficiency.

### 5.3 Great Belt East Bridge

The Great Belt East Bridge viaduct spans consist of 193 m continuous closed steel box girders [1], which offer numerous merits. The tender design initially proposed 160 m spans using steel grade S355, whereas the contractor proposed increasing the span to 193 m and using steel grade S420, with the span increase roughly proportional to the higher steel grade. Since the vertical load on each pier is not overly demanding and ship collision loads must also be considered, this increase in steel grade led to a significant reduction in overall material quantities due to the fewer required piers.

For the installation of the 193 m spans, the far end was lifted by a floating crane and temporarily supported in an elevated position (Figure 12).



Figure 12. Great Belt East Bridge, placing of span

After continuity welding with the previous span, the section was lowered, thereby introducing the desired hogging moment over the pier.

The principle of jacking is illustrated in Figure 13. Its advantage lies in introducing a significant hogging moment at the pier, which would otherwise experience only a minimal load effect (i.e., the material would not be fully utilised).

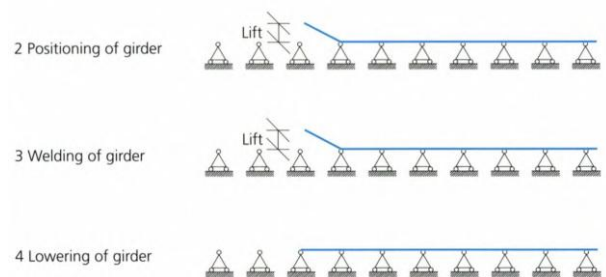


Figure 13. Great Belt East Bridge, jacking principle

This hogging moment at the pier reduces the sagging moment over longer lengths of the span in approximately a 1:1 ratio, leading to an overall reduction in material quantities.

An example where the benefit of continuity may be questioned is the suspended bridge deck spanning between the pylon legs of a suspension bridge (Figure 1). In this case, continuity induces additional hogging moments from live loads, requiring locally around 500 tons of extra steel.

### 5.4 The Øresund Bridge viaducts



Figure 14. Erection of Øresund Bridge viaduct span

When spans were lifted by Svanen (figure 14), the massive 10,000 tonnes lifting vessel previously employed on the Storebælt West Bridge project, the girders cantilevered 40m at either end. This



induced significant tension in the upper concrete deck, a load case that would not otherwise occur in situ. The girders were assembled with the concrete decks cast onshore in full-span lengths, which meant that the decks would be almost wholly in longitudinal compression when placed simply supported on the piers, a desirable condition for the concrete elements.

As most of the railway deck is in the tension flange of the girder, the target was to limit the large tension forces that would normally have arisen.

The girders were thus erected with only parts of the railway deck in place, leaving out the concrete / steel connections, which were done in situ when the girders were in their final position on piers.

The original erection scheme agreed with the Contractor assumed that the concrete railway elements would be installed on a continuous girder, and the detailed design documentation was based on this assumption.

Once located on the pier tops, the free end was temporarily at some 0.3m above final position (figure 16). After continuity welding of the steel truss at the other end of span, the free end was lowered into place, introducing a negative moment over the pier which exactly compensated for the modified erection sequence of the railway containments.

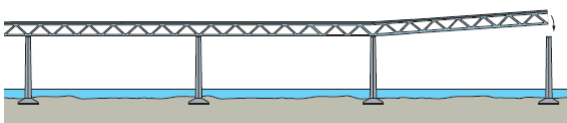


Figure 15. Øresund Bridge viaduct span, jacking

It is worth noting that the hogging moment built into the piers is, after all, relatively small. The concrete roadway deck slab has secondary tensile capacity compared with the lower steel chords at mid-span, but it still contributes to reducing the large sagging moment over the span, thereby helping to minimise structural steel consumption.

### 5.5 Link-slab bridge girders

A continuous bridge solution that is effectively simply supported on the piers is referred to as a

link-slab bridge. This concept was developed, for example, for a causeway connection between Qatar and Bahrain in 2009 (COWI / Vinci / Hochtief) (figure 16). The design was based on mass fabrication of large elements, followed by a minimum of in-situ works.

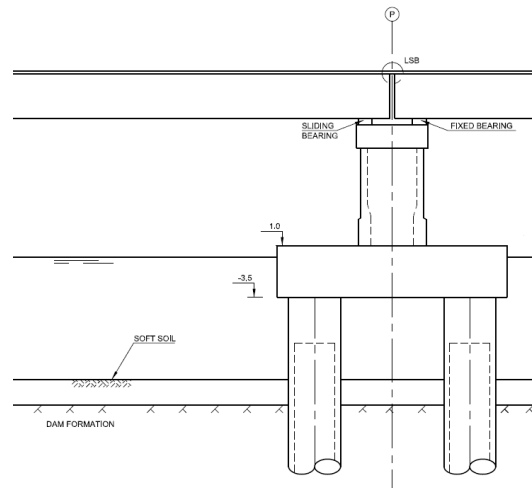


Figure 16. Qatar-Bahrain Causeway Bridge, link slab

In this project, the spans were only 50 m long, and the link-slab concept was preferred over an alternative with continuous girders at 80 m spans. The decision was influenced by the available equipment, which suited the 50 m elements, and by the fact that the 80 m alternative would have required substantially more in-situ work, such as pulling all the post-tensioning (PT) cables for continuity after in-situ stitching of the girders.

