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Effects of Thermal and Railway Actions on Long-Span Road-Rail Suspension Bridges: The Case of Tsing Ma Bridge in Hong-Kong

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Abstract

Long-span bridges that carry both road and rail traffic on a steel truss deck must resist several service actions; this work compares two of them, the railway traffic and the temperature, which are of very different character. The railway loads are transferred locally through a deep cross-section, while temperature acts on the whole structure through atmospheric exposure and solar radiation. Because the chords of a truss deck are far apart, both in depth and across the width, even small temperature differences between them cause differential elongations. Restrained by the suspension system and by the supports, these elongations produce axial forces in the chords and bending of the deck. Whether the resulting effects are comparable to those of the railway traffic is the question this thesis addresses, taking the Tsing Ma Bridge, a suspension bridge with a 1377 m main span that carries a dual three-lane highway and a twin-track railway on a steel truss deck, as a case study. A three-dimensional finite element model of the bridge was built in ANSYS Mechanical APDL. A geometrically non-linear static analysis under self-weight brings the model to its tensioned dead-load configuration, which was checked against the design dead load, the main-cable tension, and the natural frequencies and mode shapes measured on the bridge. Each action (temperature and railway action) is then applied as an increment on this configuration. Three temperature components are considered, a uniform effective temperature and a vertical and a transverse gradient. The uniform and the vertical components are taken from the long-term monitoring of the Tsing Ma Bridge, while the transverse gradient, for which no field record exists, is adopted from a comparable road-rail truss bridge as a representative loading scenario. They are applied together with the reference values of EN 1991-1-5. The railway action is introduced through two load models: the HSLM-A2 high-speed load model of EN 1991-2, used as a conservative envelope, and the actual service train of the line, reconstructed from available literature. The analyses indicate that the thermal effects represent a large part of those produced by the railway traffic: each temperature component reaches a substantial fraction of the railway effect at the deck chords. The comparison with EN 1991-1-5 further shows that the transverse component, which the code addresses only through a single low recommended value, is under-represented relative to the effect obtained here. The thermal actions therefore appear to deserve more attention in the design and assessment of long-span road-rail truss decks than current practice tends to assume, the transverse gradient being the component

least well represented by the current code provisions.

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Chapter 1

Introduction

1.1 Context

Long-span cable-supported bridges increasingly carry road and rail traffic on a single structure. A deck of this kind is subject to two service actions of very different character. The railway loads are transferred locally, entering the deck at the track and reaching the suspension system through a deep cross-section. The temperature, by contrast, acts on the whole structure through atmospheric exposure and direct solar radiation, and its effect is distributed rather than local. The two actions therefore load the deck in distinct ways, and their relative weight is not self-evident.

The geometry of a steel truss deck makes it strongly exposed to the thermal action. The upper and the lower chords are separated by a large structural depth, and the section is wide, so that even a small temperature difference between regions of the cross-section produces appreciable differential elongations. Restrained by the suspension system and by the supports, these elongations induce axial forces in the chords and deformation of the deck. The same depth and width that the deck needs in order to meet the stiffness demands of railway traffic are thus the features that expose it to thermal gradients.

On a deck of this form the relative weight of the thermal and the railway action effects has not been established. The temperature enters mainly through monitoring studies that reconstruct the temperature field of an instrumented deck, without carrying that field through to the action effects it produces, so the thermal effects and the railway effects have not been placed on a common footing. Whether those thermal effects reach the same order as the effects of the railway traffic for which the deck is dimensioned therefore remains an open question. This work takes up that question, with the Tsing Ma Bridge, a suspension bridge of 1377 m main span that carries a dual three-lane highway and a twin-track railway on a two-level steel truss deck, as its case study.

1.2 Objectives

The primary objective of this work is to assess the relative weight of the thermal and the railway action effects on a long-span road-rail truss deck, placing the two on a common footing so that their importance can be judged at the level of the resulting stresses. The railway action is taken as the reference service load against which the structural role of the temperature is weighed. The Tsing Ma Bridge serves as the case study throughout.

Meeting this objective requires a model that reproduces the behaviour of the bridge under load and that can be trusted before any action is applied. A three-dimensional finite element model is therefore built and brought to its tensioned dead-load configuration through a geometrically non-linear static analysis under self-weight, and is validated against the design dead load, the main-cable tension, and the natural frequencies and mode shapes measured on the bridge. Each action is then applied as an increment on this configuration. Three thermal components, a uniform effective temperature and a vertical and a transverse gradient, and two railway load models, a code-based envelope and the measured service train of the line, are introduced as independent load cases, and the action effects are quantified at a mid-span reference cross-section, together with the axial force in the main cable and the vertical displacement and slope of the deck. A secondary objective follows from the same comparison, namely to weigh the thermal provisions of EN 1991-1-5 against the response obtained here.

1.3 Structure of the thesis

The thesis is organised in seven chapters. The present chapter sets the context, states the objectives and describes the organisation of the work. Chapter 2 reviews the state of the art, covering the structural limits of super-long-span cable-supported bridges and the typologies of dual-use road-rail bridges, and identifies the research gap that motivates the study. Chapter 3 introduces the Tsing Ma Bridge, its setting, its design and construction, its general arrangement and the instrumentation from which the validation data originate. Chapter 4 presents the finite element model, the modelling strategy, the geometry, the elements, the material properties, the cross-sections, the connections and boundary conditions, and the analysis procedure, closing with the validation against the dead load and the measured modal properties. Chapter 5 applies the thermal and the railway actions, sets out the load cases and quantifies and compares the resulting action effects, answering the question raised in this introduction. Chapter 6 discusses the sustainability of the work and its ethical implications along the economic, environmental and social dimensions, and relates it to the Sustainable Development Goals. Chapter 7 draws the conclusions, sets out the limitations of the study and gives the recommendations for further work.

Chapter 2

State of the Art

2.1 Evolution and Structural Limits of Super-Long Span Cable-Supported Bridges

2.1.1 Introduction

Long-span cable-supported bridges are among the most demanding achievements of structural engineering. In recent years, bridges with main spans exceeding 2000 m have moved from being regarded as exceptional structures to becoming feasible, owing to progress in ultra-high-strength materials, the refinement of nonlinear calculation methods, the use of efficient aerodynamic cross-sections and advances in construction technology, as reviewed by Huang et al. [1]. Below these lengths, classical structural theories remain valid, although they are pushed to their limits.

Above 2000 m, the mechanisms that govern the behaviour of these structures begin to change. As discussed by Tang [2], feasibility is no longer controlled by the strength of the materials alone; the reduction of stiffness, geometric nonlinearity and global stability become the decisive factors in the design process, while the strength checks, still necessary, cease to be the main constraint. Tang [2] observes that gravity, seismic and vehicle-impact loads do not set the limit on the achievable span; the binding constraints are the effective axial stiffness of the main cables, which is reduced by their own sag, and the aerodynamic stability of the deck.

Extending the main span beyond these limits is therefore not a matter of resizing and scaling existing solutions. The entire system undergoes a transformation with respect to shorter spans. As the span grows, the bridge behaves less like a stiff beam and more like a flexible, dynamically articulated system. Wind and second-order effects are no longer secondary corrections; they become integral components of equilibrium.

Technological improvements, such as high-strength steel wires, enhanced anchorage systems, advanced nonlinear finite element analyses and refined wind-tunnel testing, have

allowed designers to approach and slightly exceed the 2000 m threshold [1]. Beyond this limit the dominant issue becomes system behaviour. The challenge is not only to resist larger forces, but to govern deformation, stability and the interaction with environmental actions.

2.1.2 Structural Scaling Effects and Increasing Role of Dead Load

One of the most immediate consequences of increasing the span is the dominance of dead load. In the conceptual 4000 m proposal of Wang, Lai and Wei [3], the live-to-dead load ratio falls to approximately 0.05, a value comparable to that of a 2500 m suspension bridge. Under such conditions the response of the main suspension cables, the piers and the foundations is governed almost entirely by self-weight.

This change has marked implications: cable forces increase disproportionately, structural efficiency decreases, and geometric nonlinearities grow. The system shifts from a deck-bending-dominated configuration to a tension-stabilised mechanism in which global stability prevails over cross-sectional resistance. The structural logic changes accordingly, and the problem moves from one of bending in the deck to one of stability of the whole bridge.

As the span increases, the global stiffness, defined here as the ratio between an applied load and the resulting displacement, decreases while the total structural mass grows. The natural frequencies follow the proportional relationship

$$f \propto \sqrt{\frac{k}{m}}, \quad (2.1)$$

so that their reduction becomes a key parameter and increases the sensitivity to wind-induced excitation and to modal coupling effects [2]. The dynamic response can no longer be treated as a secondary verification step.

For ultra-long spans, beyond the 2000 m threshold discussed above, stability under wind loading tends to determine the main design decisions. In particular, the ultimate limit state of loss of static equilibrium and of global instability must be addressed with priority, since the high flexibility of the system makes it susceptible to large-scale deformations before material failure occurs. The focus of the design then shifts to the control of stiffness and dynamic performance.

Geometric nonlinear analysis becomes essential as the span increases [2], because linear models systematically underestimate the stiffness softening and the load-redistribution mechanisms, especially when axial and bending effects interact under large displacements.

The interaction between deck and cables emerges as the dominant structural mechanism, since the axial cable stiffness and the geometric configuration increasingly determine the

overall response of the structure. The bending stiffness of the deck remains relevant but takes a secondary part, so that axial and geometric effects become primary while purely flexural interpretations lose relevance.

Decks of ultra-long span therefore cannot be reliably treated through conventional beam analogies, since their behaviour is nonlinear and system-dependent.

A measure often invoked to frame this transition is the slenderness of the deck, expressed through the ratio between its structural depth and the main span, which [4] take as a fundamental parameter for judging the deformability of a super-long-span bridge.

These trends are already observed for spans shorter than 2000 m. Projects such as the Sutong, Edong and Hutong Yangtze River bridges [1] show how modern design moves toward advanced structural solutions in order to raise the global stiffness of the system and to manage complex actions, namely wind-induced loads, traffic loads and their dynamic interaction, as shown in Figure 2.1. The Edong bridge adopts a mixed steel–concrete girder, with steel over the central span and concrete over the lateral spans, to optimise the redistribution of material. The Hutong bridge, a combined road and rail steel-truss cable-stayed structure, and the Sutong bridge rely on large-scale deep foundations dimensioned to limit the deformations and dynamic effects that become critical at these spans. These cases confirm that increasing spans call for new structural strategies, ranging from mixed and composite girders to large-scale foundation systems, to ensure the stability and safety of the works.

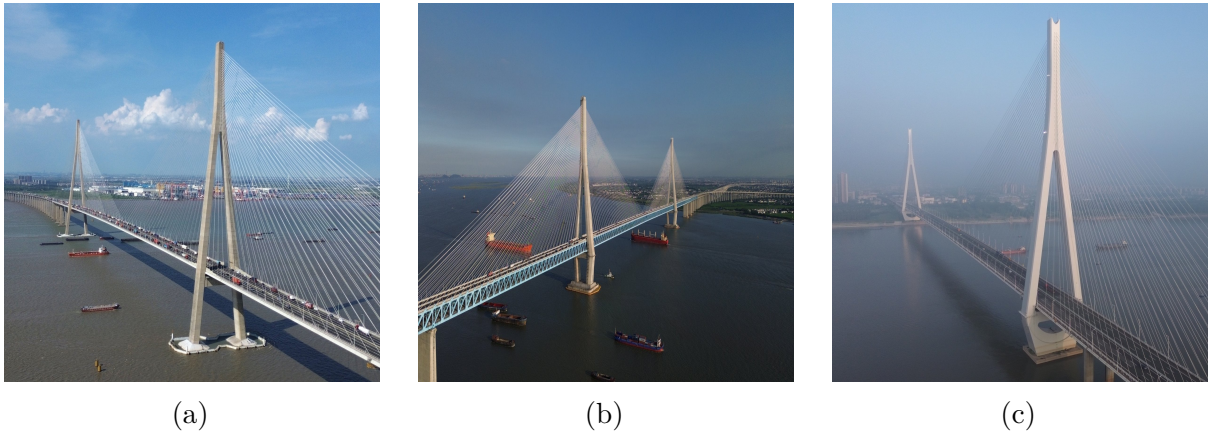


Figure 2.1: (a) Sutong Yangtze River Bridge; (b) Hutong Yangtze River Bridge; (c) Edong Yangtze River Bridge.

2.1.3 Aeroelastic Stability

For spans longer than 2000 m, aeroelastic stability becomes the constraint that determines feasibility, and therefore the way the design is carried out, as historical failures due to flutter make clear. Systems of this kind are highly vulnerable in this respect, so that for spans above 2000 m Ostenfeld and Larsen [5] treat the wind–structure interaction as a fully coupled dynamic problem.

Tests on section models are insufficient to describe the behaviour of bridges with spans in the range between 3000 m and 5000 m. Working on a suspension bridge with a 5000 m main span, arranged as a 2000+5000+2000 m configuration suspended by four main cables, Ge et al. [6] show that a full aeroelastic model is required to capture the modal coupling and the antisymmetric vibration modes, and that the flutter velocity is highly sensitive to the torsional stiffness and to the aerodynamics of the cross-section. For that span the analysis of flutter proves more critical than that of aerostatic divergence.

Most studies within this span range remain conceptual. At present the absence of operational bridges of such dimension leaves a gap between theoretical feasibility and empirical validation.

The deck configuration and the aerodynamic profile are decisive for the aerodynamic performance. Separated or slotted twin-box girders raise the torsional stiffness and increase the critical flutter velocity, although such configurations may also increase the susceptibility to vortex-induced vibrations [6, 7]. Aerodynamic design cannot therefore be reduced to the maximisation of a single parameter; a balance is required between torsional stiffness, mass distribution, cross-section geometry and constructability. In these projects the structural and the aerodynamic design proceed together.

The existing Xihoumen Bridge, a suspension bridge in China with a main span of 1650 m, documented by Huang et al. [1] and shown in Figure 2.2, adopts a slotted double-box stiffening girder to improve flutter resistance, and its satisfactory performance under severe typhoon conditions confirms the relevance of the aerodynamic optimisation of the cross-section.

The conceptual study extending to 5000 m by Ge et al. [6] indicates that multi-box or spaced-out arrangements may further delay aeroelastic instability, with the geometry of the deck strongly affecting the flutter velocity. For spans beyond 3000 m, however, implementation remains largely theoretical. Aeroelastic stability appears to define the outer boundary of structural feasibility, yet full-scale validation is still lacking.



Figure 2.2: Xihoumen Bridge.

2.1.4 Hybrid Structural Systems

To meet the high stiffness required by these works, hybrid structural concepts have been proposed. A hybrid cable system combines the suspension action over the central span with cable stays near the pylons; a mixed or composite girder combines different deck materials, typically steel and concrete, along the span. The hybrid cable system, often associated with prestressed concrete in the regions subjected to significant axial forces, establishes a cooperation between these elements that improves the distribution of the internal forces and the overall stiffness [3].

Although the hybrid solution improves the structural performance, it complicates the internal force paths, which makes advanced nonlinear analyses and extensive construction control indispensable. The solution therefore involves a trade-off between improved performance and increased computational and constructional effort.

The Third Bosphorus Bridge (Yavuz Sultan Selim Bridge) is a notable example of this approach, discussed by Casciati and Faravelli [8]. By combining suspension hangers and stay cables, as shown in Figure 2.3, the structure reaches a higher stiffness than a conventional suspension system, and the integration of road and railway traffic illustrates the role of the hybrid cable system in controlling deformation within demanding serviceability limits.



Figure 2.3: Third Bosphorus Bridge (Yavuz Sultan Selim Bridge).

A further example is the Tsugaru Strait Bridge proposed by Wang, Lai and Wei [3], which would cross the strait between the islands of Honshu and Hokkaido in northern Japan (Figure 2.4), with two central spans of 4000 m under demanding environmental conditions in which purely cable-stayed or suspension systems would be inadequate. The proposal adopts a hybrid solution: cable-stayed sections with prestressed concrete decks near the main towers withstand the high axial compression, while a suspended system with

truss-stiffened steel box decks over the central span reduces the self-weight and improves the aeroelastic stability. The bridge remains a proposal.

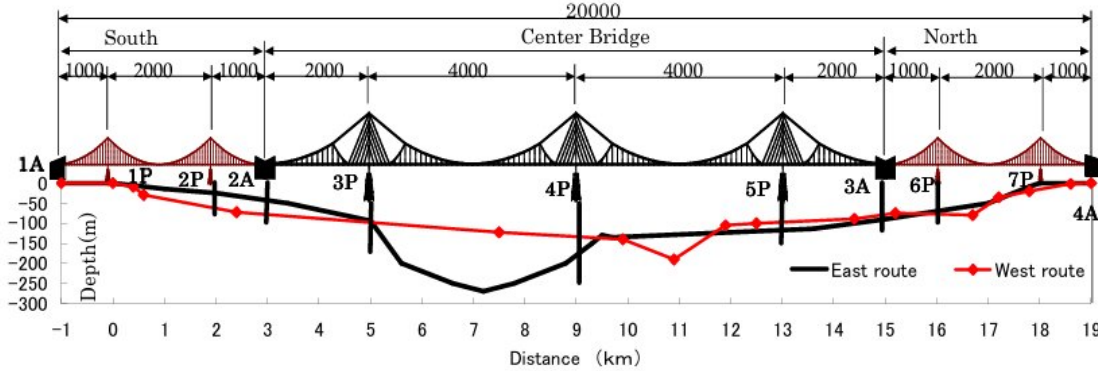


Figure 2.4: Longitudinal profile of the proposed Tsugaru Strait Bridge. From [3].

A different form of hybridisation combines a bridge and a tunnel, as in the proposed crossing of the Strait of Gibraltar between Europe and Africa. This project addresses critical infrastructure needs but faces considerable challenges, owing to the seismic activity of the region, the strong tidal currents and the environmental concerns. No bridge or tunnel has been built so far, and several proposals have been examined to establish feasibility at this location [9].

2.1.5 Current Status of Bridges Exceeding 2000 m

Although conceptual studies explore spans up to 4000–5000 m, operational bridges exceeding 2000 m remain few. Recent projects such as Zhangjinggao and Shiziyang [10, 11] show that the 2 km barrier is being actively challenged. These developments reveal the growing scale of the structural components and of the displacement-accommodation systems required to manage the global stiffness and to ensure constructability. The planned Strait of Messina Bridge, whose definitive design was approved in 2025, is intended to carry six road lanes and a double-track railway across a single suspended span of 3300 m, which would extend this frontier well beyond any existing structure [12].

The field remains little explored but is expected to expand markedly in the coming years, with projects under construction and a growing number of studies for future works.

Although suspension bridges have now exceeded the 2000 m threshold, their functional typology remains predominantly highway-oriented. Table 2.1 summarises representative cases, while Figure 2.5 shows the bridges among those listed that are already in service.

Bridge	Country	Span [m]	Year	Type
1915 Çanakkale Bridge	Turkey	2023	2022	Suspension
Akashi Kaikyō Bridge	Japan	1991	1998	Suspension
Nanjing Xianxin Yangtze River Bridge	China	1760	2025	Suspension
Yangsigang Yangtze River Bridge	China	1700	2019	Suspension
Strait of Messina Bridge	Italy	3300	2032 (Planned)	Suspension
Zhangjinggao Yangtze River Bridge	China	2300	2028 (u.c.)	Suspension
Second Sutong Yangtze River Bridge	China	2300	2031 (u.c.)	Hybrid
Shiziyang Bridge	China	2180	2028 (u.c.)	Suspension

Table 2.1: Super-long span bridges.

(u.c.) = under construction.



(a) 1915 Çanakkale Bridge



(b) Akashi Kaikyō Bridge



(c) Nanjing Xianxin Yangtze River Bridge



(d) Yangsigang Yangtze River Bridge

Figure 2.5: Existing bridges with a main span of at least 1700 m.

None of the currently operational bridges exceeding 1700 m carries high-speed railway traffic. This absence is not incidental; it reflects the structural incompatibility between ultra-long spans and the stiffness required by railway serviceability.

2.2 Dual-Use Bridges

2.2.1 Introduction and Structural Complexity

Bridges designed to carry both highway and railway traffic represent one of the most demanding structural typologies, since road and rail impose fundamentally different

serviceability requirements. High-speed rail (HSR), in particular, requires strict control of deflection, acceleration and vibration, whereas road traffic allows comparatively greater flexibility. On high-speed lines, bridges account for a very large share of the route, up to 91% on some Chinese corridors [13], which makes the behaviour of these structures decisive for the operation of the railway.

Integrating both systems within a single structural framework requires careful management of deformation, dynamic behaviour, wind effects and track–bridge interaction. A coherently designed track–bridge system is essential to ensure serviceability and long-term performance.

2.2.2 Track–Bridge Interaction

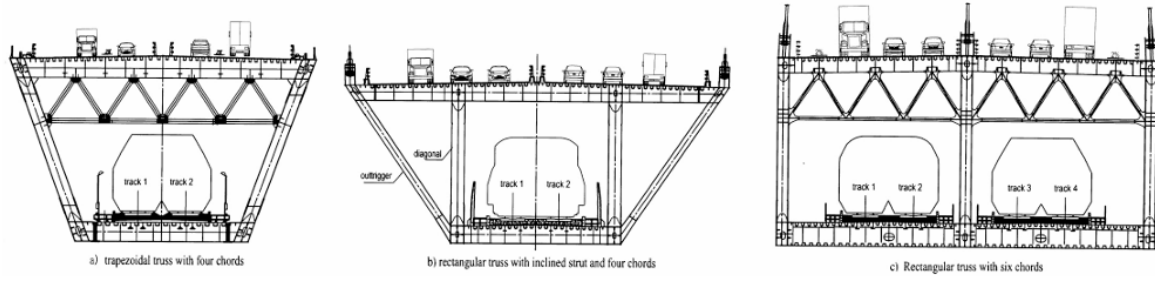
A critical aspect of dual-use bridges is the track–bridge interaction, since the deflection of the bridge directly affects the geometry of the track, with the consequent risk of alignment problems, increased maintenance and a reduction in passenger comfort. Su et al. [13] note that ballastless track, which now predominates on high-speed lines, is markedly more sensitive than ballasted track to the deformations imposed by temperature and by the bridge, so that the track and the structure have to be treated as an integral system during the design. To limit the relative vertical displacements between the tracks, Casciati and Faravelli [8] propose placing a stiff longitudinal continuous girder beneath each track, a principle that addresses the serviceability of the railway directly.

2.2.3 One-Level and Two-Level Arrangements

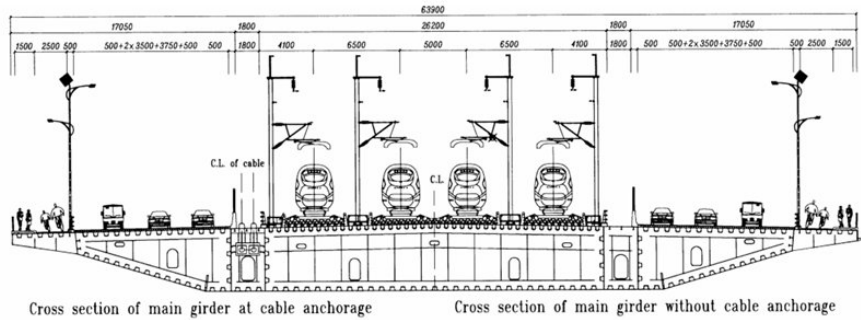
Dual-use bridges were historically realised in two-level configurations with deep truss girders. Early examples showed that railway serviceability governs the structural depth and the stiffness requirements. With the development of wide steel box girders and multi-cable systems [14], one-level arrangements have become feasible for medium and long spans.

