Kap Shui Mun Cable-Stayed Bridge

Steven L. Stroh and Thomas G. Lovett, *Greiner, Inc., Florida*

The Kap Shui Mun Bridge is one of the world's largest double-deck cable-stayed bridges and features the first fully enclosed lower deck. The bridge spans Kap Shui Mun Channel linking Lantau Island and Ma Wan Island, providing access to the proposed new airport in Hong Kong. The bridge carries six lanes of roadway traffic on the upper deck and two lanes of emergency roadway traffic and two tracks of light rail on the lower deck. The winning design/build tender is described; it is an innovative hybrid design with a steel composite superstructure for the central 387 m of the 430-m center span and an all concrete superstructure for the remainder of the main span and for the 160-m side spans.

A new system of access roads and railways is required as a part of the relocation of the Hong Kong airport from Kai Tak to Chek Lap Kok at the northern end of Lantau Island. A major part of this system is the Lantau Fixed Crossing, composed of a system of bridges linking Tsing Yi Island to Lantau Island. The Kap Shui Mun Bridge forms a major part of the Lantau fixed crossing, spanning the Kap Shui Mun Channel between Ma Wan Island and Lantau Island. Figure 1 presents a photomontage of the Kap Shui Mun Bridge, with Lantau Island in the foreground and Ma Wan Island in the central part of the photo. The Kap Shui Mun Bridge is a double-deck, five-span, cable-stayed bridge carrying both vehicular and rail traffic.

Design/Build Tender

Structure of the Design/Build Team

The winning design/build tender was prepared by a consortium of international contractors and engineers. The contracting team was composed of Kumagai Gumi, Maeda, Yokogawa, and Hitachi Zosen. The bridge was designed by the Greiner International Ltd. Tampa and Timonium offices in the United States and Leonhardt Andrä and Partner Ltd., Stuttgart, Germany, in conjunction with Fugro Hong Kong Ltd. for geotechnical engineering and site work, J. Roger Preston & Partners for electrical and mechanical work, Hong Kong, and Kennedy & Donkin Ltd., Hong Kong, for track work.

Geometric Requirements

The crossing of the Ma Wan Channel requires a navigation channel width of 275 m and a vertical clearance of 47 m above reference datum. The horizontal alignment of the cable-stayed portion of the bridge is straight with the exception of approximately 45 m of one side span, which is located in a transition spiral. The vertical alignment is curved for the central 305 m of the main span with R = 10,175 m, and the tangent for the remainder of the cable-stayed structure on a 1.5 percent grade.

The tender requirements stipulated that the lower carriageways be constructed with a constant 2.5 percent

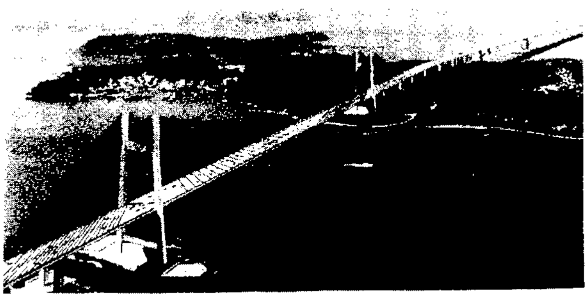

FIGURE 1 Photomontage of Kap Shui Mun Bridge.

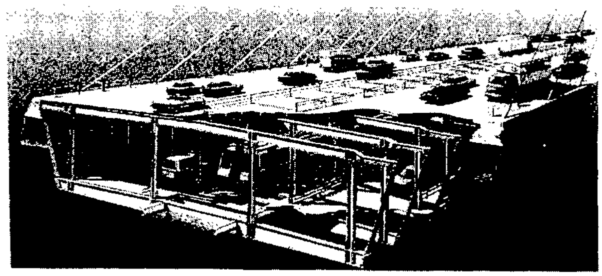

FIGURE 2 Artist's rendering of the deck configuration for the Kap Shui Mun Bridge main span.

cross slope while the upper carriageways maintained the same 2.5 percent cross slope through the tangent sections with a transition to a maximum 2.82 percent superelevation through the curved portion of the alignment. This requirement resulted in an extremely complex geometric criteria for the concrete box sections and greatly complicated the reuse and standardization of the form system. This provision was necessary to ensure that spillage of any hazardous materials on the lower carriageways would not drain into the rail corridor. After the contract award the owner relaxed this requirement, thus allowing both upper and lower carriageways to remain parallel. This concession was granted given that this design maintained a solid concrete web wall between the rail and lower carriageway alignments throughout the superelevated sections, thereby eliminating the concern for contamination of the rail corridor.