Although the concept is very practical, it may use slightly more material than continuous girder spans due to the reduced benefit of hogging moments at the piers. On the other hand, regarding PT-cable optimisation, each cable in the girder can be precisely allocated to match the exact bending moment curve of a simply supported beam. In this sense, some optimisation is still achievable.

The concept has also been applied to a 200 m long Maasned Sund railway bridge in Denmark [12].

### 5.6 Gibraltar bridge study

The conceptual idea of the Øresund Bridge's truss-type composite girder was also applied to the viaduct spans in the Gibraltar Bridge Study (COWI /



CFC / D+W) (Figure 17). The design included viaduct spans with 300 m spacing between the piers.

Because of the great water depth, the viaducts required 300 m spans. It was therefore considered optimal to use a very deep girder pipe with a single bottom flange in the form of a steel pipe.

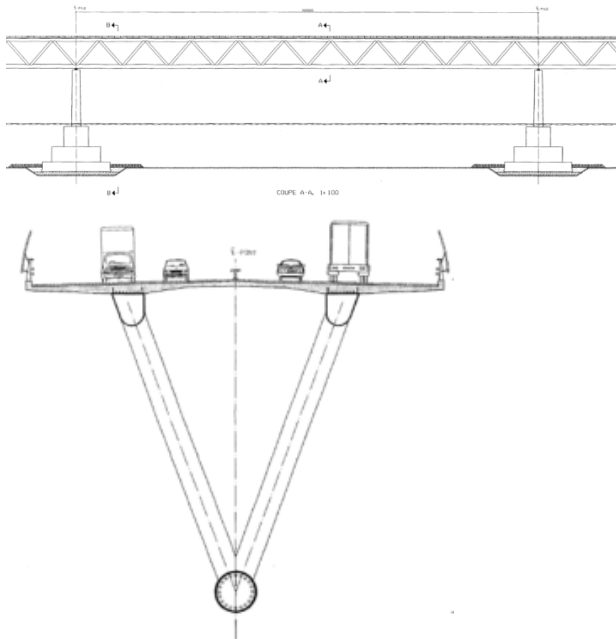


Figure 17. Gibraltar Bridge (study), viaduct

The steel pipe contained internal post-tensioning (PT) cables, which initially pre-compressed the pipe. However, once the bridge span was positioned in its final alignment, the stresses in the pipe changed from compression to tension. This highly sophisticated principle effectively doubled the tensile capacity of the lower pipe — an idea developed by Georg Haas, Chief Engineer at COWI.

Due to the extreme span lengths, jacking operations were not considered feasible. The upper concrete flange was designed to avoid significant tensile forces from dead load. Nevertheless, with its very deep and open truss configuration and the efficient local transverse bearing of the concrete slab, the concept is regarded as highly efficient, achieving minimal material usage under exceptionally demanding environmental conditions.

## 6 Integrated or separated design?

It is an important discussion whether the materials of a bridge should participate in more load-demanding effects (an integral design approach), or whether it is preferable to design structural components for individual, distinct purposes.

Before the advent of finite element method (FEM) modelling, the complexity of calculating hyperstatic systems meant that independent, non-integrated design solutions were the norm. For instance, deck slabs were typically not integrated with the main structure, which itself was simply supported on piers and abutments.

With the introduction of FEM, along with a growing emphasis on structural finish, uniformity, robustness, and redundant load paths, modern structures increasingly rely on a high degree of integral behaviour. As an example, consider the box girder segment located between the pylon legs of the Great Belt Bridge. This structure serves multiple functions: it carries local loads from vehicles, provides global stiffness, distributes loads along its length, ensures torsional stability of the main span, and transmits horizontal forces from crosswinds to the pylons. These functions are not all active simultaneously, and appropriate load combinations must therefore be applied.

The author of this article recommends that the initial design approach should prioritise an integral design philosophy as the most efficient starting point. As the design develops and more information becomes available, it may prove beneficial to relax certain constraints or simplify local connections where appropriate.

A major advantage of the integral design principle is that not all load effects occur simultaneously. From a practical perspective, it is advisable during the design process to begin with the primary loads and effects, temporarily neglecting secondary ones. These can be introduced progressively as the design matures, verifying that they do not significantly increase material quantities.