Two-level configurations separate rail and road traffic vertically, which typically increases the structural depth and the substructure demands. Truss girders or box–truss systems are often adopted to achieve the required stiffness (Figure 2.6), at the cost of a higher material consumption [7].

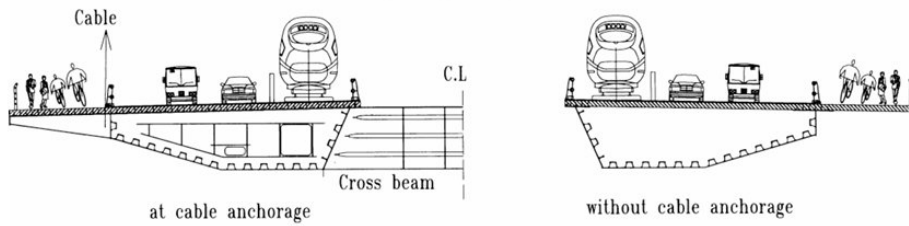
One-level configurations place both systems on a common horizontal plane, usually within a wide steel box girder. This reduces the structural height but introduces challenges related to deck width, fabrication complexity and aerodynamic performance. For spans exceeding 1000 m, box girders are often preferred for their dynamic efficiency [7].



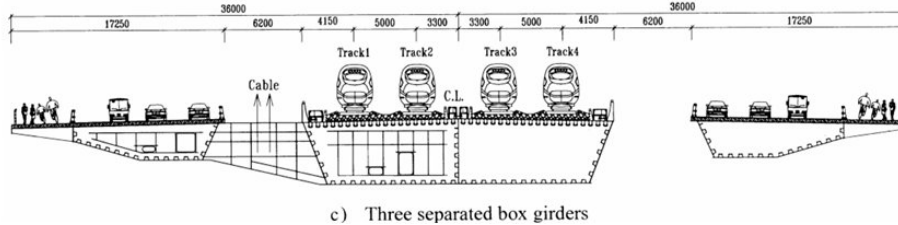
(a) Truss girders.



a) Single box girder



b) Two separated box girders



c) Three separated box girders

(b) Box girders: single, two separated and three separated.

Figure 2.6: Typical girder cross sections for cable-supported rail-cum-road bridges: (a) truss girders; (b) box girders. From [14].

The Lingang Yangtze River Bridge, described by Wang et al. [15], is a cable-stayed bridge with a main span of 522 m located to the east of Yibin, in the Sichuan Province of the People's Republic of China. It is the first bridge in the world to carry high-speed rail and a highway on the same level, accommodating four high-speed railway tracks, with a design speed of 300 km/h, and six road lanes, with a design speed of 80 km/h, within a single five-cell steel box girder, as shown in Figure 2.7. The design integrates multiple cable systems and auxiliary piers to coordinate stiffness and deformation control, while a multifunctional protective barrier addresses the problems of glare, crosswind and noise

between trains and cars.



Figure 2.7: Lingang Yangtze River Bridge.

2.2.4 Girder Typologies

Truss girders provide high axial and bending resistance and suit heavy two-level configurations, at the expense of increased maintenance. Single box girders offer better inspectability and aerodynamic performance, particularly in single-level arrangements.

For very wide decks, separated box girder systems can raise the torsional stiffness and the flutter resistance [7]. Three-box arrangements can improve the dynamic stability and reduce the transmission of vibration between corridors [7, 15]. As the span and the deck width increase, torsional behaviour becomes a governing design concern.

2.2.5 Wind and Aeroelastic Considerations

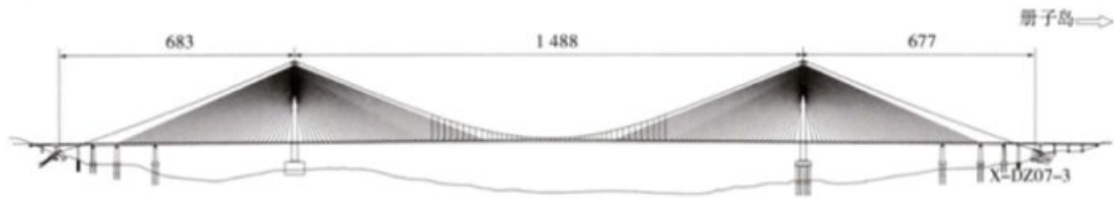
Dual-use bridges carrying HSR traffic are especially sensitive to wind-induced effects, so that comprehensive aeroelastic investigations, both experimental and numerical, are essential [6]. While separated box configurations improve the torsional stiffness, aerodynamic refinement remains necessary to avoid undesirable dynamic amplification.

The Xihoumen Rail-Road Bridge and the Jingzhou Libu Bridge [8, 16] show that long-span design can be reconciled with the strict serviceability requirements of railway traffic through very stiff deck structures and well coordinated support systems that control deformations and vibrations. Both works are currently under construction. The Jingzhou Libu Bridge, on the Yangtze River, is a railway-highway crossing with a main span of 1120 m, shown in Figure 2.8 [8]. The Xihoumen Rail-Road Bridge, on the Ningbo–Zhoushan railway and distinct from the existing 1650 m suspension Xihoumen Bridge described in subsection 2.1.3, adopts a hybrid cable system with a main span of 1488 m and a 68 m

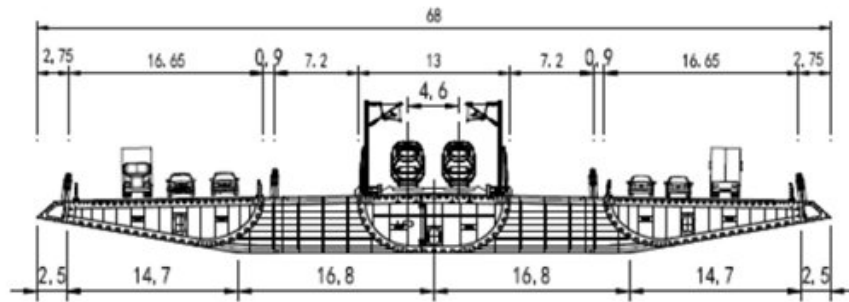
wide deck formed by three separated steel box girders, as shown in Figure 2.9; it carries a double-track railway at the centre and a six-lane expressway on the two sides on a single level, and the completion of its main span is expected toward 2028 [16].



Figure 2.8: Rendering of the Jingzhou Libu Yangtze River Railway Bridge. From [8].



(a) Longitudinal elevation.



(b) Deck cross-section (dimensions in m).

Figure 2.9: Xihoumen Rail-Road Bridge: (a) longitudinal elevation; (b) deck cross-section. From [8].

2.2.6 Dual-Use Bridge Examples

The Tsing Ma Bridge in Hong Kong is a suspension bridge with a main span of 1377 m that carries both road and rail traffic, as shown in Figure 2.11. Its deck is arranged on two levels, with the carriageway on the upper level and the railway, together with sheltered lanes, on the lower level [17]. The bridge is examined in detail in the following chapters, where it serves as the reference structure for the finite element model.

The 25 de Abril Bridge in Lisbon, Portugal, is a dual-use structure that carries both road and rail traffic, as shown in Figure 2.10. It was originally designed for highway

traffic only and was subsequently retrofitted to accommodate a railway, which required structural modifications to address the increased loads and the more stringent serviceability requirements. With its suspension design, it manages the dynamic loads from the two types of traffic while maintaining structural stability, and its two-level arrangement separates road and rail, which improves the traffic flow and reduces the maintenance demand. It remains one of the most iconic bridges in Europe and an early example of integrated transport infrastructure [18].



Figure 2.10: 25 de Abril Bridge.

Representative examples of dual-use bridges are summarised in Table 2.2, while the existing ones are shown in Figure 2.11.

Bridge	Country	Span [m]	Year	Type
Lingang Yangtze River Bridge	China	522	2023	Cable-stayed
Yavuz Sultan Selim Bridge	Turkey	1408	2016	Hybrid
Tsing Ma Bridge	Hong Kong	1377	1997	Suspension
Minami Bisan-Seto Bridge	Japan	1100	1988	Suspension
Hutong Yangtze River Bridge	China	1092	2020	Cable-stayed
Wufengshan Yangtze River Bridge	China	1092	2020	Suspension
Strait of Messina Bridge	Italy	3300	2032 (Planned)	Suspension
Xihoumen Rail-Road Bridge	China	1488	2028 (u.c.)	Hybrid

Table 2.2: Dual-use bridges.

(u.c.) = under construction.



(a) Lingang Yangtze River Bridge



(b) Yavuz Sultan Selim Bridge



(c) Tsing Ma Bridge



(d) Minami Bisan-Seto Bridge



(e) Hutong Yangtze River Bridge



(f) Wufengshan Yangtze River Bridge

Figure 2.11: Existing dual-use bridges.

2.2.7 Synthesis and Research Gap

The works reviewed in this chapter converge on a deck that must satisfy demanding stiffness and serviceability requirements while carrying road and rail traffic together. For two-level arrangements, where road and rail are stacked and the structural depth required favours a deep girder, the steel truss remains the established form [14]; its design is governed by stiffness, dynamic behaviour, wind action and the track-bridge interaction set up by the traffic [13, 8].

Across these actions temperature appears only at the margin. The reviewed work treats it as a source of deck deformation to which ballastless track is sensitive [13], not as a structural action that loads the deck itself, although the geometry of a truss section makes it strongly exposed to thermal gradients. Where temperature is examined as such, it enters through monitoring studies that reconstruct the temperature field of an instrumented deck;

Zhang et al. [19] obtain the uniform, vertical and transverse components of a road-rail steel truss cable-stayed bridge, without carrying that field through to the action effects it produces. Whether those effects, on a deck of this form, reach the same order as the effects of the railway traffic for which the deck is designed therefore remains open.

The present work takes up this question, placing the thermal action effects and the railway action effects on a long-span road-rail truss deck on a common footing, so that their relative weight can be judged at the level of the resulting stresses. A secondary outcome follows from the same comparison, since the thermal provisions of EN 1991-1-5 [20] prescribe a large vertical difference but reduce the transverse difference to a single low recommended value, with no dependence on the depth or the width of the section: the measured field and the code values can then be weighed against each other through the action effects they produce, the transverse component being the principal concern there.

Chapter 3

The Tsing Ma Bridge

The Tsing Ma Bridge (Figure 3.1) is the principal structure of the Lantau Link, the combined road and rail connection that joins the urban areas of Hong Kong to Lantau Island and, beyond it, to the international airport at Chek Lap Kok [21]. It is adopted as the case study of this work because it combines the features the analysis requires. The bridge belongs to the class of long-span, cable-supported structures carrying both road and rail traffic on a two-level deck. Its design is documented in the public literature, and finite element models of it have been published, so the model developed in chapter 4 can be built on a firm basis and compared with established results. The structure is permanently instrumented, and ambient vibration surveys have provided measured modal properties against which a numerical model can be validated. Its two-level steel truss deck, with the carriageway above and the railway below, offers a realistic setting for the thermal and railway actions treated in chapter 5.

This chapter sets the bridge in its regional transport context, recalls the circumstances of its design and construction, and describes its general arrangement, before summarising the instrumentation from which the validation data originate.



Figure 3.1: Tsing Ma Bridge.

3.1 Setting and significance

Air and marine traffic through Hong Kong grew quickly during the closing decades of the twentieth century, and the territory sited its new airport and container port on Lantau Island [22]. The new facilities could be reached from the established centres on Hong Kong Island and Kowloon only through new highway and railway links, of which the Lantau Link is the key section. Within the crossing the Tsing Ma Bridge is the principal structure, carrying a dual three-lane highway on the upper level of the deck and two railway tracks together with two carriageways on the lower level [21]. The same airport infrastructure programme produced two further cable-supported bridges on the route, the Kap Shui Mun Bridge (770 m long, completed in 1997) and the Ting Kau Bridge (1177 m, completed in 1998), with the later Stonecutters Bridge (1596 m, completed in 2009) added to the wider network [22].

When it was completed, the Tsing Ma Bridge was the longest suspension bridge in the world to carry both road and rail traffic [23, 24]. Its dual function shapes how the structure is operated in severe weather. Once the hourly mean wind speed recorded on site exceeds 165 km/h, both decks are closed to road vehicles, and the railway on the sheltered lower level becomes the only land connection between Lantau, with its airport, and the rest of Hong Kong [22]. The bridge and the highway, bridge and tunnel connections around it are administered together as the Tsing Ma Control Area [25].

3.2 Design and construction

The bridge was designed by the consulting engineers Mott MacDonald, with A. S. Beard as project director [23, 22]. An initial scheme had been prepared as early as 1982, carrying a two-lane dual carriageway and a light railway, and the bridge was redesigned from 1989 to take the three-lane dual highway and the heavier airport railway [26]. The loading rules followed the Hong Kong Civil Engineering Manual, itself based on BS 5400, with highway, railway and wind criteria developed specifically for the exceptional span [23].

The construction contract, worth about HK\$7.2 billion, began in May 1992, and the bridge opened to traffic in May 1997 after sixty months of work [27]. The works were carried out by the Anglo Japanese Construction Joint Venture, a grouping of Costain, Mitsui and Trafalgar House [26].

The two 206 m concrete towers were raised by slipforming, and the cable saddles, each weighing 500 tonnes, were lifted onto the tower tops by strand jacks [27] (Figure 3.2). The two main cables were formed by aerial spinning, with about 27 000 tonnes of galvanised steel wire spun and adjusted over nine months [27] (Figure 3.3). The deck steelwork was fabricated abroad, mainly in Britain, Dubai and Japan, assembled into some fifty sections of roughly 36 m and about 1 000 tonnes each at a fabrication yard near Dongguan on the Pearl River, then barged to the site and erected over eight months [27] (Figure 3.4). The Lantau Link was inaugurated on 27 April 1997 and opened to traffic on 22 May of the same year [26].

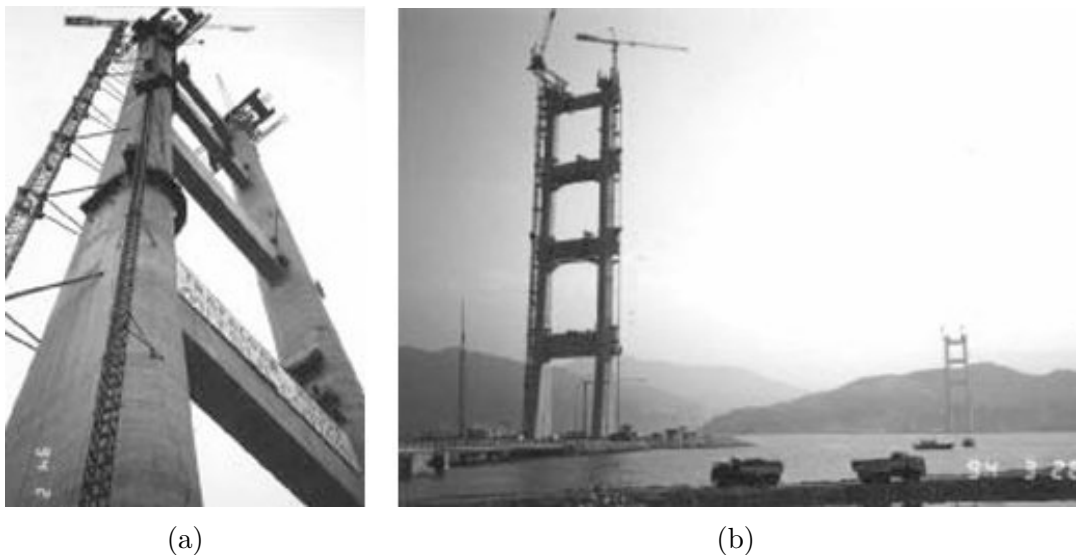
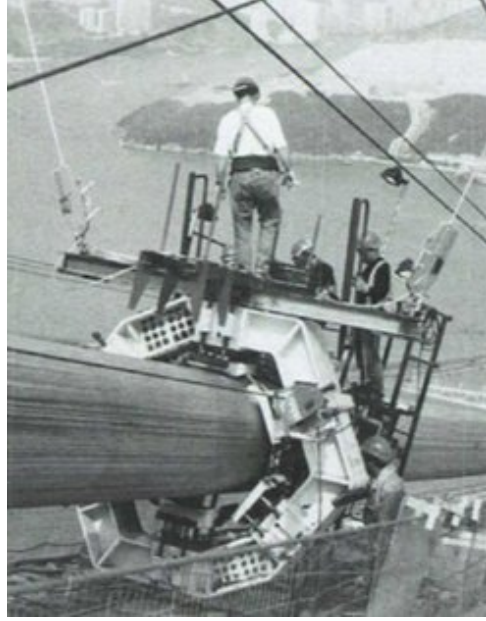


Figure 3.2: Construction of the bridge towers: (a) a tower leg during erection; (b) a tower nearing completion seen from the channel. From Xu and Xia [22].



(a)

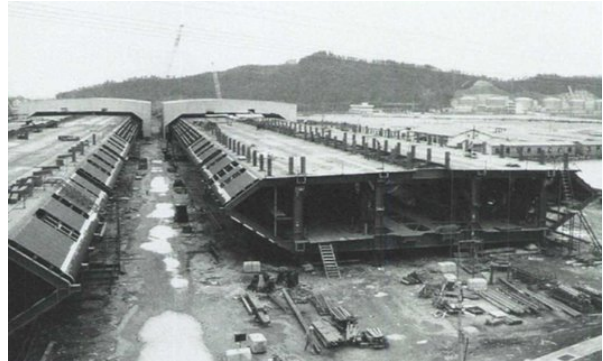


(b)

Figure 3.3: Construction of the main cables: (a) aerial spinning of a main cable; (b) work on a main cable during construction. (a) from Xu and Xia [22]; (b) from Beard [23].



(a)



(b)



(c)



(d)

Figure 3.4: Construction of the deck: (a) a fabricated deck section; (b) deck sections at the assembly yard; (c) a deck section barged to the site; (d) an erected deck section. (a), (c) and (d) from Xu and Xia [22]; (b) from Beard [23].

3.3 General arrangement

The Tsing Ma Bridge is a suspension bridge with a single main span of 1377 m, carried between the Ma Wan tower to the west and the Tsing Yi tower to the east; the overall length is about 2132 m [28, 22] (Figure 3.5). The two reinforced concrete towers stand 206 m high to the level of the cable saddles, and the two main cables run 36 m apart [22]. The deck is a two-level steel truss structure, formed by transverse Vierendeel cross-frames carried on longitudinal trusses and closed by stiffened steel plates, that carries the dual three-lane highway on its upper level and two railway tracks, with two sheltered carriageways, on its lower level, the railway running at the lower-chord level [21, 17, 22](Figure 3.6). The separation between the upper and the lower chords, together with the width of the section, is the feature that exposes the deck to thermal gradients; the composition of the truss is described in detail in chapter 4.

The suspended deck spans the main opening and the Ma Wan side span, whereas the Tsing Yi side span is supported on three piers rather than by the cables. This arrangement, together with the unequal side spans and the two piers on the Ma Wan approach, makes the structure asymmetric about midspan [21].

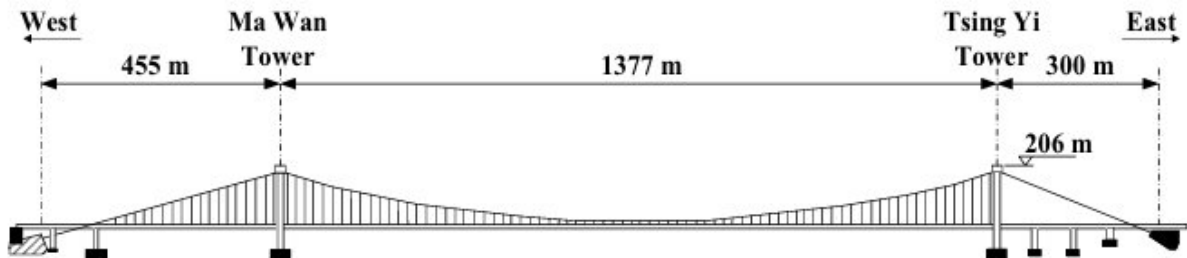


Figure 3.5: Longitudinal elevation of the Tsing Ma Bridge. From Lu et al. [29].

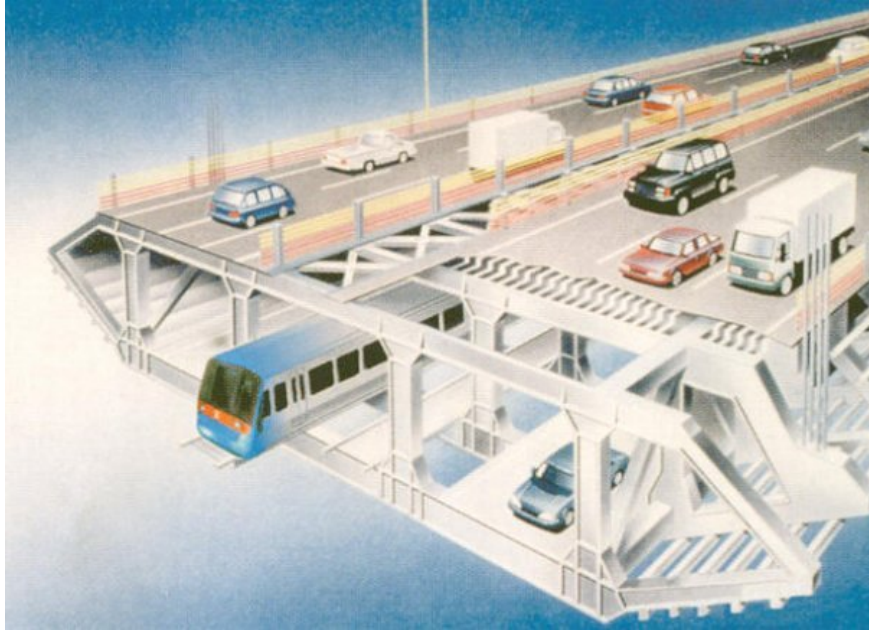


Figure 3.6: View of the two-level deck.

3.4 Instrumentation and modal measurements

The bridge is permanently instrumented with the Wind and Structural Health Monitoring System (WASHMS), installed and operated by the Highways Department of the Hong Kong Special Administrative Region, which also monitors the neighbouring Kap Shui Mun and Ting Kau bridges [22, 25]. Apart from this permanent system, ambient vibration measurements were taken on the completed structure during the construction period, first by the Hong Kong Polytechnic University after the deck welding had been finished and then by Tsinghua University after the surface paving [24]. These campaigns produced the measured natural frequencies and mode shapes used to validate the finite element model in chapter 4; the values adopted there derive from the vibration study of Xu, Ko and Zhang [28].

Chapter 4

Finite Element Model of the Tsing Ma Bridge

4.1 Modelling strategy

The assessment of the behaviour of a long-span suspension bridge such as the Tsing Ma Bridge requires a finite element model that reproduces the geometry of the towers, the deck, the suspension system, the piers and the anchorages, and equally the way these subsystems are articulated, so that it returns a reliable description of their interaction under load. The geometry, which the design drawings already provide, does not by itself determine the global response: this depends as much on the connections between the subsystems and on the stiffness introduced by the cable tension as on the shape of the structure.

4.1.1 Modelling approaches in the literature

Three distinct approaches to the modelling of the Tsing Ma deck are documented in the literature, each characterised by a different level of spatial resolution and intended for a different class of analysis.

The first approach, introduced by Xu, Ko and Zhang [28], represents the suspended deck as a single equivalent beam, commonly referred to as a spine beam. The sectional properties of this beam, comprising the cross-sectional area, the principal moments of inertia, the torsional constant and the shear areas in the two principal directions, are derived from a detailed three-dimensional model of the typical 18 m deck segment, by relating the resultant forces to imposed unit deformations of the segment. The equivalent beam, placed at the shear centre of the cross-section, is connected to the suspenders through rigid transverse arms, so that the suspended portion of the deck is reduced to a one-dimensional member while the first global modes remain available for comparison with measurements. The spine model is computationally economical and remains a standard

reference for checks on global equilibrium and on the lowest natural frequencies.