Traffic Requirements

The bridge cross section is a double-deck arrangement, providing six lanes of road traffic on the upper carriageway, two fully enclosed roadway lanes on the lower carriageway, and also two tracks of railway on the lower carriageway. Figure 2 provides an artist's cutaway of the main span deck layout. Under normal operating conditions the upper carriageways are used for roadway traffic and the two lower roadways are used for maintenance or emergency vehicles, or both. Under high wind conditions, roadway traffic is shifted to the lower carriageway. Specifically, operating requirements are as follows:

- Wind speed less than 40 kph: all vehicles use upper deck;
- Wind speed 40 to 65 kph: high-profile vehicles shifted to lower deck;
- Wind speed 65 to 90 kph: all vehicles shifted to lower deck; and
- Wind speed over 90 kph: bridge closed to all vehicles except emergency use.

The full enclosure of the lower carriageway necessitated ventilation of the interior of the structure. The main span, which has openings in the central region of the deck (both top and bottom) and small openings along the lower outside edges of the structure, is naturally ventilated. The concrete side spans have top openings in the central region of the deck (above the railway) but a closed bottom soffit (in accordance with a contract requirement to mitigate noise impacts along the Ma Wan corridor). These openings were insufficient for natural ventilation and the side spans are therefore mechanically ventilated with fans.

The upper and lower carriageways are designed for a total of six lanes of live load, positioned between the upper and lower carriageways to produce the most adverse effect. Loading intensities were developed in accordance with the British Design specifications (BS5400) with 45 units of HB loading. The rail load considered was a standard 8-car train, as shown in Figure 3. Also considered was a 10-car train with similar

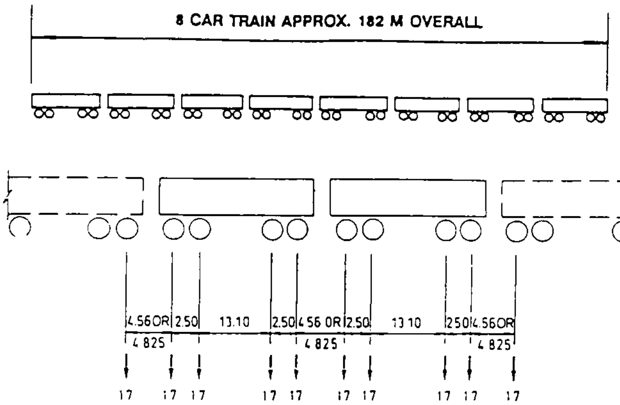

FIGURE 3 Standard railway loading (units are tonnes and meters).

bogie spacing, as shown in Figure 3, but a 13.6-tonne axle load, a check bogie composed of two 24 tonne axles 1.5 m apart, and for fatigue loading, an 8-car train configured as shown in Figure 3 but with a 13-tonne axle load. Stringent dynamic criteria were imposed for both the transverse supporting elements of the rail and the global response of the structure to ensure user comfort. The live load dynamic response criteria were evaluated in a manner consistent with the level of service required for the range of wind speeds explained above.

Wind Loading Requirements

The Kap Shui Mun Bridge, and Ma Wan Viaduct are located in an exposed site in a region frequented by tropical cyclones. In addition to static load requirements requiring investigation of sustained wind speeds of 80 m/sec, aerodynamic stability also had to be guaranteed for horizontal wind speeds of up to 95 m/sec for the unloaded bridge and up to 50 m/sec for the loaded bridge.

Aesthetic Requirements

The Kap Shui Mun Bridge and Ma Wan Viaduct were required to meet a specific set of aesthetic goals aimed at ensuring that the Lantau Fixed Crossing Route presented an integrated system from an appearance viewpoint. These aesthetic requirements included the following:

- Provision of a tower compatible with the adjacent Tsing Ma Suspension Bridge, including the overall form as well as the local member details (member rectangular cross section with 1-m radius at corners);
- Provision of intermediate piers similar in form and local member details to the adjacent Tsing Ma Bridge;
- Provision of a mid-depth feature line along the edge of the deck, with the deck edge inclined to reflect the light above and recessed to cast a shadow below the line. (In addition to the mid-depth feature line, an unbroken bottom soffit line was required); and
- Provision of bridge furniture of similar detail to the adjacent projects (barriers, maintenance walkways, sign supports, light poles, and other items not a part of the main structure).

The provision of the unbroken bottom soffit proved to be a challenging aspect of the design. Given that the interior of the section was fully utilized for highway and rail traffic, all pier diaphragms were required to be located around the perimeter of the section. Initial attempts to locally thicken the box section at the piers were rejected by the owner. Ultimately, diaphragms were developed by increasing the level of reinforcing in the webs, without an increase in thickness, combined with a locally thickened bottom slab that is blended with the pier columns.