It should also be noted that some effects are proportional to the material quantity itself. For instance, minimum reinforcement requirements



for concrete structures increase with volume—the larger the concrete section, the greater the minimum required reinforcement. Similarly, in seismic design, load effects are often proportional to the structural weight. Therefore, simply adding material does not necessarily mitigate these effects and may even be counterproductive.

## 7 The Øresund Bridge viaducts

The tender for the Øresund Bridge included two alternative bridge solutions: a one-level design with side-by-side traffic and a two-level configuration (Figure 18).

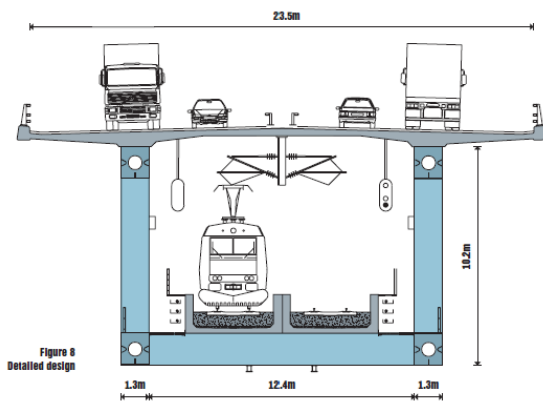


Figure 18. The Øresund Bridge, Denmark – Sweden [3]

Initial cost estimates during the bidding phase indicated that the two-level option was approximately 10% cheaper than the one-level alternatives (Figure 19).

	100m span	120m span	140m span
One-level, concrete	1.10	1.13	(1.18)
One-level, composite	1.13	1.16	(1.21)
One-level, steel	1.18	1.18	(1.20)
Two-level, composite	1.01	1.00*	1.01
Two-level, steel	1.09	1.06	1.05

\*1994 estimate: 1.00 = 3800 MDKK. ( ): Outside conditions

Figure 19. Bid estimates of various concepts

This difference is logical. In the two-level solution, both decks are utilized, whereas in the one-level alternatives, the lower deck remains unused for traffic. The upper concrete deck and the upper steel chords both contribute to the bridge's longitudinal load-carrying capacity. Since the concrete slab was cast while the truss was multi-supported, the upper steel chords could retain

minimal cross sections, allowing full utilization of the steel's capacity.

Although there were many geometrical constraints, optimization was still possible within these boundaries. Quantities were largely dependent on plate thicknesses and reinforcement requirements. The steel plate thicknesses varied individually by 1 mm, and reinforcement was curtailed according to load variations. The piers and foundations were designed with minimum concrete dimensions and filled with sand to ensure adequate robustness against ship collisions.

When the tender design's lower steel box flange was later replaced by concrete troughs, the optimal span length (based on material quantities) was reduced from 140 m to 120 m. However, as the difference in material quantity was only about 1%, the contractor preferred the fewer, larger elements, as this approach was more robust in relation to the construction schedule.

The Owner also preferred replacing the lower steel deck with a concrete girder, since a concrete solution is less susceptible to noise from passing trains compared to an all-steel deck. Initially, a solution was developed where simply supported concrete troughs spanned the steel crossbeams every 20 m without interaction with the main steel structure. However, to improve the lower deck's robustness against ship deckhouse collisions, it was decided to integrate the concrete with the steel structure, providing redundant load paths and failure mechanisms.

The viaduct spans of the Øresund Bridge can thus be characterized as a highly integrated (monolithic) design. All elements participate in complex load transfers, but the advantage of this approach is that material quantities are minimised as much as possible.

## 8 The Sutong Bridge

The Sutong Bridge pylons rise 305 m above the Yangtze River, with piles extending a further 110 m into the soft soil. In total, the structure reaches a height of approximately 415 m (see Figure 20).