The second approach, developed by Duan et al. [17], is a three-dimensional model of the entire bridge, comprising about half a million elements, organised into five deck modules: the main span, the deck module at the Ma Wan tower, the Ma Wan approach span, the deck module at the Tsing Yi tower and the Tsing Yi approach span. In each module the orthotropic plates are modelled with **QUAD4** shell elements and the plate troughs with **BAR2** beam elements; the longitudinal trusses, the cross-frames, the sway bracings and the cross bracings are modelled explicitly with beam elements; the railway track system is reproduced separately by means of track plates, rail waybeams and tee diaphragms. The resolution achieved supports component-level analyses, including the evaluation of local stresses, fatigue assessment and damage propagation, at the cost of a large computational effort and of a level of geometric detail that exceeds the information available in the public design drawings.

The third approach, presented systematically by Xu and Xia [22], combines the two preceding methods into a hybrid procedure, in which the detailed sectional model is used to determine the equivalent sectional properties of the deck and the spine model provides the basis for global analyses and for the subsequent model updating.

4.1.2 Approach adopted in the present work

The model developed in this work follows the detailed three-dimensional topology of the Tsing Ma deck proposed by Duan et al. [17], rather than the equivalent spine idealisation of Xu, Ko and Zhang [28]. The detailed topology is adopted so that the stiffness and the mass of the deck arise from an explicit assembly of its members and plates and the structure stays identifiable at the level of its individual families; the spine model condenses the suspended deck into a single beam, which is economical for the lowest global frequencies but does not retain this component identity. The modular organisation of [17] is kept, together with the distinction between the principal structural families of the deck: the outer and inner longitudinal trusses, the Vierendeel cross-frames, the sway bracings at the main cross-frames, the four systems of horizontal cross bracings, and the orthotropic deck plates. These families are the ones labelled on the prototype deck segment reproduced in Figure 4.7.

The resolution is coarser than in [17], through two simplifications introduced in the present work. The steel members whose true cross-sections are box, I and T shapes are each replaced by an equivalent solid rectangular beam section, since the dimensions of the real shapes are not given in the published literature on the bridge (section 4.5). The railway track system of the central lower deck, reproduced in [17] by separate track plates, rail waybeams and tee diaphragms, is smeared into a single equivalent orthotropic plate of calibrated thickness (subsection 4.7.6).

A distinction is kept throughout the chapter between quantities of three kinds. The overall geometry, the member layout and the support scheme are taken directly from the literature on the bridge [28, 17, 22]. The material densities, the elastic modulus and the cable and suspender areas are physical or design values, also drawn from that literature. The equivalent rectangular sections and the smeared orthotropic plate are the modelling choices specific to this work; their dimensions are not obtained from a sectional analysis, which the published data would not support, but are fixed by reference to two measurable quantities, the dead load carried by the deck and the natural frequencies recorded in the field. Each such choice is stated where the corresponding component is described, and the dead-load equilibrium, the cable forces and the frequencies that result are reported and compared with the reference values in section 4.10.

The model is implemented in ANSYS Mechanical APDL, whereas the reference model of [17] was built in MSC/NASTRAN. The change of solver leaves the topological correspondence between the two models unchanged, but it alters the element nomenclature and the formulation options available, which are stated in section 4.3.

The model is validated by comparing the computed natural frequencies and global mode shapes with the ambient vibration measurements reported by Xu, Ko and Zhang [28] and by Duan et al. [17], and by checking the dead-load equilibrium against the design values. The procedure, the static checks and the mode-by-mode comparison are presented in section 4.10.

4.2 Global geometry

The geometry of the model is referred to a right-handed Cartesian frame XYZ , shown on the overall view of the model in Figure 4.1. Its origin lies at the intersection between the base of the Ma Wan tower and the vertical axis of symmetry of the tower. The X -axis runs along the bridge longitudinal axis, from the Ma Wan side toward the Tsing Yi side; the Y -axis is transverse, from the north edge of the deck to the south edge; the Z -axis is vertical and points upward. Every position reported below is expressed in this frame.

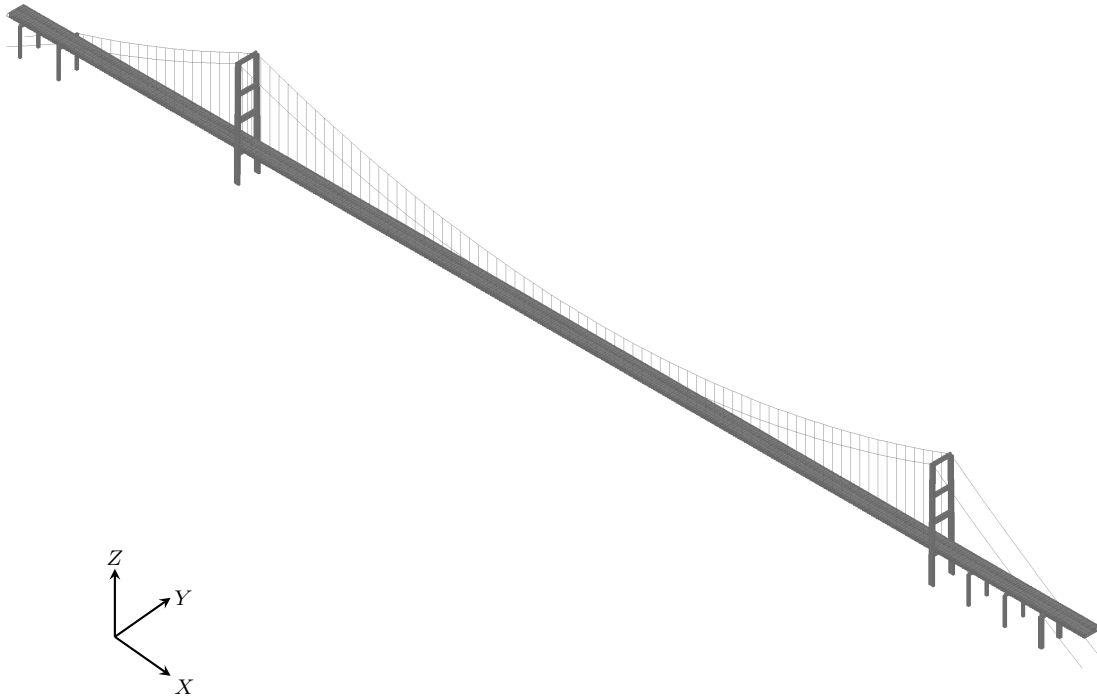


Figure 4.1: Isometric view of the finite element model of the Tsing Ma Bridge

4.2.1 Longitudinal layout

The longitudinal layout of the model, illustrated in Figure 4.2, follows the span arrangement of the Tsing Ma Bridge documented by Duan et al. [17] and by Xu and Xia [22].

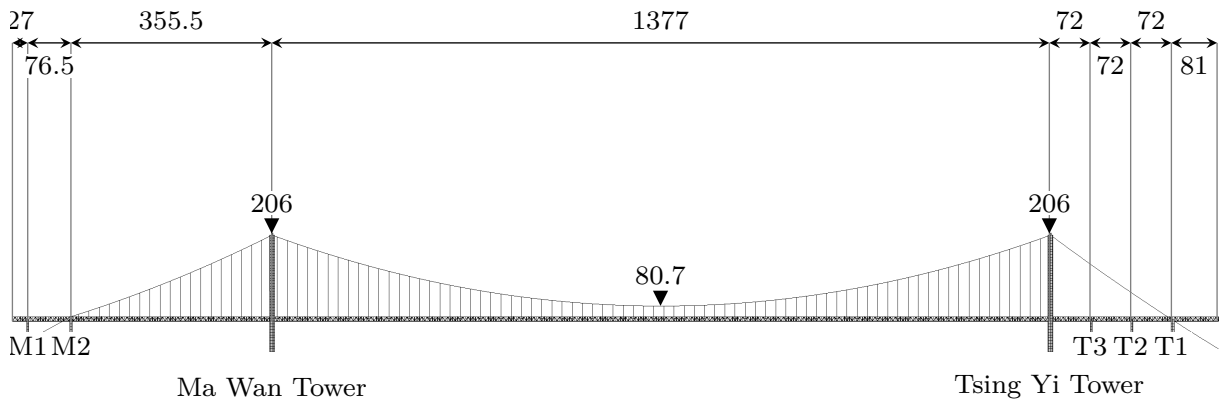


Figure 4.2: Longitudinal elevation of the finite element model, with dimensions in meters.

The main span measures 1377 m and is delimited by the two towers, placed at $x = 0$ (Ma Wan) and $x = 1377$ m (Tsing Yi). The anchorages close the structure at $x = -459$ m

on the Ma Wan side and at $x = 1674$ m on the Tsing Yi side. The two approach spans are therefore unequal, 459 m on the Ma Wan side and 297 m on the Tsing Yi side. These lengths follow from the uniform 4.5 m cross-frame spacing applied to the model: the nominal approach spans of 455 m on the Ma Wan side and 300 m on the Tsing Yi side reported by Duan et al. [17] become 459 m (102×4.5) and 297 m (66×4.5) once discretised, while the main span of 1377 m (306×4.5) is reproduced exactly.

Five piers complete the supporting system. Two of them, M1 and M2, carry the Ma Wan approach span; the other three, T1, T2 and T3, carry the Tsing Yi approach span. The cable saddle on pier M2 sits 355.5 m from the Ma Wan tower, and pier M1 stands a further 76.5 m away, at $x = -432$ m. On the Tsing Yi side the three piers are equally spaced at 72 m, with T3 nearest the tower. Table 4.1 collects these positions.

Element	x (m)
Ma Wan anchorage	-459
Pier M1 (Ma Wan side)	-432
Pier M2 (Ma Wan side, cable saddle)	-355.5
Ma Wan tower	0
Tsing Yi tower	1377
Pier T3 (Tsing Yi side)	1449
Pier T2 (Tsing Yi side)	1521
Pier T1 (Tsing Yi side)	1593
Tsing Yi anchorage	1674

Table 4.1: Longitudinal position in the reference frame of the model

4.2.2 Transverse layout

The deck is 41 m wide in the Y direction, from $y = 0$ at the north edge to $y = 41$ m at the south edge, a width consistent with the cross-section given by Duan et al. [17] and by Xu and Xia [22]. The load-bearing frame is modelled out to the edges of the two top-plate overhangs, at $y = 5.15$ m and $y = 35.85$ m; the strips beyond, from $y = 0$ to 5.15 m and from $y = 35.85$ to 41 m, carry the corrugated fairing, which provides shape and mass but no structural stiffness and is therefore not modelled as a frame [17] (subsection 4.7.1). The discretisation follows seven longitudinal nodal lines, each placed on a principal structural element, drawn in the typical cross-section of Figure 4.3. The two outer longitudinal trusses lie at $y = 7.5$ m and $y = 33.5$ m, 26 m apart. The two inner longitudinal trusses lie at $y = 13.8$ m and $y = 27.2$ m, 13.4 m apart. The seventh line is the deck centreline at $y = 20.5$ m, the geometric centre of the section, about which the outer trusses, the inner trusses and the main cables are symmetric.

The two main cables run parallel to the longitudinal axis at $y = 2.5$ m and $y = 38.5$ m, 36 m apart. This spacing matches the value reported by Duan et al. [17]. The suspension corners of the main cross-frames lie on these same two transverse lines, directly below

the cables. The suspenders hang the deck at the main cross-frames; their attachment is detailed in subsection 4.8.2. Table 4.2 lists the transverse coordinates.

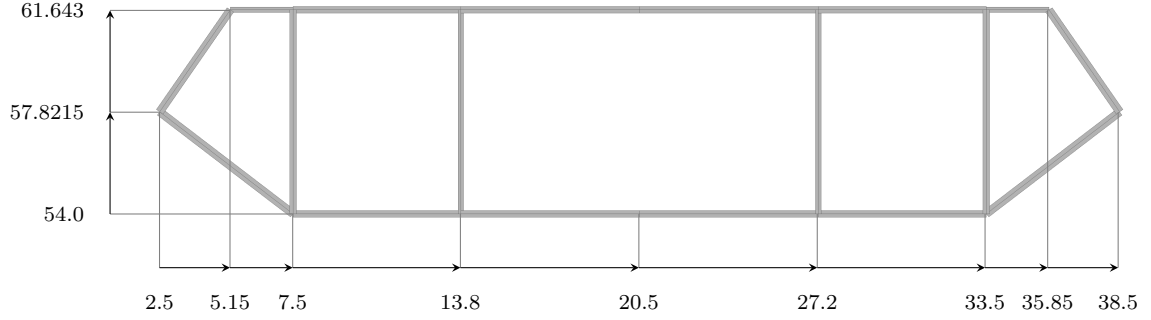


Figure 4.3: Transverse cross-section of a typical cross-frame in the model, with dimensions in meters.

Element	y (m)
North main cable and suspension corner	2.5
North overhang edge	5.15
North outer longitudinal truss	7.5
North inner longitudinal truss	13.8
Deck centreline	20.5
South inner longitudinal truss	27.2
South outer longitudinal truss	33.5
South overhang edge	35.85
South main cable and suspension corner	38.5

Table 4.2: Transverse position of the nodal lines of the deck discretisation, of the main cables and of the suspension corners. The deck edges at $y = 0$ and $y = 41$ m are the tips of the non-structural fairing and are not nodes of the model.

4.2.3 Vertical layout and main-cable geometry

In the vertical direction the reference plane $z = 0$ is set at the base of the towers. The deck occupies the band between $z = 54$ m and $z = 61.643$ m, the elevations of the lower and upper chords of the cross-frames, so that its structural depth is 7.643 m, as reported in [22] (§2.5.2.2). Each main cross-frame carries two octagonal corners at $z = 57.8215$ m, halfway between the two plates, at the transverse positions of the two main cables; these are the vertices of the octagonal outline of the cross-frame (Figure 4.3). The suspenders attach at these octagonal corners, as detailed in subsection 4.8.2.

The towers rise to $z = 206$ m, the elevation of the cable saddles, while the two cable anchorages sit at $z = 5$ m.

In the main span the cable is taken as a parabola. It passes through the two tower saddles at $(0, 206)$ and $(1377, 206)$ and reaches its vertex at midspan, $(688.5, 80.7)$ m. The profile is

$$z(x) = z_{\text{mid}} + a(x - x_{\text{mid}})^2, \quad a = \frac{z_{\text{top}} - z_{\text{mid}}}{x_{\text{mid}}^2}, \quad (4.1)$$

with $z_{\text{top}} = 206$ m, $z_{\text{mid}} = 80.7$ m and $x_{\text{mid}} = 688.5$ m. The initial sag is then $s_0 = 125.3$ m and the sag-to-span ratio $s_0/L = 0.091$. This parabola is the initial geometry; the equilibrium profile under permanent load is obtained from the non-linear static analysis (section 4.10).

The side-span cables follow the description of Xu, Ko and Zhang [28] and of Xu and Xia [22]. On the Ma Wan side the cable descends from the tower saddle at $z = 206$ m, over the pier M2 saddle at $z = 62$ m, to the anchorage at $z = 5$ m. On the Tsing Yi side it runs from the tower saddle directly to the anchorage.

4.2.4 Cross-frame and suspender discretisation

The longitudinal discretisation uses a uniform cross-frame spacing of 4.5 m. This follows the typical 18 m deck segment of [17] (§3.1.1) and [22], made of five cross-frames spaced 4.5 m apart: a main cross-frame at the centre, with two intermediate cross-frames on each side. The model contains 475 cross-frames, indexed from 0 at the Ma Wan anchorage to 474 at the Tsing Yi anchorage, and therefore 474 bays. These split into 102 bays over the Ma Wan approach span, 306 over the main span and 66 over the Tsing Yi approach span. Cross-frame 102 falls on the Ma Wan tower, cross-frame 408 on the Tsing Yi tower.

Suspenders occur at every fourth cross-frame, a longitudinal spacing of 18 m [17, 28]. The main span carries 76 pairs, at cross-frames 104, 108, $\dots$, 404; the first of them stands 9 m from the Ma Wan tower. A further 19 pairs hang on the Ma Wan side, at cross-frames 26, 30, $\dots$, 98. The Tsing Yi side has none, because there the deck rests on the piers T1, T2 and T3.

4.3 Finite element types

The model draws on three families of elements from the Ansys Mechanical APDL library: beam, link and shell. The towers, the piers and the stiffening system of the deck are carried by beam elements, the cables and hangers by links, and the deck plating by shells. This description follows the detailed modelling line established for the Tsing Ma Bridge by Duan et al. [17] and by Xu and Xia [22], rather than the equivalent spine idealisation. Four element types are declared, two of which share the **BEAM188** formulation under different interpolation orders. Across these types the assembled mesh comprises 21 328 elements connected at 7 649 nodes.

4.3.1 Beam elements

BEAM188 represents the towers, the piers and the entire stiffening framework of the deck, the latter including the cross-frames, the longitudinal trusses and the sway and plane bracings. The element is a two-node beam based on Timoshenko theory, so that transverse shear deformation is included rather than neglected as in the Euler-Bernoulli model; this is relevant for the stocky tower legs and for the members of the deck [30]. Six degrees of freedom act at each node. The optional seventh degree of freedom, the warping, is enabled through KEYOPT(1)=1; Ansys recommends activating it for the sections with thin walls used here, where neglecting warping can misrepresent the torsional behaviour [30].

The two beam types differ only in interpolation order. Towers and piers use the quadratic option (KEYOPT(3)=2), a three-node formulation that captures the bending of the tall legs with a coarse mesh. The deck framework keeps the default linear interpolation, since the close spacing of the cross-frames already provides a fine subdivision and the accuracy there rests on mesh density rather than on element order.

4.3.2 Cable and hanger elements

The main cables and the hangers are modelled with LINK180, a three-dimensional spar carrying two nodes, each with three translational degrees of freedom. The element transmits axial force only and has no bending stiffness, appropriate to a cable system [30]. The cables are pretensioned; the equilibrium configuration of the suspended structure under self-weight, on which the global stiffness of the bridge depends, is obtained through the geometrically non-linear static analysis described in section 4.10.

4.3.3 Deck plate elements

The orthotropic deck plating, that is the top and bottom steel deck plates, is modelled with SHELL181, a four-node shell element with six degrees of freedom at each node, suited to thin and moderately thick shells [30]. The full-integration option with incompatible modes is selected through KEYOPT(3)=2, in place of the default reduced integration. The incompatible internal modes improve the response of the element in in-plane bending and lessen its sensitivity to distorted meshes, which suits the wide plate strips that span between the trusses [30].

4.4 Material properties

All materials are treated as linear elastic. No inelastic law is introduced, since the analyses reported in this chapter concern the dead-load equilibrium and the small-amplitude vibration modes, for which a linear elastic description is sufficient. Seven materials are

defined. Five are isotropic, and they describe the reinforced concrete of the towers and piers, the steel of the two main cables, the wire rope of the hangers, the structural steel of the deck framework, and a deliberately stiff material assigned to the edge members of the main cross-frames. The remaining two are orthotropic and stand for the stiffened deck plating, which is represented through equivalent smeared properties.

The isotropic materials are collected in Table 4.3, each defined by its modulus of elasticity E , Poisson's ratio ν and density ρ . None of these quantities is calibrated in the present work. The elastic moduli are physical or design values reported in the literature on the bridge, and the densities are those of the global-model material table of Duan et al. [17].

The reinforced concrete of the towers and the piers takes $E = 34$ GPa, the value adopted by Xu and Xia [22], with a density of 2500 kg m^{-3} [17]. The two cable materials are distinguished by their construction: the main cables, formed of parallel galvanised wires, take $E = 196$ GPa, whereas the hangers, made of wire rope, are more flexible at 134 GPa, both as estimated by Xu, Ko and Zhang [28]; both are assigned the steel density of 7850 kg m^{-3} [17]. The deck framework uses a structural steel with $E = 205$ GPa and a density of 9000 kg m^{-3} . This density is larger than that of structural steel because it is an equivalent value: following Duan et al. [17], the mass of the pavement above the deck plates and of the many accessory components is smeared into the density of the deck steel rather than applied as separate loads, which raises it from 7850 to 9000 kg m^{-3} while the modulus is kept at that of steel.

The edge members of the main cross-frames receive a modulus ten times as large, 2050 GPa, together with the same equivalent density of 9000 kg m^{-3} . The large modulus is not a physical property of a real steel; it is a modelling device that renders these members effectively rigid and so reproduces the heavily stiffened condition of the deck at the suspension points, where the suspender force is delivered to the cross-frame. The device is taken from Duan et al. [17], who define a dedicated rigid-steel material, with an elastic modulus ten times that of steel, for the edge members of the main cross-frames for the same reason. The main cables also carry a coefficient of thermal expansion of $1.2 \times 10^{-5} \text{ K}^{-1}$, which the analysis uses to introduce the cable pretension through an imposed temperature change (section 4.10).

Component	Material	E [GPa]	ν	ρ [kg m^{-3}]
Towers and piers	reinforced concrete	34	0.20	2500
Main cables	parallel wire steel	196	0.30	7850
Hangers	wire rope steel	134	0.30	7850
Deck framework	structural steel	205	0.30	9000
Cross-frame edge members	rigid steel	2050	0.30	9000

Table 4.3: Isotropic materials of the model.

The deck plating is described by two equivalent orthotropic materials, listed in Table 4.4. Their in-plane constants, the moduli E_x , E_y and the shear modulus G_{xy} , are the two stiffened-plate parameter sets that Xu and Xia obtained by model updating, here labelled Set 1 and Set 2 [22]. These are not the constants of a homogeneous plate but equivalent quantities that smear the stiffeners and the secondary steel of the orthotropic deck over a plain plate. The density is the equivalent steel density of 9000 kg m^{-3} of Duan et al. [17], the value also used for the deck framework, so that the smeared pavement and accessory mass is treated consistently across the deck. The deck regions to which each set is assigned are described in section 4.7.

Property	Set 1	Set 2
$E_x = E_z$ [GPa]	185	198
E_y [GPa]	1810	1807
G_{xy} [GPa]	120	901.7
$G_{yz} = G_{xz}$ [GPa]	80	80
$\nu_{xy} = \nu_{yz}$	0.031	0.033
ν_{xz}	0.3	0.3
ρ [kg m^{-3}]	9000	9000

Table 4.4: Equivalent orthotropic materials of the deck plating.

4.5 Cross-sections

The cross-sections fall into four groups, one for each element type. The tower legs and the piers take beam sections, hollow rectangular for the legs and solid rectangular for the other members. The deck framework takes solid rectangular beam sections. The deck plates are given shell thicknesses, and the cables and hangers cross-sectional areas. The steel members of the deck are not rectangular in reality: as described by Xu and Xia [22], the chords of the longitudinal trusses are box sections, their verticals and diagonals are I-sections, the cross-frame chords are mostly T-sections, the struts and the inclined edge members are I-sections, the cross bracings are box sections, and the sway bracings are circular hollow sections.

The dimensions of these shapes are not reported in the available literature, so the true sections could not be reproduced directly. An equivalent solid rectangular beam section is adopted for each member instead. The cross-sectional shapes and the structural families are those described by Xu and Xia [22], whereas the equivalent rectangular dimensions are a modelling choice introduced in the present work. They are adjusted iteratively so that two measurable reference quantities are reproduced, the dead load carried by the deck and the natural frequencies of the bridge; the resulting dead load, cable forces and frequencies, with their comparison against the design and field values, are reported in section 4.10. The tower-leg, pier, cable and suspender dimensions are not calibrated,

but taken from the design drawings and the literature as stated below. The equivalent rectangular representation keeps the section input compact, at the cost of reproducing the true shapes only in an equivalent sense.

The tower and pier sections are collected in Table 4.5. Each tower leg is modelled as a hollow rectangular box, 9.0 by 3.5 m in the lower part and 9.0 by 3.0 m above, with wall thicknesses of 1.5 and 1.2 m. The four horizontal cross-beams that link the legs decrease in size towards the top, from 4.2 by 12.0 m down to 2.8 by 7.0 m. The piers are solid rectangular members.

Member	Section	Dimensions [m]
Tower leg, lower part	hollow box	9.0×3.5 , walls 1.5
Tower leg, upper part	hollow box	9.0×3.0 , walls 1.2
Tower cross-beams	solid rect.	4.2×12.0 to 2.8×7.0
Piers M1, M2, T2, T3	solid rect.	4.0×4.0
Pier T1	solid rect.	6.0×6.0

Table 4.5: Beam sections of the towers and piers.

The equivalent sections of the deck framework are given in Table 4.6, each alongside the real shape it represents. The edge members of the cross-frames carry the rigid material of Table 4.3, described in section 4.4, which renders the heavily stiffened suspension joint effectively rigid. The sway and cross-bracing sections were sized to match the axial area of the circular and box members they replace.

Member	Real shape	Equivalent rect. [m]
Outer truss, upper chord	box	0.35×0.50
Outer truss, lower chord	box	0.30×0.45
Outer truss, diagonal	I	0.20×0.40
Inner truss, chord	box	0.25×0.30
Inner truss, diagonal	I	0.15×0.20
Cross-frame, upper chord (centre)	T	0.25×0.25
Cross-frame, lower chord (centre)	T	0.25×0.25
Cross-frame, chord (overhangs)	T	0.20×0.20
Cross-frame, outer strut	I	0.25×0.35
Cross-frame, inner strut	I	0.20×0.25
Cross-frame, edge member	I	0.25×0.30
Sway bracing	circular hollow	0.10×0.10
Cross and plane bracing	box	0.10×0.15

Table 4.6: Equivalent rectangular beam sections of the deck framework and the real cross-sectional shapes they stand for.

The deck plates take a shell thickness of 13 mm; the central lower strip that carries the railway tracks over the vent openings is given a reduced equivalent thickness of 4 mm, into

which the track plates, rail waybeams and tee diaphragms are smeared (subsection 4.7.6). The link elements take the cross-sectional areas given in the design drawings: 0.759 m^2 for a main cable in the main span and 0.801 m^2 in the side spans, as reported by Xu [28], and 0.018 m^2 for a suspender unit, the value used by Xu and Xia [22].

4.6 Towers and piers

Two reinforced concrete towers carry the main cables, one at each end of the main span. The Ma Wan and the Tsing Yi towers share the same geometry [22]. Each tower is a portal frame formed by two legs joined by four deep portal beams (Figure 4.4). In the model the legs rise from the base at $z = 0$ to the saddle at $z = 206 \text{ m}$, and the portal beams are placed at the deck level ($z = 54 \text{ m}$) and at $z = 107, 152$ and 206 m . The legs lean towards each other, their transverse spacing reducing from 40 m at the base to 36 m at the top, a slope close to one in one hundred [22]. This top spacing matches the distance between the two main cables, so that each cable meets its saddle above the corresponding leg.

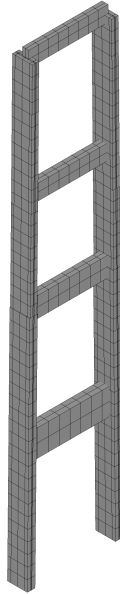


Figure 4.4: Isometric view of the finite element model of a bridge tower.

The cross-section of a real tower leg is far from prismatic. Its breadth tapers from about 18 m at the base to 9 m at the top while the width stays at 6 m , and two rectangular hollow shafts run inside each leg above roughly 15 m from the base [22]. Carrying this geometry into a beam model adds little at the global scale, so the legs are given uniform hollow box sections, in the manner of the averaging adopted by Xu et al. [28]: a $9.0 \times 3.5 \text{ m}$ box below $z = 107 \text{ m}$ and a $9.0 \times 3.0 \text{ m}$ box above it (Table 4.5). The four portal beams instead keep their design proportions, decreasing in four steps from $12 \times 4.2 \text{ m}$ at the deck level to $7 \times 2.8 \text{ m}$ at the top [22]. The storey levels are reported in Figure 4.5.

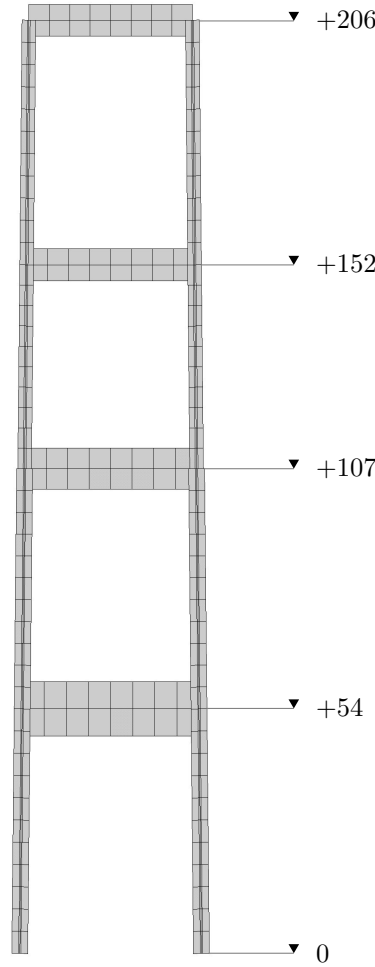


Figure 4.5: Elevations of the portal beam levels of the tower model above the base in meters.

The deck is supported on five piers, two on the Ma Wan side and three on the Tsing Yi side. Piers M1 and M2 stand 432 m and 355.5 m from the Ma Wan tower, and piers T3, T2 and T1 follow at intervals of 72 m from the Tsing Yi tower [22]. Each pier is represented by a two-legged portal frame (Figure 4.6), as in the model of Xu et al. [28], fixed at the base and closed by a transverse beam at the deck. Most piers are hollow over much of their height and are reproduced here by solid rectangular sections of 4.0×4.0 m. Pier T1 departs from this: it is almost a solid wall and was treated by Xu et al. [28] as a portal frame of very large in-plane stiffness, so it is given a larger 6.0×6.0 m section. Pier M2 differs in turn, since it alone restrains the deck laterally and carries two saddles above the deck that deflect the main cables; its legs are therefore extended above the deck to reproduce these saddle supports [28, 17].

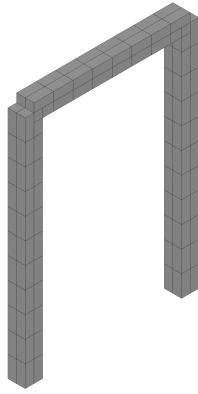


Figure 4.6: Isometric view of the finite element model of a pier.

The towers and piers are built from the quadratic beam elements of section 4.3, with each geometric segment meshed into five elements. The restraints applied at their bases are described in section 4.9.

4.7 The bridge deck

The deck is a steel structure of modular configuration, repeated along the bridge axis: transverse Vierendeel cross-frames carried on longitudinal trusses, acting together with stiffened steel plates that form the upper and lower carriageways, with a railway track system positioned in the central part of the lower deck [22, 17]. A typical segment of the prototype deck, with its principal components, is shown in Figure 4.7. In the model the deck is generated as a continuous assembly of 475 cross-frames spaced 4.5 m apart, running from $x = -459$ m at the Ma Wan anchorage to $x = 1674$ m at the Tsing Yi anchorage. A representative length of the assembled deck is shown in Figure 4.8. The cross-frames are indexed by the order in which they are created: cross-frame 0 lies at the Ma Wan anchorage, cross-frame 102 at the Ma Wan tower, cross-frame 408 at the Tsing Yi tower, and cross-frame 474 at the Tsing Yi anchorage. The cross-section is shown in Figure 4.3, and all the member sections are collected in Table 4.6.

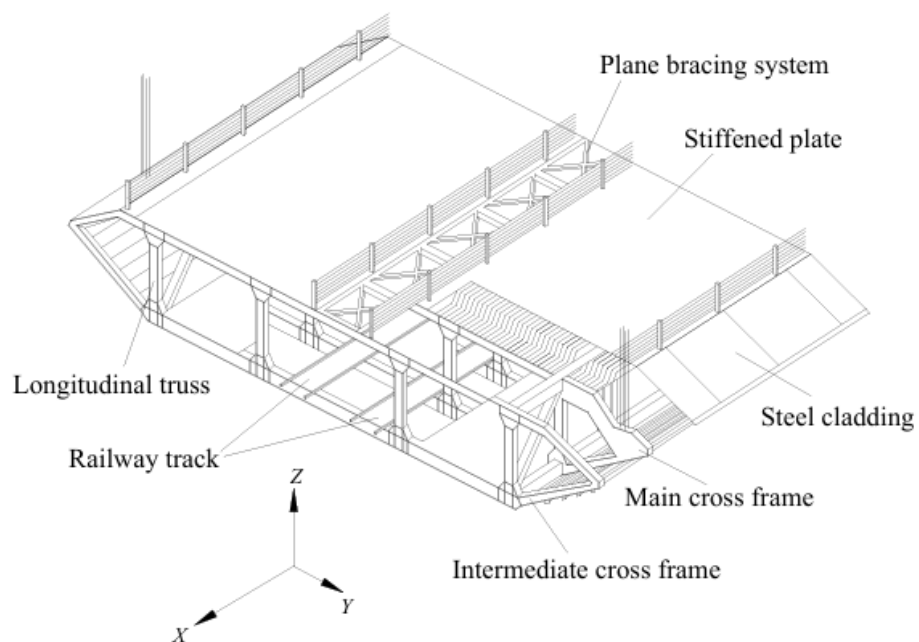


Figure 4.7: Typical suspended deck segment of the Tsing Ma Bridge, with its principal structural components, reproduced from Xu and Xia [22].

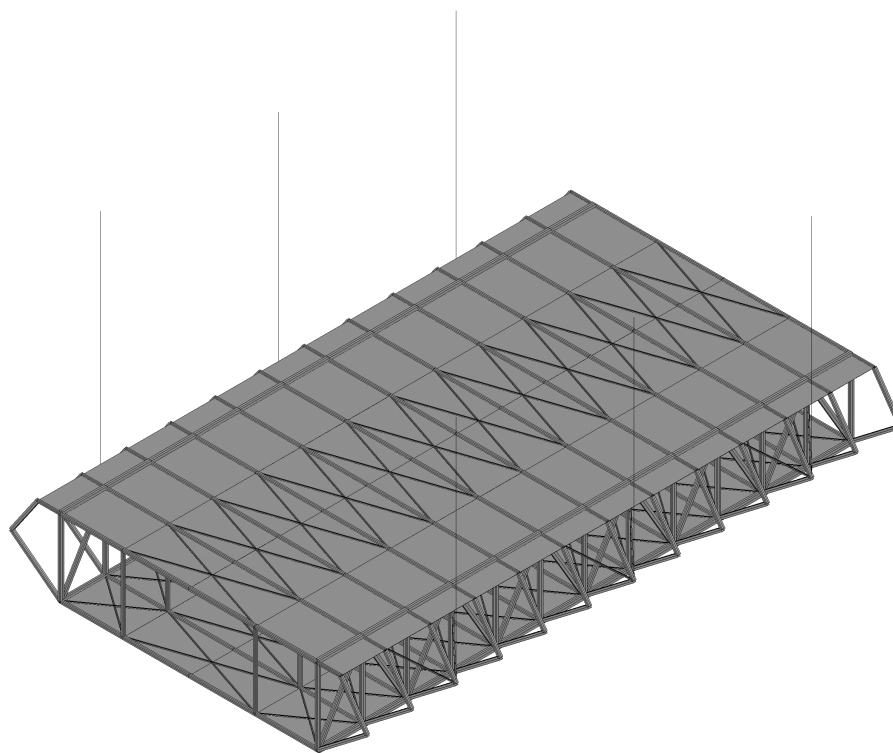


Figure 4.8: Finite element model of a representative length of the deck.

4.7.1 Cross-frames

Each cross-frame is a Vierendeel frame in the transverse vertical plane, following the description of the deck given by Xu and Xia [22]. It spans transversely between the two octagonal corners at $y = 2.5$ and $y = 38.5$ m and vertically from $z = 54$ m at the lower chord to $z = 61.643$ m at the upper chord, a structural depth of 7.643 m. The two octagonal corners lie at mid-height between the chords, $z = 57.8215$ m, and are the points at which the suspenders are attached (subsection 4.8.2).

The upper chord runs between the two overhang corners, from $y = 5.15$ to $y = 35.85$ m. It is made of four central segments, between the outer trusses at $y = 7.5$ and 33.5 m and through the inner-truss lines at $y = 13.8$, 20.5 and 27.2 m, together with two overhang segments that cantilever 2.35 m beyond the outer trusses, out to $y = 5.15$ and 35.85 m. This overhang length follows the deck cross-section reported by Xu and Xia [22] in their Figure 2.11(b). The lower chord has only the four central segments, since the bottom plate begins at $y = 7.5$ m and no overhang is present below. Two vertical outer struts join the chords at $y = 7.5$ and 33.5 m, and two vertical inner struts at $y = 13.8$ and 27.2 m; the inner struts are present in every cross-frame.

Four inclined edge members close the section against the two octagonal corners and give it its octagonal outline (Figure 4.9) [17]. On each side an upper inclined edge member runs from the overhang corner of the top chord ($y = 5.15$ or 35.85 m, $z = 61.643$ m) down to the octagonal corner, and a lower inclined edge member runs from the outer end of the bottom chord ($y = 7.5$ or 33.5 m, $z = 54$ m) up to the same corner. At the 95 main cross-frames, where the suspenders attach, the four edge members carry the rigid material of Table 4.3, which reproduces the heavily stiffened suspension joint; at the remaining 380 intermediate cross-frames they carry the ordinary deck steel, a distinction also drawn by Duan et al. [17].

The two overhangs are short cantilevers, 2.35 m long, that carry the edge strip of the top highway plate; they neither support the suspenders, which are attached at the octagonal corners, nor carry the railway, which runs on the central part of the lower deck. Beyond the overhang corners, from $y = 0$ to 5.15 m and from $y = 35.85$ to 41 m, the section is closed by corrugated fairing sheets. Following Duan et al. [17], these sheets cover the edge members and protect the deck against wind and rain; they define the aerodynamic profile and carry their own weight, but they do not contribute to the structural stiffness. They are therefore not modelled as part of the frame, and their mass is accounted for through the equivalent densities of the deck steel (section 4.4).

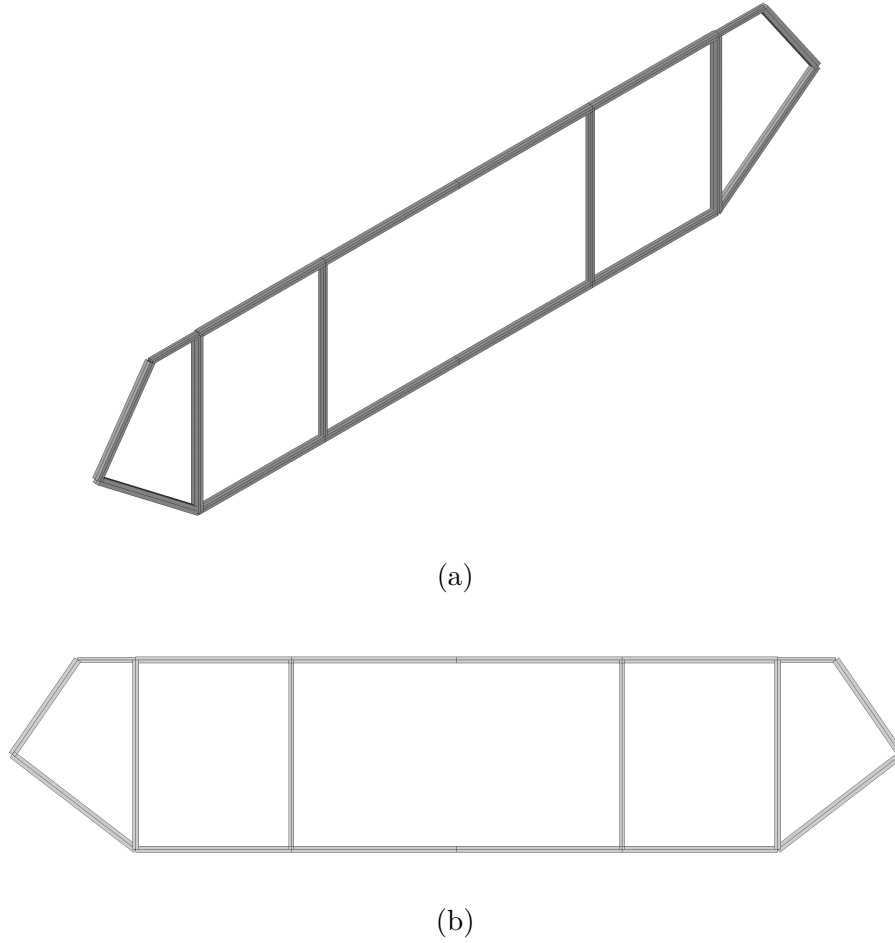


Figure 4.9: Finite element model of a cross-frame: (a) isometric view; (b) transverse view. The dimensions of the cross-section are given in Figure 4.3.

4.7.2 Outer longitudinal trusses

The two outer longitudinal trusses stand 26 m apart, at $y = 7.5$ and 33.5 m, and extend over the whole length of the deck. They act as the main girders, providing the vertical bending stiffness together with the deck plates [22]. Each truss has an upper and a lower chord, which connect the corresponding nodes of consecutive cross-frames, and one diagonal per bay. The slope of the diagonals alternates from one bay to the next, so that the diagonals, together with the outer struts of the cross-frames acting as the vertical members, triangulate the truss; this single-diagonal layout follows the vertical and diagonal members reported by Xu and Xia [22], as shown in Figure 4.10.

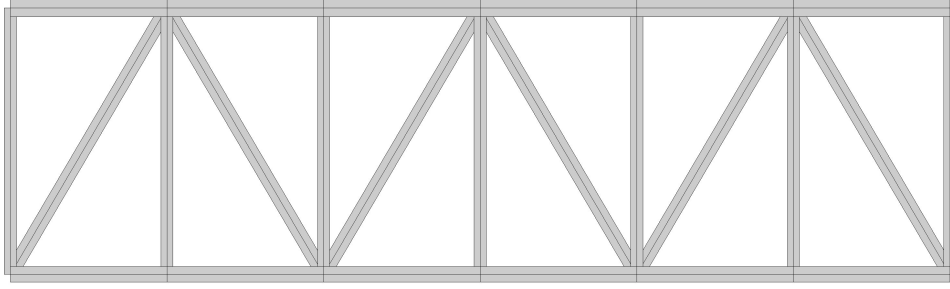


Figure 4.10: Finite element model of an outer longitudinal truss in the longitudinal view.

4.7.3 Inner longitudinal trusses

The two inner longitudinal trusses stand 13.4 m apart, at $y = 13.8$ and 27.2 m. Unlike the outer trusses, they are present only near the towers, as described by Xu and Xia [22]. In the model they span 24 bays near the Ma Wan tower, about 108 m, and 13 bays near the Tsing Yi tower, about 58.5 m, in agreement with the lengths reported by Duan et al. [17]. Each inner truss has upper and lower chords and one diagonal per bay, alternating in slope as in the outer trusses, with the inner struts of the cross-frames acting as the vertical members. Both stretches are shown in Figure 4.11, where the inner trusses are drawn in red.

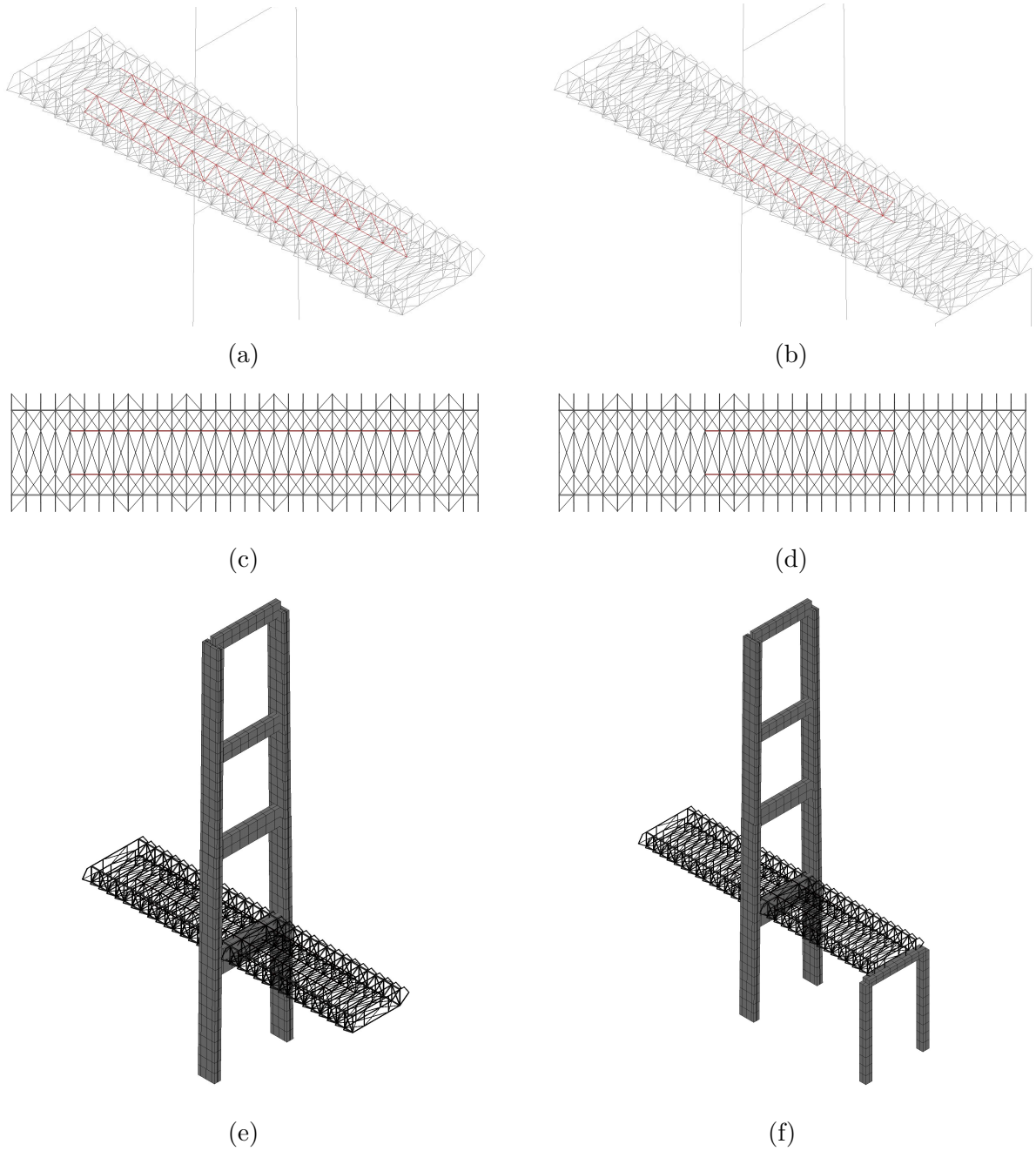


Figure 4.11: Finite element model of the inner longitudinal trusses, present only in the deck regions adjacent to the towers, shown for the Ma Wan side (left column) and the Tsing Yi side (right column): (a, b) isometric views and (c, d) plan views of the deck wireframe, with the inner trusses drawn in red; (e, f) isometric views of the same deck region together with the tower. In (a)-(d) the section thicknesses are switched off, so the members are drawn as lines.