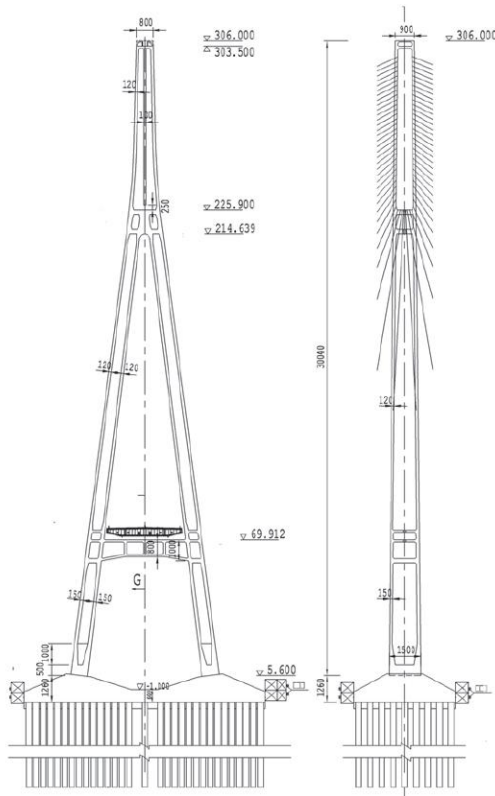


Figure 20. Sutong Bridge [9] pylons

A review of the existing design, which includes an exceptionally large number of long, large-diameter piles (lengths truncated in the illustration figure 21), indicates that at the top of the pile cap, the combined weight of the piles and the pile cap itself already utilizes about 50% of the total geotechnical bearing capacity.

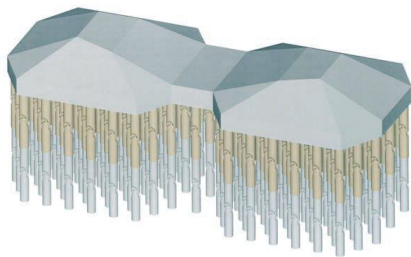


Figure 21. Sutong Bridge, pylon foundations

The pylon wall thickness is 1.2 m at the top, increasing to 1.5 m at the base. The pylon’s self-weight accounts for more than 80% of the total vertical load acting on the pile cap. While the pile length is determined by geotechnical requirements, the number of piles (N) and their diameter (D) are design variables. A large diameter is necessary to prevent buckling, and together with

minimum spacing requirements, this causes the pile cap footprint to scale proportionally with both N and D. As the pile cap thickness is proportional to its width, its volume—and therefore its weight—becomes roughly proportional to  $N^2$ .

For illustration, assume that 0.6 m thick pylon walls at the top would be sufficient to carry the relatively modest loads from the few upper stays, and that 1.0 m thick walls would suffice at the base. Reducing the wall thickness in this way would lower the pylon weight to approximately 60% of the original design value. Since the pylon’s self-weight represents about 80% of the total load on the pile cap, this corresponds to a reduction of roughly one-third in total load. This one-third reduction in vertical load above the pile cap would, due to the  $N^2$  relationship, result in approximately a 50% reduction in the required pile and pile cap structure below.

## 9 Large bridges and CO<sub>2</sub>

It is evident that the CO<sub>2</sub> footprint of large bridges can be significantly reduced. In this article, the focus has been on quantities, and it should be understood that any reduction in material quantities leads to a proportional reduction in CO<sub>2</sub> emissions, even if this relationship has not been explicitly stated.

Bridge engineers and architects must recognise that long spans and large cantilevers may, in fact, conflict with the objective of minimizing CO<sub>2</sub> emissions. The author expects that, in response to the growing importance of CO<sub>2</sub> reduction, future bridge designs will tend toward shorter spans, a greater number of columns, and a revival of more traditional design details.

In addition to quantity reduction, material selection also plays a crucial role. For instance, a glass fibre panel can support pedestrian loads similar to a steel deck but at only a fraction of the associated CO<sub>2</sub> emissions. Therefore, it is important that the selection of structural materials for large bridges — including cable steel, steel grades, and concrete grades — is carefully reconsidered.



For large bridges, where the material quantities themselves often contribute significantly to the overall load, reducing quantities is likely to be more effective than merely optimizing material grades. For example, lowering the concrete grade from C50 to C35 may be technically acceptable, but the potential CO<sub>2</sub> savings from the lower-grade material could quickly be offset by the increased quantities and resulting additional loads.

## 10 Conclusions

Large bridges are characterized by their own weight, which contributes significantly to the total loads of the structure. The part of the bridge farthest from its ground supports is particularly important: if the design assumes an overly robust element in this area, the resulting increase in material quantities throughout the entire bridge is inevitable. The example calculation for the very tall Sutong Bridge pylon is only indicative, but it illustrates that the primary design decisions made at the “beginning” of the load path are crucial.

The same principle applies to the span: a 1-tonne concrete block may cost around €250, but if it is placed in the middle of a 3,000-metre suspended span, the indirect cost could reach €30,000 — 120 times its apparent price.

For continuous beam structures, it is worth revisiting the traditional concept of varying structural depth (haunched girders), as this can significantly reduce material quantities. If a constant-depth continuous beam design is used, ensure that both the mid-span and support sections are fully utilized — no structural capacity should remain unused.

The author generally recommends beginning the design process with relatively optimistic initial assumptions for structural dimensions, so as not to overlook the potential for overall optimization.

Although this article discusses quantities, CO<sub>2</sub> emissions are largely proportional to these. Therefore, the conclusions apply equally to material quantities, construction cost, and — importantly — the potential for CO<sub>2</sub> savings.

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