4.7.4 Sway bracings

The sway bracings stabilise the deck out of its plane at the suspension points. At every main cross-frame, four members run from the two octagonal corners to the outer ends of the upper chords of the two adjacent cross-frames, as shown in Figure 4.12, two on the

north side and two on the south, which reproduces the pair of sway bracings connected from each suspension point to the adjacent cross-frames described by Xu and Xia [22] and by Duan et al. [17]. With 95 main cross-frames, the model contains 380 such members.

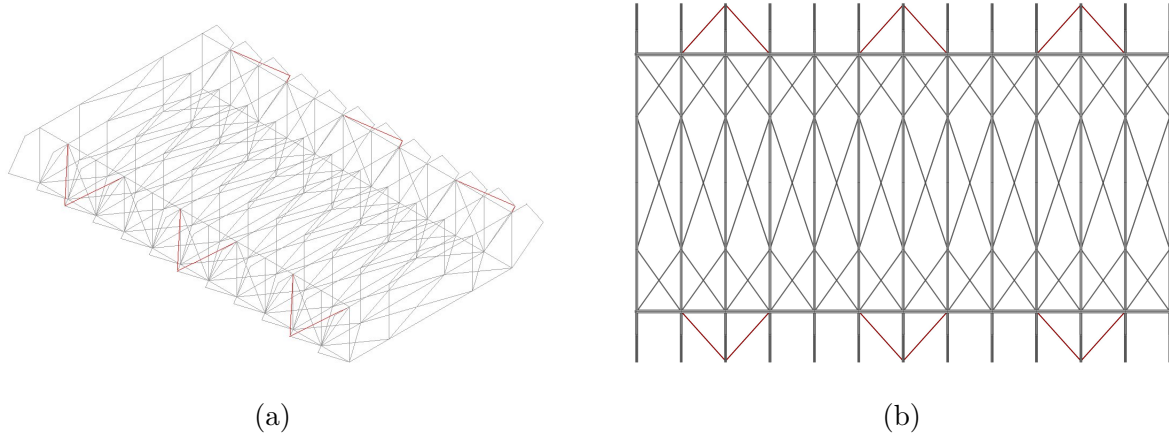


Figure 4.12: Finite element model of the sway bracing, drawn in red: (a) isometric view and (b) plan view of the deck.

4.7.5 Horizontal cross bracings

The carriageway planes are braced by horizontal cross bracings, arranged as crossed diagonals between consecutive cross-frames. Four rows are modelled: one at the centre of the upper plane, one at the centre of the lower plane, and two at the outer parts of the lower plane, between the outer and the inner truss lines on each side, as shown in Figure 4.13. The row at the centre of the upper plane follows the central cross bracing of the top deck reported by Xu and Xia [22], while the bottom centre and the two bottom outer rows follow the bottom-deck cross bracing described by Duan et al. [17] and by Xu and Xia [22]. These bracings span the vent openings of the lower deck and tie the plates together, contributing to the transverse shear stiffness [22].

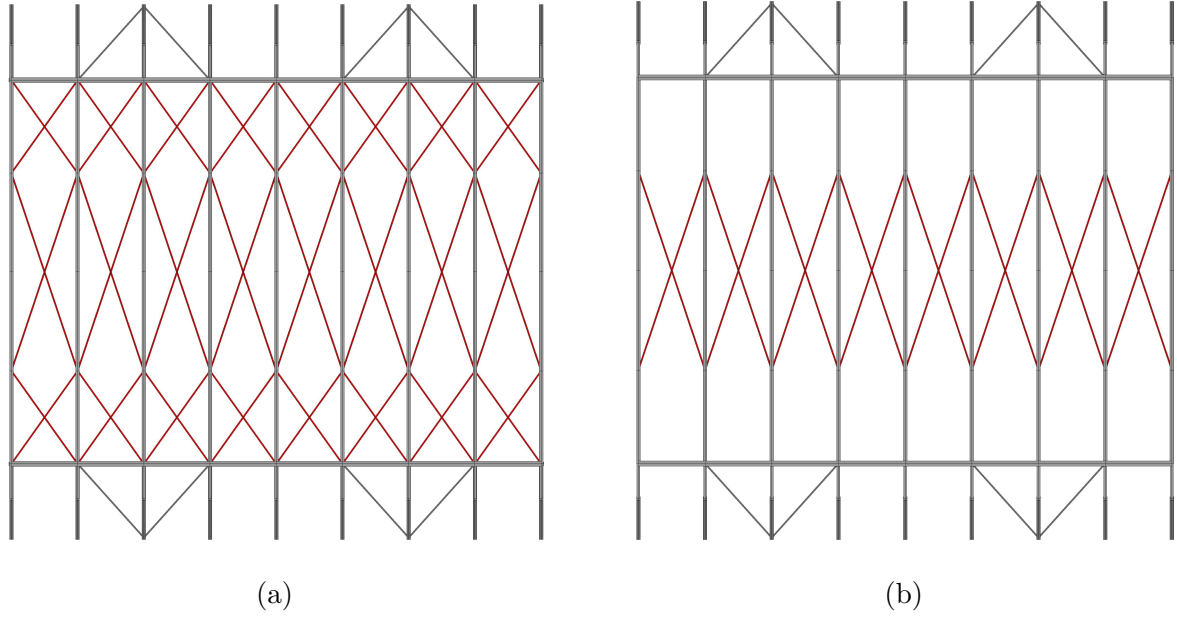


Figure 4.13: Finite element model of the horizontal plane bracing, drawn in red, seen in plan: (a) the bottom plane, (b) the top plane.

4.7.6 Orthotropic deck plates

The deck plates are represented by equivalent orthotropic shell elements, as shown in Figure 4.14. The two sets of equivalent orthotropic properties listed in Table 4.4, whose in-plane stiffness is taken from the model updating of Xu and Xia [22] and whose density is the equivalent steel density of Duan et al. [17] (section 4.4), are assigned as follows: Set 1 to the top central plate and to the entire bottom plate, and Set 2 to the two top overhangs.

At the upper level the plate spans between the two overhang edges, from $y = 5.15$ to 35.85 m, covering the central part between the outer trusses together with the two overhangs; beyond the overhang edges the deck carries only the non-structural fairing (subsection 4.7.1). At the lower level the layout reproduces the real deck. The two outer bays, between the outer and the inner truss lines, carry highway plates along the whole length, as reported by Duan et al. [17]. The central bays are plated at full thickness only where the prototype has no vent openings, that is near the two towers and along the Tsing Yi approach span, where Xu and Xia report that the deck plates extend over the full width [22]. Over the main span and the Ma Wan side span the centre of the lower deck carries the two railway tracks above the vent openings; there the track plates, rail waybeams and tee diaphragms described by Duan et al. [17] are condensed into a single shell of reduced equivalent thickness (section 4.5). This thickness, like the equivalent beam sections of the framework, is fixed by the calibration on the dead load and the natural frequencies described in section 4.10.

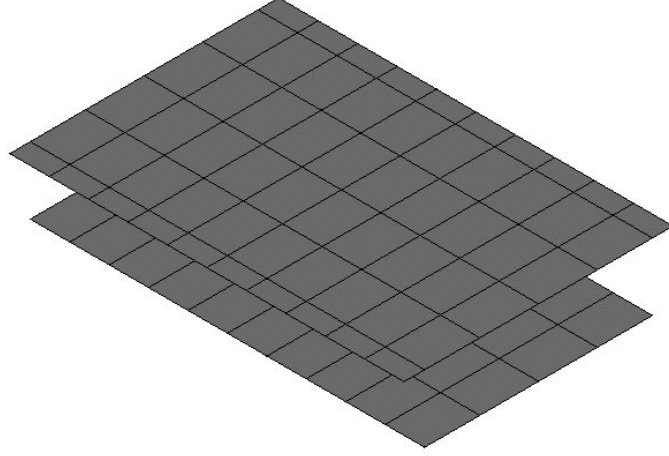


Figure 4.14: Finite element model of the deck plates: isometric view of the shell-element.

4.8 Suspension system

The dead load of the deck is carried to the towers and the anchorages by two parallel main cables and by the suspender units that hang the deck from them. Both families are discretised with LINK180 (element Type 2), a two-node spar that transmits axial force only and has no rotational degrees of freedom at its ends. This reproduces the tension-only behaviour adopted for the cables by Xu, Ko and Zhang [28], who modelled the system with cable elements accounting for the geometric stiffness induced by the internal tension, and it is consistent with the rotational release applied to the cable beam elements by Xu and Xia [22]. Three cross-sections are assigned: the main span cable uses Section 5 with an area of 0.759 m^2 , the side span cables use Section 7 with 0.801 m^2 , and the suspenders use Section 6 with 0.018 m^2 . The modulus of elasticity of each material, 196 GPa for the cables and 134 GPa for the suspenders, was introduced in section 4.4.

4.8.1 Main cables

The two cables run in vertical planes placed at $y = 2.5 \text{ m}$ and $y = 38.5 \text{ m}$, a lateral separation of 36 m that matches the design spacing given by Xu, Ko and Zhang [28] and by Duan et al. [17]. Each cable is built as a single continuous chain of truss elements that starts at the Ma Wan anchorage, passes over the saddle on pier M2, rises over the Ma Wan tower saddle, spans the main opening, crosses the Tsing Yi tower saddle and ends at the Tsing Yi anchorage. The cross section changes with the span: the elements over the main one carry Section 5, while those over the two approach spans carry Section 7, reflecting the 91 strands of the main span and the 97 strands of the side spans documented by Xu, Ko and Zhang [28].

Over the main span the cable nodes lie on a parabola fitted to the dead-load profile,

with the lowest point at mid-span ($x = 688.5$ m) at an elevation of 80.7 m rising to the saddles at the tower tops at 206 m, which gives a mid-span sag of 125.3 m. The overall geometry of the two cable planes, with the parabolic main span, the inclined approach spans and the segments running down to the anchorages, is shown in Figure 4.15.

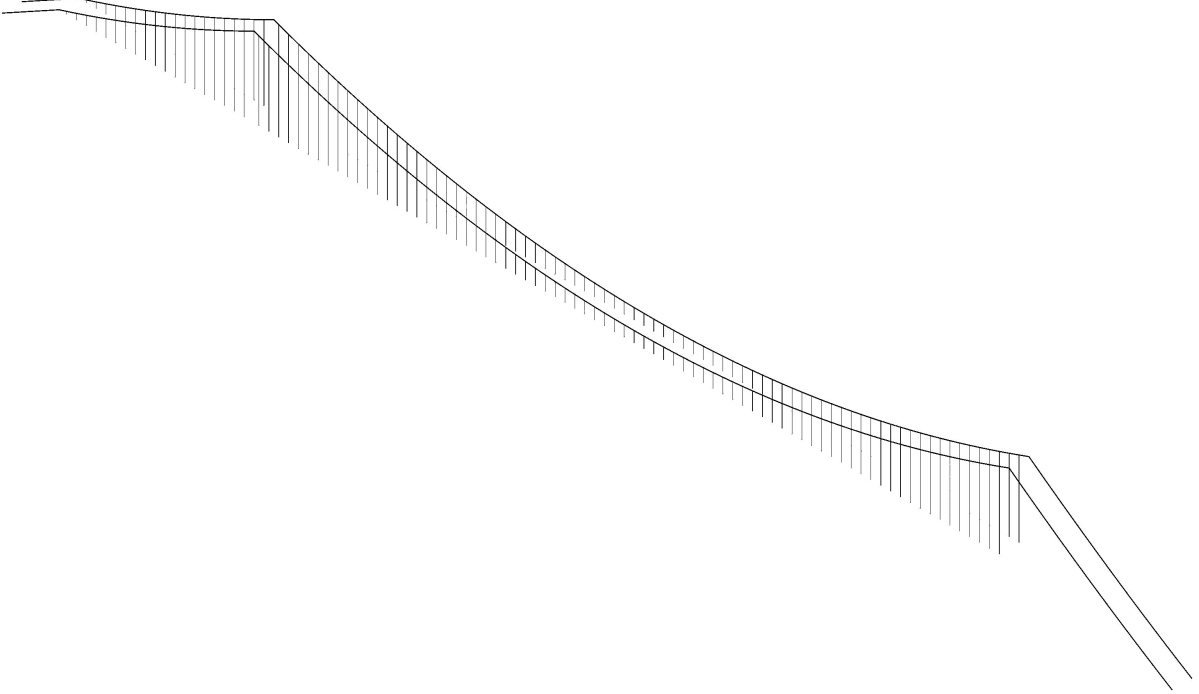


Figure 4.15: Suspension system of the finite element model.

The cable between two adjacent suspension points is represented by one truss element, so the continuous parabola is approximated by a polygonal chain whose vertices coincide with the suspender attachments, in the same way as in the cable models of Xu, Ko and Zhang [28] and Duan et al. [17]. The shape on the two approach spans follows the inclined sagged geometry described by Xu and Xia [22]. On the Ma Wan side the cable descends from the tower saddle to the saddle on pier M2 along a sagged profile that carries the side-span suspenders, and from pier M2 it continues as straight inclined segments to the anchorage at ground level, where no suspenders are present. On the Tsing Yi side the cable runs from the tower saddle directly to the anchorage as a single inclined sagged profile without any suspenders.

4.8.2 Suspenders

The deck is hung from the two main cables by suspenders, modelled with the same LINK180 spar used for the cables and assigned the hanger material of Table 4.3, with a modulus of 134 GPa. Each suspender is given a cross-sectional area of 0.018 m^2 , the value adopted by Xu and Xia [22] for a suspender unit. In the bridge a suspender unit is not a single rope but a pair of wire ropes of 76 mm diameter, so four strands in all, clamped to the

main cable and anchored to the deck by steel sockets [28]; the single link of area 0.018 m^2 stands for this complete unit.

The suspenders are placed at every fourth cross-frame, at a longitudinal spacing of 18 m, so that they coincide with the main cross-frames. There are 76 suspender units along the main span and 19 along the Ma Wan side span, between the tower and pier M2. The Tsing Yi side span carries no suspenders, since there the deck is supported on the piers rather than hung from the cable.

Each suspender runs from a cable node down to the suspension corner of the main cross-frame below it, that is the octagonal corner at $y = 2.5$ or $y = 38.5$ m and $z = 57.8215$ m where the two inclined edge members meet (subsection 4.7.1). The deck is attached to the suspender by sharing this node, rather than through an auxiliary node and coupling equations. This reproduces the load path of the real bridge, in which the suspender units are connected to the intersections of the edge members of the main cross-frame and the deck module is thereby hung from the cable, as described by Duan et al. [17] and by Xu and Xia [22]. The cable node sits directly above the suspension corner, at the same x and y , so the suspender is a purely vertical link. The rigid material of the edge members (section 4.4) acts at the same corner, representing the heavily stiffened joint through which the suspender force is delivered to the cross-frame.

The pretension of the suspension system is introduced on the cables and not on the suspenders; the force carried by each suspender follows from the dead-load equilibrium of the suspended deck, obtained in the geometrically non-linear static analysis of section 4.10.

4.9 Connections and boundary conditions

The connections among the deck, the towers, the piers and the cable system, and the external supports of the bridge, are introduced once all components have been meshed. Two mechanisms are used. Coupling equations, set with the `CP` command, force selected degrees of freedom of different nodes to take the same value, which reproduces the kinematic effect of the bearings without adding any stiffness to the model. Fixed-value constraints, set with the `D` command, impose zero displacement or rotation at the external supports. The overall scheme follows the support layout of the bridge described by Xu, Ko and Zhang [28] and by Duan et al. [17]: the deck is held in the longitudinal direction at one end only, at the Ma Wan abutment, and is free to move along the bridge axis at every other support, so that thermal expansion is accommodated toward the Tsing Yi side. The support scheme of the bridge is shown in Figure 4.16, and the restraint imposed at each connection and support is summarised in Table 4.7. The same supports are visible on the meshed model in Figure 4.17, together with the coupling equations distributed along the deck.

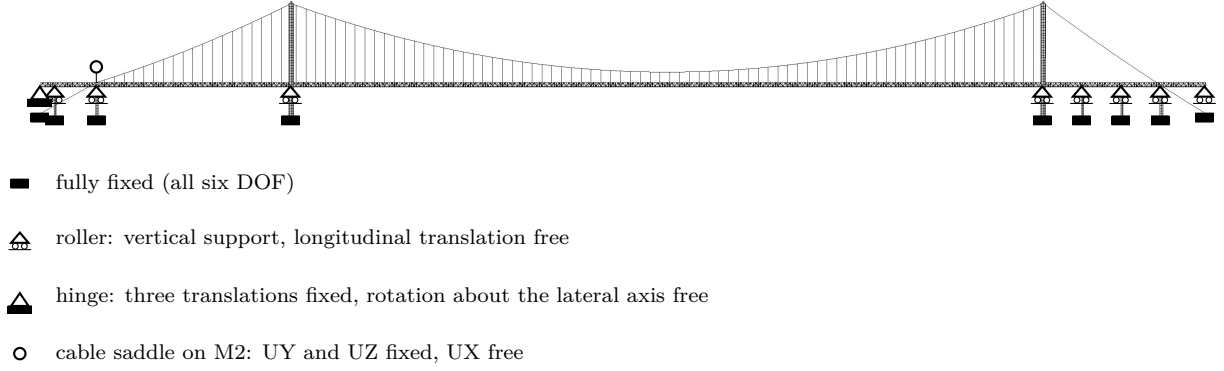


Figure 4.16: Constraint scheme of the bridge model. The symbols describe the restraint in the longitudinal–vertical plane of the elevation; the lateral restraint, which acts perpendicular to the drawing and is present at the two towers, at piers M2 and T1, and at the Tsing Yi abutment, is given in Table 4.7.

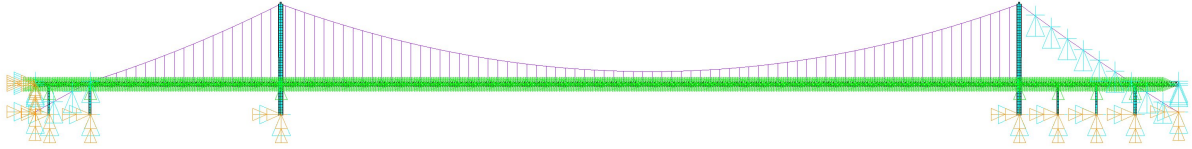


Figure 4.17: Displacement constraints and coupling equations in the finite element model. The symbols at the tower bases, the pier bases, the cable anchorages and the two deck ends mark the fixed and sliding supports, while the markers distributed along the deck correspond to the coupling equations of the cross-frame connections.

Connection or support	UX	UY	UZ	Rotations
Tower bases (Ma Wan, Tsing Yi)	fixed	fixed	fixed	fixed
Pier bases (M1, M2, T3, T2, T1)	fixed	fixed	fixed	fixed
Cable anchorages (Ma Wan, Tsing Yi)	fixed	fixed	fixed	—
Pier M2 cable saddle	free	fixed	fixed	—
Deck–tower (Ma Wan, Tsing Yi)	free	coupled	coupled	free
Deck–pier M1, T2, T3	free	free	coupled	free
Deck–pier M2, T1	free	coupled	coupled	free
Ma Wan abutment (deck end)	fixed	fixed	fixed	ROTY free
Tsing Yi abutment (deck end)	free	fixed	fixed	free

Table 4.7: Restraint applied at each connection and support of the model [17].

4.9.1 Foundations, cable saddles and anchorages

Both towers and the five piers are fully fixed at their base. Every node at foundation level ($z = 0$) under the Ma Wan tower, the Tsing Yi tower and the piers M1, M2, T3, T2 and T1 has its six degrees of freedom set to zero, which represents the fixed foundation supports adopted by Duan et al. [17] and by Xu and Xia [22]. The two main cables are

fixed in all degrees of freedom at their ends, at the Ma Wan and Tsing Yi anchorages. Following the same reasoning as Duan et al. [17], the anchorage blocks are not modelled, and each cable is simply restrained at the point where it would enter the anchorage.

At the tower tops the end nodes of the cable coincide with the saddle nodes already meshed as part of the towers, so the cable is connected there without a separate element, in the way described by Duan et al. [17] for the coupling of the saddle with the tower leg top. The saddle on pier M2, which deflects the Ma Wan side-span cable, is represented by fixing the lateral and vertical translations of the cable node at the pier top and leaving the longitudinal translation free, so that the cable can slide over the saddle. This matches the vertical and lateral coupling between pier M2 and the main cable reported by Duan et al. [17].

The two straight runs of side-span cable, from the Ma Wan anchorage to the pier M2 saddle and from the Tsing Yi tower saddle to the Tsing Yi anchorage, carry no suspenders and are almost rectilinear. Their intermediate nodes are fixed in the lateral and vertical directions, leaving only the axial translation free.

4.9.2 Deck bearings at the towers and the piers

The bearings that link the deck to the towers restrain its lateral and vertical motion and leave the longitudinal motion free, as described for both towers by Xu and Xia [22]. At each tower this is reproduced by coupling the lateral (UY) and vertical (UZ) translations of the five bottom-chord nodes of the support cross-frame and of the corresponding tower node, while the longitudinal translation (UX) is left independent. The same kinematic condition was obtained by Xu, Ko and Zhang [28] and by Duan et al. [17] through swing links and multipoint constraints for the bottom and lateral bearings; the restraint on the deck, lateral and vertical at the towers with free longitudinal sliding, is identical.

The deck to pier connections are not the same at every pier. The free-standing piers M1, T2 and T3 carry only vertical bearings, so the model couples the vertical translation of the deck bottom-chord node to the pier-top node on each side and leaves the lateral and longitudinal translations free. Piers M2 and T1 carry both vertical and lateral bearings, and for these two the lateral translation is coupled as well. This distinction follows the bearing layout of Xu, Ko and Zhang [28], who added horizontal links at piers M2 and T1 to restrain the lateral motion, and of Duan et al. [17], who list lateral bearings at M2 and T1 only. Pier M2 therefore has a dual role in the model: it provides the lateral and vertical restraint of the deck on the Ma Wan approach span and, at its top, the saddle that carries the side-span cable.

4.9.3 Abutment conditions

The two abutments close the longitudinal restraint scheme. At the Ma Wan abutment the deck end is held as a hinge: the three translations are fixed, the rotations about the longitudinal and vertical axes are restrained, and only the rotation about the lateral axis is left free. This reproduces the condition stated by Xu, Ko and Zhang [28], namely that the translational movement of the deck at the Ma Wan abutment is restricted in three directions while the deck is free to rotate about the lateral axis. The constraint is applied to all the nodes of the end cross-frame, including the two octagonal corners. At the Tsing Yi abutment the deck end is a sliding support: the lateral and vertical translations are fixed, and the longitudinal translation is free together with the rotations, which matches the condition given by the same authors for the Tsing Yi end, where only the vertical and lateral movements are restrained and the longitudinal movement and the rotation about the lateral axis are allowed. As the longitudinal translation of the deck is fixed at the Ma Wan abutment alone, the deck acts as a single girder anchored at the Ma Wan end and free to expand toward Tsing Yi.

4.10 Analysis procedure and modal results

A suspension bridge draws a large part of its stiffness from the tension in the cables rather than from the bending of its members. A modal analysis carried out on the unstressed geometry would therefore miss the geometric, or stress, stiffness that the dead load introduces, and would return frequencies with little physical meaning. For this reason the analysis is organised in two stages. A geometrically non-linear static analysis under self-weight first brings the structure to its tensioned dead-load equilibrium, and a linear perturbation modal analysis is then performed about that prestressed configuration. The same reasoning guided the models of Xu, Ko and Zhang [28] and of Duan et al. [17], who accounted for the cable tension by activating the stress-stiffening effect before extracting the modes.

4.10.1 Non-linear static analysis under self-weight

The static step is run with `ANTYPE,STATIC` and large-deflection effects enabled through `NLGEOM,ON`. Self-weight is applied as a gravity field with `ACEL,0,0,9.81`, acting on the mass of every element. Restart files are written at each substep (`RESCONTROL,LINER,ALL,1`) so that the modal step can reuse the converged state. Because the cable system carries almost no transverse stiffness until it picks up tension, the early part of the loading is strongly non-linear and prone to ill-conditioning. Convergence is obtained with automatic time stepping (`AUTOTS,ON`, with the substep range set by `NSUBST,200,1000,50`), up to two hundred equilibrium iterations per substep (`NEQIT,200`), a force convergence tolerance of one per cent on

the L_2 norm (CNVTOL,F), and energy stabilisation (STABILIZE,CONSTANT,ENERGY,0.001) to damp the spurious motions of the slack configuration.

The initial tension of the cables is not imposed as a nodal force; it is introduced by the thermal pre-strain described in subsection 4.8.2. The temperature decrease applied to the cable elements, $\Delta T = -227.5$ K, was chosen so that the resulting axial pre-stress, $\sigma = E \alpha |\Delta T| = 196 \text{ GPa} \times 1.2 \times 10^{-5} \times 227.5 \approx 535 \text{ MPa}$, produces a force of $\sigma A \approx 406 \text{ MN}$ over the main-cable area of 0.759 m^2 , equal to the design horizontal tension reported by Xu, Ko and Zhang [28]. The static step equilibrates this pre-stress against the self-weight, so that the tensioned dead-load configuration on which the modes are subsequently computed is consistent with the design state of the bridge.

4.10.2 Dead-load equilibrium checks

Before the modes are examined, the converged dead-load state is checked against the design quantities of the bridge. The mass of the model, broken down by component, is collected in Table 4.8. The total mass is $2.10 \times 10^5 \text{ t}$, which corresponds to a total weight of about 2065 MN , carried to the ground at the tower bases, the pier bases and the anchorages. As a first check on the static solution, the vertical support reactions returned by the analysis sum to this total weight, so the model is in global vertical equilibrium under its own self-weight. The two reinforced concrete towers and the five piers account for a little over half of the mass, the steel deck for about one third, and the cables and hangers for the remainder.

Component	Mass [t]	Weight [MN]
Towers and piers	113 650	1115
Main cables	27 590	271
Hangers	1 750	17
Deck	67 467	662
Total	210 458	2065

Table 4.8: Mass of the model by component and the corresponding weight under $g = 9.81 \text{ m s}^{-2}$.

The deck weight along the main span amounts to 307.8 kN m^{-1} . Xu and Xia [22] estimate the self-weight of the structural components of the deck at about 204 kN m^{-1} and the superimposed dead load of the non-structural elements at 88 kN m^{-1} , a combined dead load of about 292 kN m^{-1} ; the model exceeds this value by 5.4 per cent, a small surplus that follows from the rectangular sections of the deck.

The cable tension is recovered in the same way. The horizontal tension of the main cable, taken as the axial force at the main-span midspan where the cable is horizontal, is $401\,722 \text{ kN}$, that is 1.0 per cent below the design horizontal tension of $405\,838 \text{ kN}$ reported

for the main span by Duan et al. [17] and by Xu and Xia [22]. This close agreement confirms that the thermal pre-strain reproduces the design pre-tension and that the model rests in its intended dead-load configuration.

4.10.3 Modal analysis about the prestressed configuration

The modal step is set up as a linear perturbation that restarts from the end of the static analysis (ANTYPE,STATIC,RESTART,,PERTURB with PERTURB,MODAL). The element matrices are regenerated from the base state through SOLVE,ELFORM, so that the tangent stiffness used in the eigenvalue problem includes the stress stiffening produced by the cable tension and the change of geometry accumulated during the static step. The modes are extracted with the Block Lanczos method (MODOPT,LANB) and expanded together with their element results (MXPAND). Computing the modes about the prestressed state, rather than about the undeformed geometry, is what makes the extracted frequencies representative of the bridge under dead load.

4.10.4 Modal frequencies and comparison with measurements

Six principal global modes of the deck are identified: the first lateral, vertical and torsional modes, for which a clear measured counterpart is available. The remaining computed modes are higher-order global modes or local vibrations of the cables, the side spans and the towers, and are not retained for the comparison. The frequencies of the six principal modes are compared with the measured values in Table 4.9. The measured values and the mode labels are taken from Table 4.1 of Xu and Xia [22] and originate from the ambient vibration survey of Xu, Ko and Zhang [28]. The lowest mode is the first lateral mode of the deck, in agreement with the field observation that the fundamental frequency of the bridge corresponds to a lateral motion.

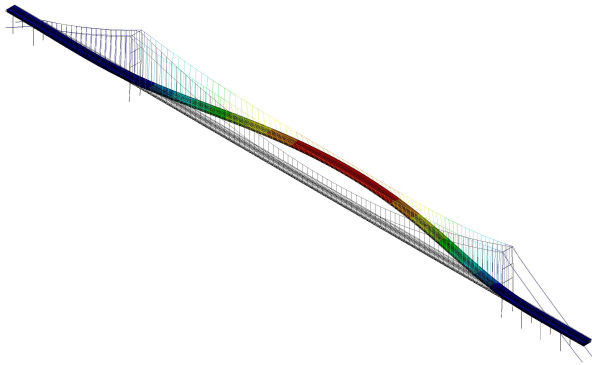
The computed frequencies stay close to the measured ones, with a mean absolute difference of 6.4 per cent. Five of the six modes fall within about ten per cent, and only the first vertical antisymmetric mode reaches 12.5 per cent. Most of the differences are positive, so the model is on the whole slightly stiffer or lighter than the bridge; the first lateral antisymmetric mode is the exception, lying 2.0 per cent below its measured value.

Mode	Mode shape	f_m (Hz)	f_{FE} (Hz)	Diff. (%)
L1	First lateral, symmetric	0.069	0.0741	+7.4
V1	First vertical, antisymmetric	0.113	0.1271	+12.5
V2	First vertical, symmetric	0.139	0.1427	+2.7
L2	First lateral, antisymmetric	0.164	0.1608	-2.0
V3	Second vertical, symmetric	0.184	0.2016	+9.6
T1	First torsional	0.267	0.2792	+4.6
Mean absolute difference				6.4

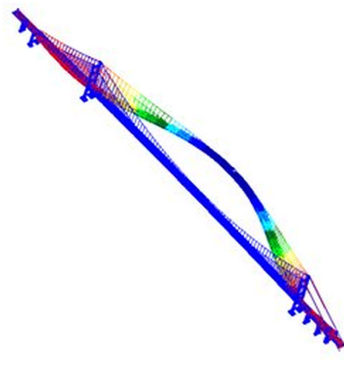
Table 4.9: Computed natural frequencies of the finite element model, f_{FE} , compared with the measured frequencies f_m

The tendency of the computed frequencies to exceed the measured ones is consistent with the modelling assumptions, the equivalent rectangular sections of the deck framework and the smeared orthotropic plates, fixed against the dead load and the measured frequencies, and the coupling-based bearings, which impose their kinematic restraint without flexibility, act in the same direction. These effects are most visible on the vertical modes, which depend on the deck bending stiffness and mass, and least on the lateral antisymmetric mode.

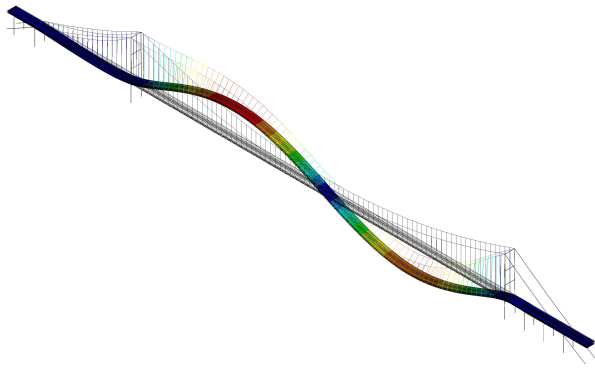
The computed mode shapes are compared in Figure 4.18 with the corresponding shapes reported by Duan et al. [17]. The sequence of the six modes and their deformed patterns reproduce the published results, which confirms that the model captures the global vibration modes in shape as well as in frequency.



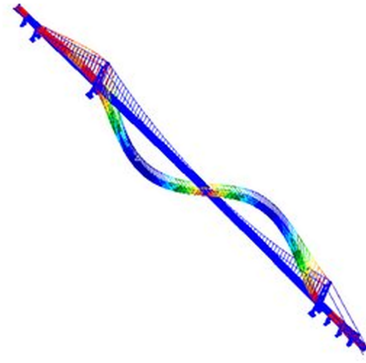
Present work, L1



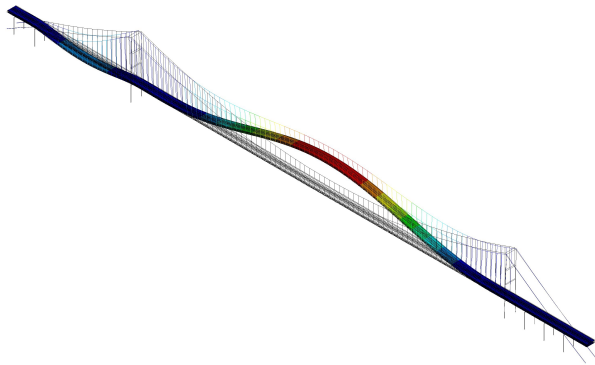
Duan et al. [17], L1



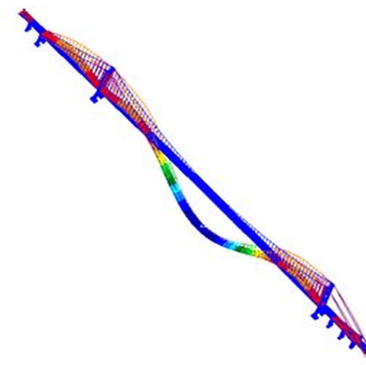
Present work, V1



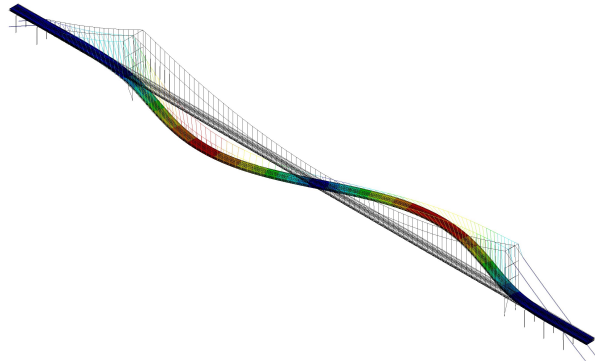
Duan et al. [17], V1



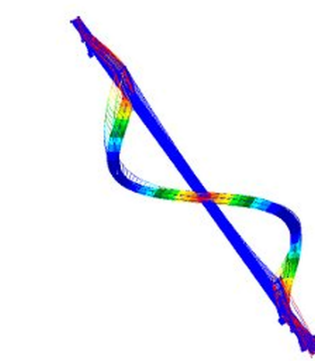
Present work, V2



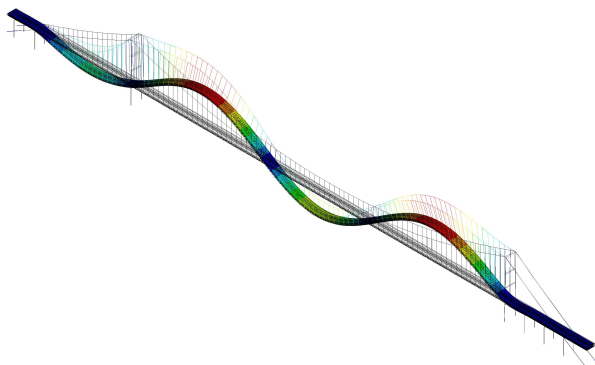
Duan et al. [17], V2



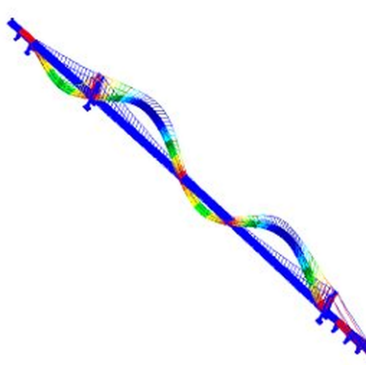
Present work, L2



Duan et al. [17], L2



Present work, V3



Duan et al. [17], V3

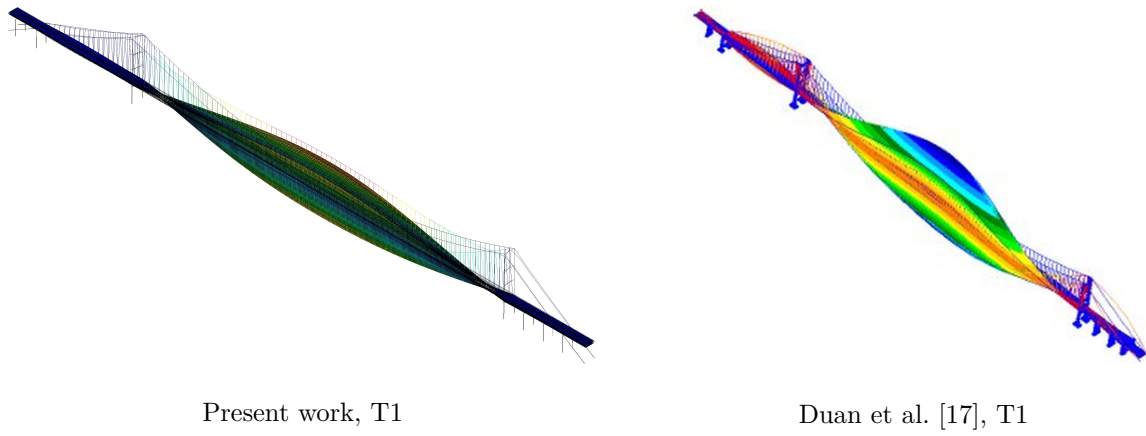


Figure 4.18: Comparison between the mode shapes obtained in the present work and the corresponding modes reported by Duan et al. [17].

Chapter 5

Thermal-induced and railway-induced effects on the Tsing Ma Bridge

5.1 Introduction

Long-span road-rail bridges with steel truss decks combine two demanding service conditions. The deck must transfer rail traffic loads through a deep cross-section, and at the same time it must accommodate the daily and seasonal redistribution of temperature that follows from atmospheric exposure and direct solar radiation. The two actions produce stress fields of different character.

For a truss deck the thermal problem is amplified by both the depth and the width of the section. In the Tsing Ma deck the top and bottom chords are separated vertically by 7.643 m, and the section is 41 m wide [31]. A small temperature difference between regions of the section produces differential thermal elongations. Restrained by the boundary conditions of the deck and by the suspension system, these elongations induce internal forces in the chord members and deformation of the deck cross-section. Whether the resulting effects are comparable with those of service traffic is the question that motivates this chapter.

The question has acquired practical relevance with the growing record of structural health monitoring on operating bridges. Zhang et al. [19] report a long-term temperature analysis of a road-rail steel truss cable-stayed bridge, in which the uniform effective temperature, the vertical temperature difference between the top and bottom chord members, and the transverse temperature difference between the chord members on the two sides of the deck are obtained by integrating the measured temperature field, with a deep-learning model used for prediction. For the Tsing Ma Bridge itself, Zhang et al. [32] report twenty-six years of monitoring and identify temperature as the primary driver of the mid-span deck displacement, alongside the vertical temperature gradient measured separately on the north and south sides of the deck.

For comparison, EN 1991-1-5 [20] prescribes the equivalent linear vertical temperature difference for steel decks of bridge Type 1 as 18°C for heating and 13°C for cooling, and addresses the transverse component only through a single recommended value of 5°C between the outer edges of the deck.

The present chapter applies the uniform, vertical, and transverse temperature components, together with two reference rail traffic load models, a code-based envelope and the measured service train, to the finite-element model of the Tsing Ma Bridge, and quantifies the resulting action effects at a mid-span reference cross-section. The Tsing Ma Bridge carries combined road and rail traffic on a steel truss deck, a deck typology shared with the bridge analysed by Zhang et al. [19].

5.1.1 Objective and scope

The objective of the analyses presented in this chapter is to quantify the structural response of the Tsing Ma Bridge to three thermal loading components and to assess that response against the effects of the rail traffic carried by the bridge. The uniform effective temperature is derived from the long-term temperature monitoring of the Tsing Ma Bridge reported by Xu et al. [33], and the vertical temperature difference from the monitoring reported by Zhang et al. [32]. No transverse temperature difference is reported in the monitoring studies available for the bridge; as no field record of this component exists for the Tsing Ma Bridge, the transverse difference is adopted from the road-rail steel truss cable-stayed bridge analysed by Zhang et al. [19] and is used as a representative loading scenario rather than a site-specific measurement. The thermal response is assessed against two reference rail traffic load models, a code-based envelope and the measured service train. The comparison serves as a structural assessment of the relative importance of thermally induced and railway-induced effects for a long-span road-rail truss deck, and as a check of the measured vertical and transverse components against the values prescribed by EN 1991-1-5 [20].

The thermal analysis is conducted in steady state, that is, the temperature field is applied as a final equilibrium condition rather than as a transient process. The mechanical analysis is static and geometrically non-linear, consistent with the suspension bridge behaviour.

5.1.2 Approach

The three thermal components and the two rail traffic models are applied to the finite-element model as independent load cases, each acting on the dead-load equilibrium of the bridge. The vertical temperature difference is taken from the long-term monitoring of the Tsing Ma Bridge reported by Zhang et al. [32], which records the gradient separately on the north and the south sides of the deck. The transverse temperature difference, for which no record exists for the bridge, is taken from the road-rail steel truss cable-stayed

bridge analysed by Zhang et al. [19] and is used as a representative loading scenario. The uniform effective temperature is established from the climatic record of the site and from the monitored relation between air, deck, and cable temperatures, as set out in section 5.2. The uniform component is applied in the summer and winter senses that bound the annual cycle. The vertical difference is applied in its governing summer sense, with the top of the deck warmer than the bottom; the monitoring record of Zhang et al. [32] shows the gradient keeping this sense through the year, its summer maximum being the governing value. The transverse difference is applied in its governing sense, with the south side warmer than the north. The values prescribed by EN 1991-1-5 [20] for the vertical and the transverse difference are introduced as additional load cases. For the transverse component this allows the monitoring-derived and the code-prescribed actions to be compared at the level of the resulting action effects; for the vertical the two values represent different quantities and are not compared directly.

The rail traffic is represented by two models, a code-based envelope and the measured service train, set out in section 5.5. Each model is applied in two transverse configurations, both tracks loaded and a single eccentric track, and at two longitudinal positions, centred on the mid-span and adjacent to the Tsing Yi tower.

Each load case is computed as an increment on the dead-load equilibrium. The self-weight and the cable pretension are solved first, in a load step that converges to the dead-load configuration of the bridge. The action is then introduced in a second load step that continues from this configuration, and the action effect is obtained as the difference between the loaded and the dead-load states. Both load steps are solved with geometric non-linearity, consistent with the behaviour of the suspension system. The action effects are recovered at a reference cross-section at the mid-span of the main span, where six stress components are extracted, and the axial force in the main cable is examined along its length, with the largest increment and its position recorded. Each action is applied on its own, and no load combinations are formed. The purpose here is to weigh the effect of each action individually, on the common dead-load footing, so that the thermal and the railway contributions can be compared term by term. The load cases considered are collected in Table 5.1. Each component is then examined in a dedicated section.

Action	Load case	Configuration
Uniform temperature	Summer	$\Delta T_N = +25^\circ\text{C}$ deck, $+19^\circ\text{C}$ cables
	Winter	$\Delta T_N = -16^\circ\text{C}$ deck, -22°C cables
Vertical difference	Measured	17°C (N) / 14°C (S), heating
	Eurocode	18°C , uniform across width
Transverse difference	Measured	5.7°C (upper) / 14.91°C (lower)
	Eurocode	5°C between deck edges
Railway – HSLM-A2	Both tracks	mid-span and Tsing Yi tower
	Single track	eccentricity $e = 2\text{--}5\text{ m}$
Railway – measured train	Both tracks	mid-span and Tsing Yi tower
	Single track	eccentricity $e = 2\text{--}5\text{ m}$

Table 5.1: Load cases applied as independent increments on the dead-load configuration.

5.2 Uniform effective temperature

The uniform effective temperature is the temperature, constant over the cross-section, that governs the overall expansion or contraction of the structure (EN 1991-1-5 [20], §1.5.7). It is applied in the two senses that bound the annual cycle. In the summer sense the deck reaches its maximum effective temperature of 46°C , the long-term monitoring of the bridge showing the maximum effective deck temperature to run about 8°C above the maximum ambient air temperature [33]. In the winter sense the minimum effective temperature is almost the same as the minimum ambient air temperature [33], taken as 5°C .

The mechanical effect of a uniform temperature change does not depend on the absolute temperature reached by the structure, but on its departure from the temperature at which the structure became restrained. EN 1991-1-5 expresses the expansion and contraction ranges relative to an initial temperature T_0 through expressions (6.1) and (6.2) [20]. The uniform component applied to the model is therefore

$$\Delta T_N = T_e - T_0, \quad (5.1)$$

positive in the summer sense and negative in the winter sense.

No closure or setting temperature is documented for the bridge, which was opened to traffic in 1997 [32]. For the case in which T_0 cannot be predicted, EN 1991-1-5, Annex A.1(3), directs that the average temperature during the construction period be taken [20]. A reference temperature of $T_0 = 21^\circ\text{C}$ is adopted, the mid-range of the ambient air temperature recorded for the bridge, which varies between about 5 and 38°C [33]. The uniform component is then $\Delta T_N = +25^\circ\text{C}$ in the summer sense and -16°C in the winter sense [33].

The main and side cables run cooler than the deck through the year. The cable temperature, monitored by thermocouples embedded in the main cables, stays below that of the deck by 6°C through the year [33], so that the uniform change carried by the cables is $\Delta T_N - 6^\circ\text{C}$, that is $+19^\circ\text{C}$ in the summer sense and -22°C in the winter sense. The difference does not reverse between the two senses, the cables remaining the cooler element in both. The resulting load cases are summarised in Table 5.2.

Sense	T_e (deck) ($^\circ\text{C}$)	ΔT_N (deck, towers, piers) ($^\circ\text{C}$)	cable difference ($^\circ\text{C}$)	cable change ($^\circ\text{C}$)
Summer	46	+25	-6	+19
Winter	5	-16	-6	-22

Table 5.2: Uniform effective temperature load cases, with $T_0 = 21^\circ\text{C}$.

The uniform component ΔT_N is applied to the steel truss deck, the concrete towers and piers. The main and side cables carry the same component reduced by the cable-deck difference, $\Delta T_N - 6^\circ\text{C}$, in addition to their pretension, which is defined at the stress-free reference state.

5.3 Vertical temperature difference

The vertical temperature difference is the temperature difference across the depth of the truss girder, between the top and the bottom chord members. EN 1991-1-5, §6.1.4.1 [20] treats it as a linear temperature difference over the depth, the heating sense, with the top of the deck warmer than the bottom, taken as positive.

5.3.1 Measured values

The vertical temperature difference of the Tsing Ma deck is obtained from the long-term monitoring reported by Zhang et al. [32], which records the gradient separately on the north and the south sides of the deck. The monthly maximum vertical difference reaches about 17°C in summer and about 8°C in winter, the top of the deck remaining warmer than the bottom throughout the year [32]; the summer maximum governs and is adopted here. The two sides differ in summer. The north side gradient exceeds the south side by 3°C , the bridge lying south of the Tropic of Cancer so that its north face receives direct solar radiation in summer [32]. The vertical difference is therefore applied in the heating sense with a magnitude of 17°C on the north side and 14°C on the south side.

5.3.2 Application to the finite-element model

The vertical difference is applied to the deck as a linear variation along the depth coordinate z , between the bottom-chord level $z_b = 54.0$ m and the top-chord level $z_t = 61.643$ m, with the deck mid-height at $z_m = (z_b + z_t)/2 = 57.82$ m. The vertical difference varies linearly across the width, between the outer edges of the upper deck at $y_N = 5.15$ m on the north side and $y_S = 35.85$ m on the south side. The value at a transverse position is

$$G(y) = \text{VTD}_N + (\text{VTD}_S - \text{VTD}_N) \frac{y - y_N}{y_S - y_N}, \quad (5.2)$$

and the temperature field is

$$T_{\text{VTD}}(z, y) = G(y) \frac{z - z_m}{z_t - z_b}, \quad (5.3)$$

with $\text{VTD}_N = 17^\circ\text{C}$ and $\text{VTD}_S = 14^\circ\text{C}$. The field ranges from $-G(y)/2$ at the bottom-chord level to $+G(y)/2$ at the top-chord level, so that its average over the depth is zero and no uniform component is introduced.

The field is implemented as a two-dimensional lookup table in (z, y) , evaluated at each deck node. The chord members at $y = 7.5$ m and $y = 33.5$ m and the inner struts at $y = 13.8$ m and $y = 27.2$ m receive the values interpolated from (5.2). The truss diagonals and verticals that connect the two deck levels are not assigned a temperature directly; each is a beam with end nodes at the two deck levels, and the temperature along its axis is recovered as the linear interpolation between the end-node values, consistent with the linear field of (5.3). The resulting field applied to the cross-section is shown in Figure 5.1. Because the gradient is stronger on the north side than on the south, the response is not symmetric about the deck axis, the north chords carrying a slightly larger effect than the south chords. This asymmetry follows from the measured field and distinguishes it from a gradient applied uniformly across the width. The main and side cables and the towers are not affected by the vertical difference, which is a deck-level effect; the cables retain their pretension.

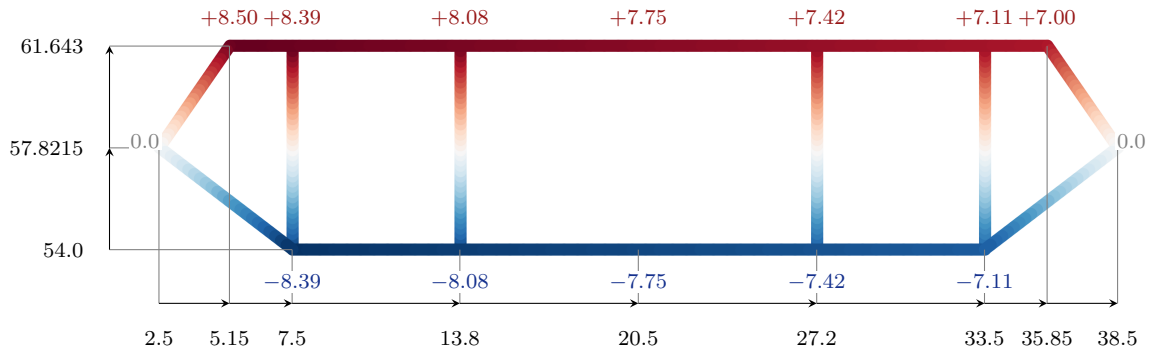


Figure 5.1: Vertical temperature difference applied at the cross-section, in $^\circ\text{C}$.

5.3.3 Eurocode reference value

EN 1991-1-5, Table 6.1 [20] prescribes the equivalent linear vertical temperature difference for steel decks of bridge Type 1, the heating value of 18°C being the larger of the prescribed values and the governing case for the upper-deck chords. The Eurocode value does not distinguish between the two sides of the deck and is applied uniformly across the width, $\text{VTD}_\text{N} = \text{VTD}_\text{S} = 18^\circ\text{C}$, through the same linear variation along the depth as the measured field.

5.4 Transverse temperature difference

The transverse temperature difference is the difference in temperature across the width of the deck, between its northern and southern sides. It develops when one side is more exposed to solar radiation than the other, which for an east–west oriented bridge occurs through the day as the sun moves across the bridge axis. Unlike the vertical difference, the transverse component is not recorded by the Tsing Ma monitoring system: the deck instrumentation resolves the temperature distribution through the depth of the cross-section but not across its width. In the absence of field records for this bridge, the transverse difference is therefore taken from a comparable structure rather than measured directly, and the values below should be read as borrowed estimates rather than site data. The reference structure is a road-rail steel truss cable-stayed bridge monitored by Zhang et al. [19], which shares the present deck typology, a double-deck steel truss carrying a highway above and a railway below, so that its across-width thermal behaviour is representative of the Tsing Ma deck.

5.4.1 Reference values

The reference bridge is instrumented with six chord-mounted temperature sensors, Ch1 to Ch3 on the upper deck and Ch4 to Ch6 on the lower deck, numbered as in Figure 5.2. Zhang et al. define the transverse difference as the difference in effective temperature between the chords on the two sides of a deck,

$$T_{\text{ttd}} = T_{\text{CH4}} - T_{\text{CH6}}, \quad (5.4)$$

the largest value occurring between the lower chords Ch4 and Ch6, on the two sides of the railway deck, which are alternately exposed to direct solar radiation as the sun crosses the bridge axis, while the upper deck is shaded by its asphalt surfacing [19]. Two quantities are retained. The upper-deck difference, between Ch1 and Ch3, has a recorded extreme of 5.7°C ; this small value is consistent with the near-uniform temperature of the paved upper deck. The lower-deck difference, between Ch4 and Ch6, which governs, has a recorded

extreme of 14.91 °C [19]. The lower deck dominates because its chords are directly exposed, whereas the road surfacing attenuates the difference on the upper deck. These two values, 5.7 °C on the upper deck and 14.91 °C on the lower deck, are the differences applied to the model.

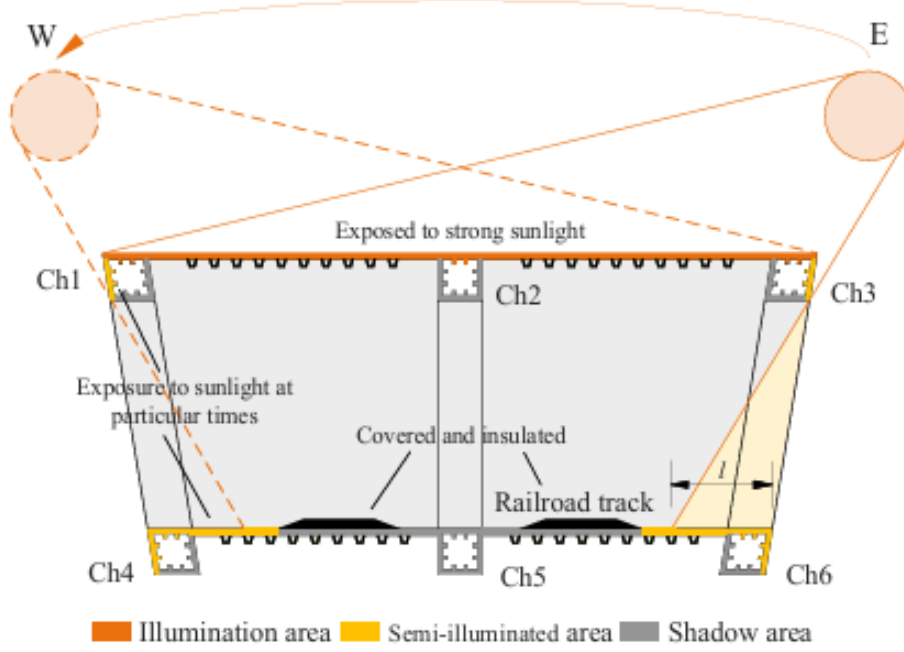


Figure 5.2: Cross-section of the road-rail steel truss girder of the reference bridge, with the chord members numbered. From [19].

5.4.2 Application to the finite-element model

The difference is applied as an antisymmetric linear distribution across the width, so that the mean temperature over the cross-section is zero and the load is a pure gradient with no associated uniform component. Each deck level carries its own ramp. For a deck level with northern anchor at y_N and southern anchor at y_S carrying a north–south difference ΔT_t , the applied temperature varies as

$$T_t(y) = \frac{\Delta T_t}{2} \frac{2y - y_S - y_N}{y_S - y_N}, \quad (5.5)$$

which equals $-\Delta T_t/2$ at y_N , $+\Delta T_t/2$ at y_S , and vanishes at mid-width. The upper-deck ramp is anchored at the two edges of the structural upper deck, $y_N = 5.15$ m and $y_S = 35.85$ m, with $\Delta T_t = 5.7$ °C. The lower-deck ramp is anchored at the lower chords, $y_N = 7.5$ m and $y_S = 33.5$ m, with $\Delta T_t = 14.91$ °C. Both ramps share the same centreline at $y = 20.5$ m, where the applied temperature is zero.

The two profiles are implemented as one-dimensional lookup tables indexed by the y coordinate of each node. The table of the upper-deck ramp is applied to nodes in the depth range $60 \leq z \leq 63$ m, which selects the top deck, and the table of the lower-deck

ramp to nodes in the range $52 \leq z \leq 56$ m, which selects the bottom deck; the linear interpolation between the two endpoints of each ramp is provided automatically by the analysis software. The truss diagonals and verticals receive their temperature through linear interpolation along the axis between the end-node values, in the same manner as for the vertical component.

The suspension corners are treated differently from the vertical case. There, the field varies continuously along the depth and reaches the corner nodes at mid-height directly. Here the difference is applied separately to each deck level, so the corner nodes, which lie between the two levels and outside the depth range of either ramp, are not reached by the ramps and are assigned the mean of the upper-deck and lower-deck temperatures at the adjacent chord. The resulting field applied to the cross-section is shown in Figure 5.3. The main and side cables, the hangers, and the towers are not affected by the transverse difference, consistent with its deck-level character; the cable pretension is unchanged across all thermal cases.

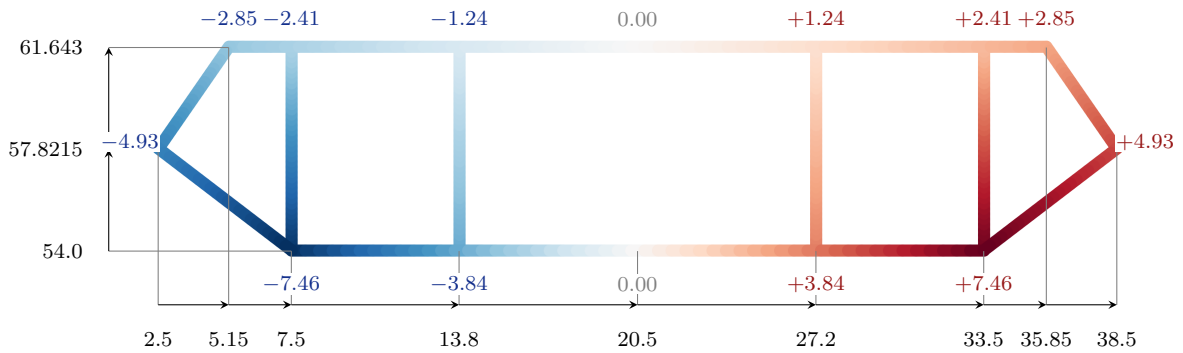


Figure 5.3: Transverse temperature difference applied at the cross-section, in °C.

5.4.3 Eurocode reference value

Eurocode EN 1991-1-5 treats the transverse difference as a special case. Its clause on horizontal components states that the temperature difference component need only be considered in the vertical direction in general, but that a horizontal component should be taken into account where the orientation or configuration of the bridge leaves one side more exposed to sunlight than the other [20]. In the absence of other information, the recommended value is a linear difference of 5 °C between the outer edges of the deck, independent of the width of the bridge [20]. This recommended difference is applied to the model as a uniform 5 °C across both decks, following the same antisymmetric distribution of (5.5). The borrowed lower-deck extreme of 14.91 °C is close to three times the Eurocode value, which indicates that the recommended value can substantially underestimate the transverse difference of a directly exposed truss deck of this type, in line with the conclusion drawn by Zhang et al. for the same class of structure [19]. The structural consequences of both cases are reported in section 5.6.

5.5 Rail traffic load

The rail traffic action provides the reference service load against which the thermal action effects of section 5.2 to section 5.4 are assessed. The Tsing Ma deck carries the twin tracks of the airport railway at the lower chord level of the truss girder. Two complementary rail load models are adopted for the comparison: a code-based envelope, the High Speed Load Model HSLM of EN 1991-2 [34], and the measured service train of the line, reconstructed from the structural health monitoring data by Lu et al. [29] and Duan et al. [17]. The first provides a conservative, normative reference; the second represents the action actually exerted by the traffic that operates on the bridge. This section sets out the two load models, their common application to the finite-element model, and the dynamic factor applied to the code action. The resulting action effects are reported, alongside the thermal ones, in section 5.6.

5.5.1 Code-based load model: HSLM-A2

The comparison of the railway action against the measured thermal components requires a finite convoy of realistic length, representative of the passenger traffic carried by the bridge. EN 1991-2 provides such a convoy in the High Speed Load Model HSLM, whose universal trains are defined in §6.4.6.1.1, Figure 6.12 and Table 6.3, with the maximum train length limited to 400 m in Annex E [34]. For continuous and complex structures, and for spans of 7 m or more, EN 1991-2 (Table 6.4) prescribes HSLM-A, a family of ten universal trains, A1 to A10, each composed of two power cars, two end coaches, and N intermediate coaches [34]. Each universal train is characterised by the number of intermediate coaches N , the coach length D , the bogie axle spacing d , and the point force per axle P ; as shown in Figure 5.4, the two power cars contribute four point forces P each and the two end coaches three each, while every intermediate coach contributes two, so that the number of point forces is $M = 14 + 2N$ and the load per track is $W_{\text{tr}} = (14 + 2N) P$.

Because the comparison is conducted as a static analysis, and not as the speed-dependent resonance analysis for which HSLM is formulated, the governing universal train is the heaviest of the family rather than the one selected through the dynamic procedure of Annex E. Among the ten trains the load per track W_{tr} is maximised by train A2, with $N = 17$, $D = 19$ m, $d = 3.5$ m and $P = 200$ kN, so that $M = 48$, $W_{\text{tr}} = 9600$ kN, and the convoy length is $L_{\text{tr}} = (N + 2) D = 361$ m, within the 400 m limit. The parameters adopted are collected in Table 5.3.

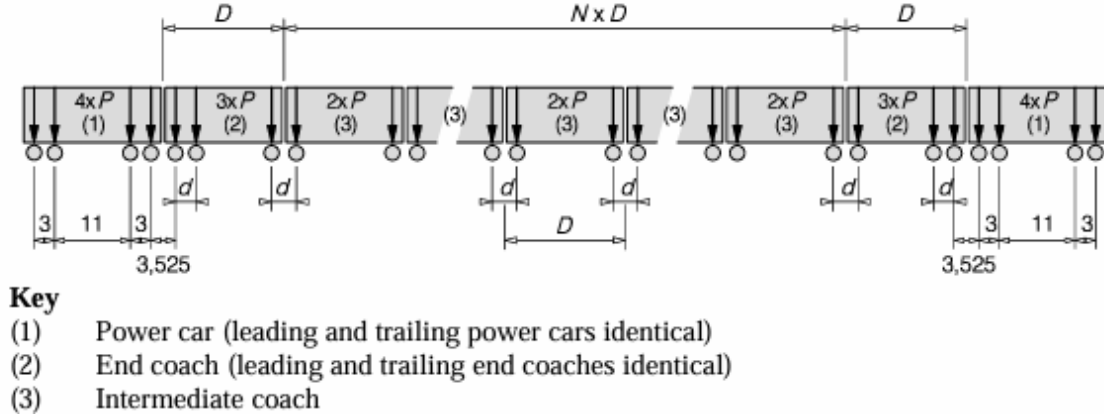


Figure 5.4: HSLM-A universal train (EN 1991-2, Figure 6.12) [34].

Parameter	Value
Intermediate coaches N	17
Coach length D	19 m
Bogie axle spacing d	3.5 m
Point force P	200 kN
Number of point forces $M = 14 + 2N$	48
Load per track W_{tr}	9600 kN
Convoy length L_{tr}	361 m

Table 5.3: Parameters of the governing universal train HSLM-A2 (EN 1991-2, Table 6.3) [34].

HSLM is defined for lines carrying passenger traffic at speeds exceeding 200 km/h [34], whereas the airport railway over the Tsing Ma Bridge operates at about 108 km/h [29]. The model is therefore adopted here not as the prescribed action for the line, but as a standardised passenger convoy suitable for the intended comparison, and as a conservative envelope.

5.5.2 Measured service train

A representation of the actual service train is available from the studies that have monitored and modelled the Tsing Ma Bridge against its measured response. The airport railway has carried predominantly eight-car trains since the end of 2005, each car running on two bogies [22]. Lu et al. [29] derive a standard train from the records of the health-monitoring system: an eight-car convoy of 338 t gross weight, travelling at 30 m/s (108 km/h). Duan et al. [17] give the geometry of the same rolling stock, shown in Figure 5.5: each car runs on two bogies 14.350 m apart, and the gap between the rear bogie of one car and the front bogie of the next is 6.125 m, with eight-car and seven-car trains reported as the typical formations.

The 338 t gross weight is a representative value rather than a fixed one: the loaded weight varies with occupancy and the formation varies between seven and eight cars [17, 29]. This convoy is adopted as the second rail load model, complementary to the HSLM-A2 envelope, representing the action actually exerted by the traffic that operates on the bridge. It is modelled with two concentrated loads per car, at the bogie positions given by Duan et al. [17], sixteen loads in total for the eight-car train. The gross weight is distributed uniformly over the sixteen bogie loads, 207.2 kN each, so that the total of 3316 kN per track is preserved exactly. The studies cited apply the train as a moving load through a transient analysis [17, 29]; in the present work it is applied statically.

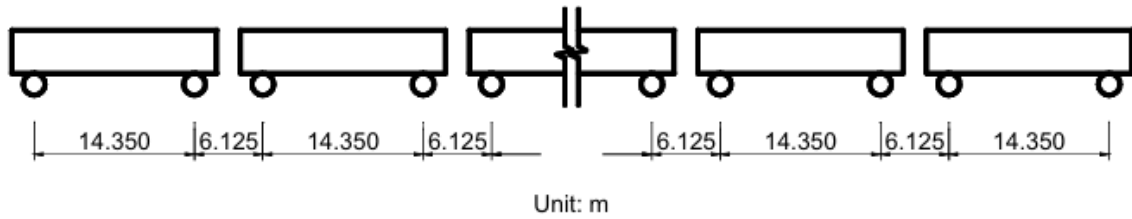


Figure 5.5: Longitudinal configuration of the measured service train. From [17].

5.5.3 Application to the finite-element model

On the Tsing Ma Bridge the railway tracks are carried on the lower chords of the cross-frames, between the inner struts, from where the load reaches the suspension system [17]. The track is of the direct-fastened type, with steel track plates resting on rail waybeams over the lower-chord cross-frames, without ballast [17, 22]. Both load models are applied consistently with this path, as vertical nodal forces at the lower-chord nodes located at the inner struts, at the transverse positions $y = 13.8$ m and $y = 27.2$ m (Figure 5.6).

The longitudinal representation differs between the two models. For the HSLM-A2 convoy the 48 axle forces are represented by an equivalent uniform line load of intensity $w = W_{\text{tr}}/L_{\text{tr}} = 26.6$ kN/m per track over the 361 m footprint, applied as nodal forces at the cross-frames of the main span, spaced at 4.5 m [17, 22], with the tributary length of the nodes at the edges of the footprint reduced accordingly. For the measured train the sixteen bogie loads are applied at their actual longitudinal positions; each load, falling in general between two cross-frames, is distributed to the two adjacent nodes in inverse proportion to its distance from them, which preserves both the value and the position of the load.

Each model is applied at two longitudinal positions. In the first the convoy is centred on the mid-span of the main span, in the second the convoy is shifted towards the Tsing Yi tower until the leading edge of its footprint reaches the tower at $x = 1377$ m.

Two configurations are considered for each model: both tracks loaded, which gives the most severe symmetric response, and a single track loaded, which is eccentric with respect

to the deck axis. With both tracks occupied the total weight is 19200 kN for the HSLM-A2 convoy and 6632 kN for the measured train.

The two tracks occupy the central strip of the lower deck, between the inner struts. Their exact transverse positions are not reported in the sources consulted for the present work, and are therefore treated through a sensitivity parameter rather than a fixed value. With both tracks loaded the resultant is symmetric about the deck axis and divides equally between the two inner struts, independently of the track spacing, so that no net torsion arises. For a single loaded track at an offset e from the axis, the share carried by each inner strut follows from the static distribution over the 13.4 m between the struts,

$$c_{13} = \frac{6.7 + e}{13.4}, \quad c_{27} = \frac{6.7 - e}{13.4}, \quad (5.6)$$

where the indices denote the struts at $y = 13.8$ and $y = 27.2$ m. The offset e is varied between 2 and 5 m.

5.5.4 Dynamic factor

The dynamic factor is applied to the HSLM-A2 action only; the measured service train is taken without amplification, as set out in subsection 5.5.2. EN 1991-2 determines the dynamic effects of HSLM through a dynamic analysis over a range of speeds, rather than through the static factor Φ used for Load Model 71 [34]. For the present static comparison the dynamic magnification of the code action is introduced instead through the dynamic factor for real trains of Annex C, applied as an equivalent static amplification,

$$1 + \varphi = 1 + \varphi' + \varphi'', \quad (5.7)$$

with

$$\varphi' = \frac{K}{1 - K + K^4}, \quad K = \frac{v}{2L_\Phi n_0} \quad (K < 0.76), \quad (5.8)$$

where v is the maximum permitted vehicle speed, n_0 the first natural bending frequency of the bridge under permanent load, and L_Φ the determinant length [34]. The second term φ'' , which accounts for track irregularities, contains exponential functions of L_Φ that vanish for a span of this length, so that $\varphi'' = 0$.

The first natural vertical bending frequency of the Tsing Ma deck is $n_0 \approx 0.113$ Hz [28]. The determinant length for the global response of the main girder is taken as the main span, $L_\Phi = 1377$ m. The maximum permitted vehicle speed is taken as 130 km/h, that is, 1.2 times the operational speed of about 108 km/h reported for the line [29]. With these values $K = 0.116$ and $\varphi' = 0.131$, which gives

$$1 + \varphi \approx 1.13. \quad (5.9)$$

The expression of (5.8) is defined within the frequency limits of EN 1991-2, Figure 6.10, and for speeds up to 200 km/h; a fundamental bending frequency of 0.113 Hz on a span of 1377 m falls below the lower frequency limit of that figure, so (5.8) is applied here as a conservative extrapolation [34]. The dynamic magnification of the global response is in any case small: the loading frequency associated with the passage of the convoy at line speed, of the order of 2 Hz, lies far above the natural frequencies of the global modes of the bridge, which are below 0.3 Hz [28], so resonance of the global response cannot develop and the true dynamic amplification tends to unity. The value of 1.13 is therefore retained as a conservative upper bound on the global dynamic effect.

5.6 Results

The action effects produced by the thermal components of section 5.2 to section 5.4 and by the two railway models of section 5.5 are now compared. The comparison is drawn through three response quantities: the longitudinal stresses at the mid-span reference cross-section, the axial force in the main cable, and the vertical displacement and slope of the deck. It addresses the question raised in chapter 1, whether the thermal action effects on the truss deck are of the same order as those of the rail traffic carried by the bridge.

5.6.1 Extraction of the action effects

The reference cross-section is taken at the mid-span of the main span, at $x = 688.5$ m. Six longitudinal stresses are recovered there for each load case, as the increment between the loaded state and the dead-load configuration, and are indicated in Figure 5.6.

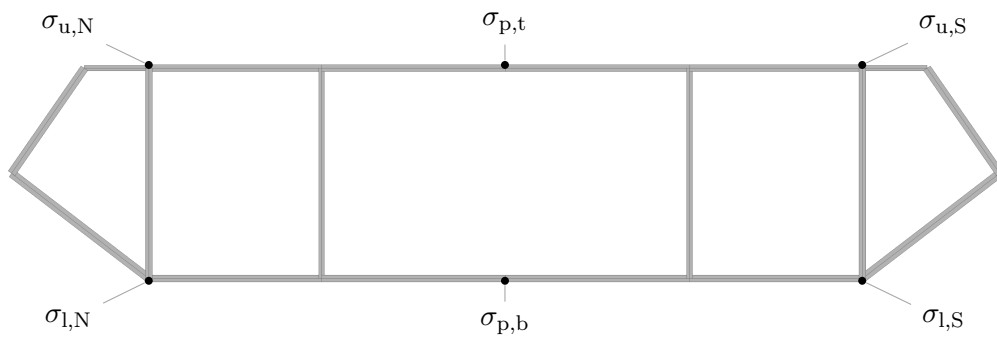


Figure 5.6: Cross-section of the Tsing Ma deck at mid-span with the six extracted longitudinal stresses.

Four of the six are axial stresses in the outer chord members, the upper and the lower chords of the north and of the south truss. The chords are modelled with two-node beam elements (BEAM188); the axial force N_c at the reference section is read from the first

section force resultant of the element and converted to a stress through

$$\sigma_c = \frac{N_c}{A_c}, \quad (5.10)$$

with $A_c = 0.175 \text{ m}^2$ for the upper chords and $A_c = 0.135 \text{ m}^2$ for the lower chords. The remaining two are the longitudinal membrane stresses in the top and in the bottom deck plate at mid-width, modelled with four-node shell elements (SHELL181); for each plate the longitudinal normal stress is read on the upper and on the lower surface and the membrane value is taken as their mean,

$$\sigma_m = \frac{1}{2} \left(\sigma_x^{\text{top}} + \sigma_x^{\text{bot}} \right), \quad (5.11)$$

denoted $\sigma_{p,t}$ for the top plate and $\sigma_{p,b}$ for the bottom plate. A positive value denotes tension.

The axial force in the main cable is examined over its whole length, the largest increment over the dead-load state and its position being recorded for each load case. The vertical displacement of the deck is read along the centreline of the lower deck, and the longitudinal slope is obtained as the change of vertical displacement between adjacent cross-frames.

5.6.2 Action effects under the individual load cases

The stress increments are collected in Table 5.4. The railway load with both tracks occupied, in either model, and the uniform component produce a response that is symmetric about the deck axis: the upper chord and the top plate in compression and the lower chord and the bottom plate in tension, with the signs reversed for the winter uniform sense. The measured vertical gradient produces the same sign pattern but is not symmetric about the deck axis, the north side carrying a larger effect than the south, by about 23 per cent at the upper chord and 15 per cent at the lower, which follows the stronger gradient on the north side adopted in section 5.3. The transverse difference is the only component that produces an antisymmetric response, the two sides of the deck carrying stresses of equal value and opposite sign; this is the stress-level signature of the transverse gradient. The single-track railway cases are reported for an eccentricity $e = 5 \text{ m}$; across the range $e = 2$ to 5 m of subsection 5.5.3 the mid-span stresses change only marginally.

Load case	$\sigma_{u,N}$	$\sigma_{l,N}$	$\sigma_{u,S}$	$\sigma_{l,S}$	$\sigma_{p,t}$	$\sigma_{p,b}$
HSLM-A2, both tracks	-19.61	28.81	-19.61	28.81	-17.32	26.62
HSLM-A2, single track	-9.76	14.98	-9.88	14.13	-8.66	13.63
Measured train, both	-13.17	19.58	-13.17	19.58	-11.56	18.01
Measured train, single	-6.96	10.71	-5.96	8.41	-5.80	9.16
Uniform, summer	-3.79	2.46	-3.79	2.46	-3.38	2.18
Uniform, winter	3.94	-3.91	3.94	-3.91	3.53	-3.51
Vertical, measured	-16.88	24.23	-13.75	21.10	-14.17	20.82
Vertical, Eurocode	-17.79	26.32	-17.79	26.32	-16.04	23.75
Transverse, measured	5.95	18.35	-5.95	-18.34	1.38	4.28
Transverse, Eurocode	5.21	6.15	-5.22	-6.15	1.21	1.43

Table 5.4: Stress increments at the mid-span reference cross-section (Figure 5.6), in MPa.

5.6.3 Measured against the Eurocode transverse difference

The transverse component admits a comparison between the measured field and the Eurocode recommendation. The measured difference produces a lower-chord stress of 18.35 MPa, against 6.15 MPa for the recommended value of 5 °C, the two sides carrying stresses of opposite sign in both cases. The measured field therefore produces a lower-chord stress about three times that of the code value, a ratio of 2.98 that matches the ratio of the temperature differences, $14.91/5 = 2.98$. The single recommended value of EN 1991-1-5 [20] understates the action effect of the measured transverse field by a factor of three.

The vertical component is not compared in the same way. The Eurocode value of 18 °C is an equivalent linear gradient, defined to reproduce the effect of the real, non-linear temperature distribution through the depth, whereas the value adopted in section 5.3 is the difference between the chord temperatures; the two represent different quantities and are not directly comparable at the level of the action effect. The Eurocode gradient is retained in Table 5.4 as a reference load case, but the comparison between the measured and the code-prescribed thermal actions is drawn only for the transverse component.

5.6.4 Thermal action effects relative to the railway load

To express the thermal action effects relative to the rail traffic, a ratio is formed at each chord and plate point as the thermal stress divided by the railway stress at that point, and the largest value over the four chord points and over the two plate points is retained for each thermal case. The ratio is computed against both the measured service train and the HSLM-A2 envelope, with both tracks occupied. The worst-point ratios are collected in Table 5.5.

Thermal load case	Chord		Plate	
	Measured train	HSLM-A2	Measured train	HSLM-A2
Uniform, summer	0.29	0.19	0.29	0.20
Uniform, winter	0.30	0.20	0.31	0.20
Vertical, measured	1.28	0.86	1.23	0.82
Transverse, measured	0.94	0.64	0.24	0.16

Table 5.5: Maximum ratio of the measured thermal stress to the railway stress with both tracks occupied at the mid-span reference cross-section, at the chords and at the deck plates.

Against the measured service train the vertical gradient produces the largest effect, reaching 1.28 of the train value at the upper north chord, so that the measured gradient values exceed the action effect of a full service train at the chords. The values of the transverse gradient reach 0.94 at the lower north chord, approaching equality with the train, while the uniform component stays close to 0.30. Against the HSLM-A2 envelope, the heavier action, every ratio is smaller, the vertical gradient reaching 0.86 and the transverse 0.64, while the relative standing of the components is unchanged.

The chord and the plate ratios together identify how each component loads the section. The transverse difference bends the deck across its width, which drives the outer chords while leaving the central plating close to the neutral axis of that bending, so its plate ratio of 0.24 falls far below its chord ratio of 0.94. The uniform and the vertical components bend the deck about a horizontal transverse axis and load the chords and the plating in comparable proportion, the vertical gradient reaching 1.23 at the plates against 1.28 at the chords.

The axial force in the main cable is governed by a load at the mid-span and is collected in Table 5.6. With the convoy centred on the main span the measured train raises the main cable force by 6225 kN and the side cable force by 6832 kN, against 19841 and 23206 kN for the HSLM-A2 envelope; placed against the Tsing Yi tower the same trains raise the cable force by less, 1935 kN for the measured train at the main cable: near the tower the deck is supported vertically on the tower cross beam, so a load there discharges largely into the tower rather than into the cable through the hangers. The vertical and the transverse gradients leave the cable force essentially unchanged, below 200 kN, the cable being affected only by the uniform component, which cools and shortens the cables in winter and raises the main cable force by 6260 kN. The values listed are the largest increment over the dead-load state and its position; the total cable force is largest at the tower saddles in every case, and the side cable, carrying the higher force throughout, shows the larger increment. The thermal and the railway maximum fall at different points, the winter increment peaking near the Tsing Yi tower and the railway increment in the Ma Wan half of the main span, which follows from the Ma Wan side span being suspended

from the cable while the Tsing Yi side span is carried on piers. Compared peak to peak, the winter uniform component gives ratios of 1.01 at the main cable and 1.11 at the side cable against the measured service train, equalling or exceeding it, and about 0.32 against the HSLM-A2 envelope.

Load case	Main cable		Side cable	
	ΔN (kN)	location	ΔN (kN)	location
Uniform, winter	+6260	TY tower, $x \approx 1368$	+7588	TY side span, $x \approx 1422$
Uniform, summer	-5365	TY tower, $x \approx 1368$	-6502	TY side span, $x \approx 1422$
Measured train, both	+6225	main span, $x \approx 360$	+6832	TY side span, $x \approx 1392$
HSLM-A2, both	+19841	MW tower, $x \approx 5$	+23206	TY side span, $x \approx 1392$

Table 5.6: Largest increment of the cable axial force over the dead-load state, with its longitudinal position. Ma Wan tower at $x = 0$, Tsing Yi tower at $x = 1377$ m.

5.6.5 Vertical displacement and longitudinal deck slope

For the thermal cases the vertical displacement of the deck and its longitudinal slope are examined along the centreline of the main span, the slope being the quantity relevant to rail traffic. The values are collected in Table 5.7. The two senses of the uniform component produce the largest displacements, the summer sense lowering the mid-span by 0.85 m and the winter sense raising it by 1.07 m, the deck following the thermal movement of the cables, which lengthen in summer and shorten in winter.

The change of mid-span displacement between the two senses, 1.92 m over the 41°C range of the effective temperature, corresponds to a sensitivity of about $-47\text{ mm}/^\circ\text{C}$, close to the $-48.11\text{ mm}/^\circ\text{C}$ that Zhang et al. [32] obtain from the regression of the measured mid-span deflection against the deck effective temperature, and to the $-50.24\text{ mm}/^\circ\text{C}$ they derive analytically; the same study attributes the movement to the main cable, the short mid-span suspender making the deck deflection almost equal to the cable displacement. The vertical gradient, by contrast, produces a mid-span displacement of only about 9 mm: the gradient tends to curve the deck, but the suspension system restrains it, so its effect appears as chord stress rather than as displacement. The transverse difference produces no longitudinal displacement, tilting the section across its width instead.

Load case	Mid-span u_z	Maximum slope
Uniform, summer	−0.85 m	0.13°
Uniform, winter	+1.07 m	0.16°
Vertical, measured	+0.009 m	0.01°
Transverse, measured	≈ 0	≈ 0
Measured train, both	−0.51 m	0.12°
HSLM-A2, both	−1.44 m	0.32°

Table 5.7: Increment of mid-span vertical displacement and maximum longitudinal deck slope. Negative u_z is downward.

The airport-railway design adopts a limit on the vertical angular deflection of the rail of 0.24° , set to avoid the high accelerations caused by abrupt changes in rail alignment under the running train [22]. All the thermal slopes, and the slope of the measured service train, stay below this limit; only the conservative HSLM-A2 envelope exceeds it. The limit is a criterion for the moving load, where abrupt alignment changes under the train govern passenger comfort, whereas the thermal slope is a slow seasonal variation of the base profile. The slow longitudinal and angular movements of the deck are taken up by the rail movement joints provided at each end of the deck for that purpose [22], so the comparison of the thermal slope against the comfort limit is indicative rather than a literal verification. On this basis no thermal action produces a longitudinal slope that would impair rail traffic: the vertical gradient, the component that most loads the chords, produces a negligible displacement, while the displacement of about one metre comes from the uniform component, whose slope nonetheless remains within the design limit.

This thermal displacement is of the same order as that of the rail traffic, and is consistent with the long-term monitoring of the bridge, which identifies temperature as the primary contributor to the mid-span deck displacement [32] and records a daily temperature-induced vertical movement of about 220 mm at mid-span, the seasonal extremes being correspondingly larger [22].

5.6.6 Summary of the comparison

The question of chapter 1, whether the thermal action effects on the truss deck are of the same order as those of the rail traffic, is answered in the affirmative for the measured thermal field. At the chords the measured vertical gradient exceeds the measured service train, reaching 1.28 of its effect, and the transverse gradient approaches it at 0.94; against the conservative HSLM-A2 envelope (high-speed load model of EN 1991-2) the same effects fall to 0.86 and 0.64. At the main cable the winter uniform component equals or exceeds the service train, at 1.01 and 1.11 for the main and the side cable. The vertical displacement induced by temperature is of the order of one metre, comparable with that of the rail traffic, while the longitudinal slope it produces stays within the design limit of

the railway. The thermal actions therefore represent action effects comparable with, and at the chords and the main cable exceeding, those of the service train.

Chapter 6

Sustainability analysis and ethical implications

In accordance with the requirement established by the Governing Council of the Universitat Politècnica de Catalunya, this chapter examines the sustainability of the work and its ethical implications. The discussion is organised along the three dimensions of sustainability, economic, environmental and social, and relates each of them to the subject of the thesis, the assessment of the actions that govern a long-span road-rail suspension bridge. The work is a research study and proposes no physical intervention, so the analysis is qualitative and concerns how a more accurate characterisation of these actions bears on the cost, the environmental footprint and the social function of structures of this kind.

6.1 Sustainability analysis

6.1.1 Economic dimension

Long-span suspension bridges are among the most expensive works of civil engineering, both in their construction and in their maintenance over a service life measured in decades. The construction contract of the Tsing Ma Bridge was worth about HK\$7.2 billion at the time of the work [27]. A structure of this scale cannot be replaced cheaply, so its economy rests on building it no heavier than necessary and on keeping it in service for as long as possible.

Both objectives depend on an accurate prediction of the actions that load the deck, both to size the structure correctly and to plan its maintenance. The analyses reported in this thesis place the thermal and the railway action effects on a common footing, so that their relative weight on a long-span road-rail truss deck can be judged at the level of the resulting stresses. They show that the thermal effects reach the same order as the effects of the rail traffic carried by the bridge. An assessment that accounts for these actions therefore supports the correct sizing of the structure and the planning of targeted

inspection and maintenance. By extending the useful life of the bridge and deferring or avoiding its replacement, which is the dominant cost, such an assessment is economically favourable, an outcome consistent with the long-term monitoring of the bridge reported by Zhang et al. [32].

6.1.2 Environmental dimension

The economic argument carries an environmental one of the same form. The material embodied in a long-span bridge represents a large quantity of carbon and of resources: the steel wire in the main cables, the structural steel in the deck, and the concrete of the two 206 m towers, all carry the emissions of their production and transport. Replacing such a structure would release a comparable quantity of emissions a second time and would consume new raw materials on the same scale.

Extending the service life of an existing bridge, rather than demolishing and rebuilding it, avoids this burden. The contribution of the present work is indirect but aligned with this aim: by quantifying the action effects on the deck, including the thermal ones, it supports condition-based assessment and maintenance that keep the structure in service. This is consistent with the principle of prolonging the use of existing infrastructure in preference to new construction. The thermal actions examined here depend on the temperature regime of the site, which a changing climate is shifting towards more frequent and more extreme values. Italy, among other countries, has in recent years recorded summer temperatures without historical precedent, and an increase in the maximum and the minimum temperatures used for structural design is expected across the whole country [35]. This bears directly on the present work: the characteristic values of EN 1991-1-5 [20] derive from historical climatic records, and the European maps for thermal design do not yet account for the influence of climate change, so the same report recommends that the climatic parameters relevant to structural design be re-examined at regular intervals of no more than ten years and that structures, bridges among them, be designed for temperature amplitudes justified by the climate projections of their region [35]. Since the thermal effects were found here to reach the same order as those of the railway traffic, an under-estimation of the thermal action carries directly into the safety margin and the service life of a deck of this form. Assessing the thermal actions on existing long-span bridges, and revisiting them as the climatic record evolves, is therefore part of adapting this infrastructure to future loading conditions and of keeping it safely in service rather than replacing it.

6.1.3 Social dimension

Bridges of this kind exist to connect places, shortening travel times and sustaining access between them. The Tsing Ma Bridge is the principal structure of the Lantau Link, the

road and rail connection between the urban areas of Hong Kong and the airport on Lantau Island. Its two-level deck carries a dual three-lane highway on the upper level and the airport railway on the lower level, and when the hourly mean wind speed recorded on site exceeds 165 km/h, both road decks are closed to vehicles and the sheltered railway on the lower level becomes the only land connection between Lantau and the rest of Hong Kong [22]. A structure with this role shortens journeys and sustains emergency access, and its loss or prolonged closure would isolate communities and disrupt the services that depend on it.

The social value of the bridge is therefore tied to its safe and continuous operation, which is in turn tied to the actions treated in this work. The serviceability of the railway is governed by limits such as the one on the vertical angular deflection of the rail of 0.24° , set to avoid the high accelerations that abrupt changes in rail alignment would cause under a running train [22]. The analyses of this thesis show that the thermal slopes, and the slope under the service train, remain within this limit, so that the actions considered do not, on their own, impair the running of the railway. An accurate account of these actions supports the safe operation of the crossing and reduces the risk to its users, which is the social purpose the structure serves.

6.2 Ethical implications

The work responds to the need to quantify the relative weight of the thermal and the railway action effects on an existing long-span road-rail bridge, a question that bears on how such structures are assessed and maintained. The principal ethical consideration follows from the finding that the single recommended value of EN 1991-1-5 for the transverse temperature difference [20] can understate the action effect of the measured transverse field by a factor of about three, which is relevant to the responsible assessment of structures of this type. The results are presented with their assumptions stated openly, including that the transverse temperature field was adopted from a comparable bridge in the absence of a record for the Tsing Ma Bridge. For this reason the work is offered as a basis for further study and not as a verification ready for design use, so that its conclusions are not applied beyond the evidence that supports them.

6.3 Relation to the Sustainable Development Goals

The work can be associated with three of the United Nations Sustainable Development Goals [36]. The main connection is to Goal 9, on industry, innovation and infrastructure, which the work supports by improving the understanding of the actions on an existing structure through advanced finite element modelling and by favouring its assessment and continued service rather than its replacement. Two further connections are secondary. The

work relates to Goal 11, on sustainable cities and communities, because a clearer account of the actions that load the deck supports the safety and the continuity of a connection on which the surrounding communities depend. It relates to Goal 13, on climate action, because the thermal actions analysed depend on the temperature regime of the site, so that their assessment contributes to the adaptation of existing infrastructure to climate-driven loading conditions.

Chapter 7

Conclusions

This work set out to establish whether the thermal action effects on a long-span road-rail truss deck reach the same order as those of the railway traffic for which the deck is dimensioned, taking the Tsing Ma Bridge as the case study. The two actions had not previously been placed on a common footing. Temperature enters the literature on these decks mainly through monitoring studies that reconstruct the temperature field without carrying it through to the resulting stresses, while the railway action is taken as the service traffic load against which the thermal effects are weighed in this work. The question was addressed at the level of the action effects, through a three-dimensional finite element model brought to its tensioned dead-load configuration and loaded, in turn, by three thermal components and two railway load models applied as independent increments on that configuration. The model reproduces the dead load, the main-cable tension and the measured modal properties of the bridge closely enough to be trusted before any action is applied, and the comparison was drawn in terms of ratios between the thermal and the railway effects at a common reference section, a form that is robust to the modelling choices on which the absolute stresses depend.

The central finding, set out in full in chapter 5, is that the thermal action effects of the measured field reach the same order as those of the railway traffic for which the deck is dimensioned. At the chords the stresses due to the vertical gradient exceed those due to the full service train, those due to the transverse gradient approach them, and at the main cable the seasonal uniform component equals or exceeds it; only against the deliberately conservative code envelope do the thermal effects fall below the railway action, and even then they remain a substantial fraction of it. The temperature therefore acts on a truss deck of this form as a structural action comparable with the traffic, rather than merely as a source of deck deformation to which the track is sensitive. This reframes the role of temperature on long-span road-rail truss decks, where the depth and the width that the section needs in order to meet the stiffness demands of railway traffic are the same features that expose it to thermal gradients, so that the action and the geometry that resists the traffic are bound together.

A second conclusion bears on the design provisions. The comparison between the borrowed transverse field and the recommendation of EN 1991-1-5 shows that the single low value the code prescribes for the transverse difference, fixed independently of the depth or the width of the section, understates the action effect of a directly exposed truss deck by roughly a factor of three [20]. Among the three components the transverse difference is thus the one least well represented by current practice relative to the effect it produces, a finding that agrees with the conclusion drawn by Zhang et al. [19]. It should be borne in mind, in any case, that the reference values of EN 1991-1-5 [20] are representative of short and medium-span bridges and are not calibrated for long-span structures such as the Tsing Ma Bridge, so that the single recommended transverse value is all the less suited to a deck of this span. The practical consequence is the one developed in the preceding chapter: an assessment that accounts for the thermal actions supports the correct sizing and the condition-based maintenance through which a structure of this scale is kept in service rather than replaced.

7.1 Limitations and recommendations for further work

The conclusions rest on assumptions that bound their reach, and the work is offered as a basis for further study rather than as a verification ready for design use. The most consequential assumption concerns the transverse component. No record of the transverse temperature difference exists for the Tsing Ma Bridge, whose monitoring system resolves the temperature field through the depth of the cross-section but not across its width, and the values applied here were borrowed from the road-rail steel truss cable-stayed bridge monitored by Zhang et al. [19]. The two structures share the deck typology but not the global system, and the borrowed field should be read as a representative loading scenario rather than as site data. A direct measurement of the transverse gradient on the Tsing Ma deck, through chord-mounted sensors on the two sides of the lower level, would replace the principal estimate of the study with a site-specific value and is the first recommendation for further work.

A second limitation follows from the modelling of the deck. The true box, I and T sections of the framework are not reported in the public literature on the bridge, so each member was represented by an equivalent solid rectangular section, with its dimensions fixed by calibration of the bridge response under the dead load and the natural frequencies. The chord stresses are recovered from the axial force divided by an assumed chord area, so the absolute stress values depend on this choice; the comparison was therefore framed throughout in terms of ratios at the same point, which are far less sensitive to it than the absolute values.

The railway action was treated statically, with the dynamic magnification of the code envelope introduced through the dynamic factor for real trains of EN 1991-2 applied as an equivalent static amplification [34]. The factor was retained as a conservative upper bound, although the loading frequency of the convoy at line speed lies far above the global natural frequencies of the bridge, so that resonance of the global response cannot develop and the true dynamic amplification tends to unity. It was applied to the code envelope only and not to the measured service train, which slightly widens the railway action against which the thermal effects are measured; the ratios reported against the measured train are accordingly free of this effect. The thermal analysis was conducted in steady state, the temperature field being applied as a final equilibrium condition; a transient thermal analysis, coupling the field reconstruction of Xia et al. [31] to the mechanical model, would capture the daily redistribution that the steady-state assumption omits.

Two further directions follow from the findings rather than from their limitations. The thermal stresses at the chords are cyclic, varying over the daily and the seasonal cycle, so their bearing on the fatigue assessment of the deck, treated for traffic alone by Lu et al. [29], deserves examination on a deck where the thermal effect reaches the order of the traffic effect. And since the transverse gradient is governed by the differential solar exposure of the two sides of an open truss, the comparison would extend naturally to the box-girder decks increasingly adopted for long-span road-rail crossings, whose closed section redistributes the thermal field differently and may attenuate the transverse component that dominates here.

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