Kap Shui Mun Bridge Details

General Layout

The main structure is a five-span cable-stayed bridge consisting of a 430-m main span and side spans composed of two 80-m span's each, for a total cable-stayed structure of 750 m (Figure 4). The sidespan lengths of 160 m (side span to center span ratio of 0.37) are governed by geometric constraints and are substantially shorter than the ideal length for optimization of a cable-stayed bridge, particularly for one with large live loading. To limit the uplift from this condition, several measures were taken. The side spans were stiffened with an intermediate pier. This pier is also used for erection of the side span superstructure, as will be discussed later. The side span is constructed of prestressed concrete, whereas the central 387 m of main span is steel composite with concrete upper and lower decks, providing a comparatively lighter section than the all-concrete side spans. This heavier section significantly counters the uplift from the relatively short side span and, taken in conjunction with the stiffening provided by the intermediate pier, provides a very stiff side span. This stiff side span further permits all of the side span stays to function as a distributed backstay and avoids the necessity of a concentrated backstay, which is common to cable-stayed bridges.

The deck is supported by two planes of cables in a semifan arrangement. The cables are arranged in a vertical plane, spaced at 8.7 m in the main span and 6.1 m in the side span. The cables are spaced at 1.5 m at the tower head. The fixed end of the stay is at the tower and the stressing end at the deck.

Side Span Superstructure

The side spans are a concrete box girder arrangement, which is composed of two separate concrete box girder sections representing the northbound and southbound roadways. These box girders are erected by incremental launching, with the northbound and southbound roadways launched separately. On completion of launching, the two sections are joined at the bottom slab level by an inverted T-beam (rail support beam) and solid bottom slab, and at the top slab level by the beams 300 ×

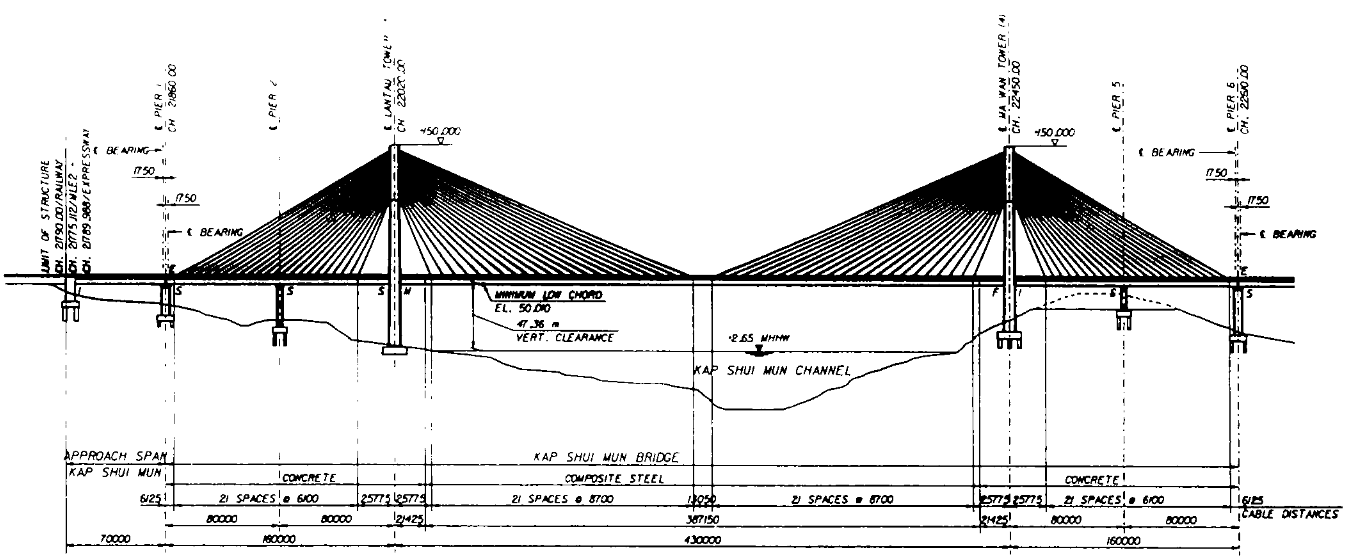

FIGURE 4 Elevation: Kap Shui Mun Bridge.

850 mm at 3-m spacing (Figure 5). This section extends through the entire 160 m of the side span and further extends 21 m into the main span.

The railway is supported by cast-in-place inverted T-beam members. These members are spaced at 3 m to provide support for the trackslabs. The side spans have a continuous closed bottom flange beneath the railway, in response to a contract requirement for a fully closed bottom soffit for noise abatement. The top slabs are joined by concrete struts, which provide diaphragm action and provide continuity for flow of torsional stresses around the overall cross section.

The side span stays are anchored in a concrete edge beam at the outer edges of the section. Transverse post-tensioning is provided in the webs and bottom slab to resist the local introduction of stay forces. Longitudinal

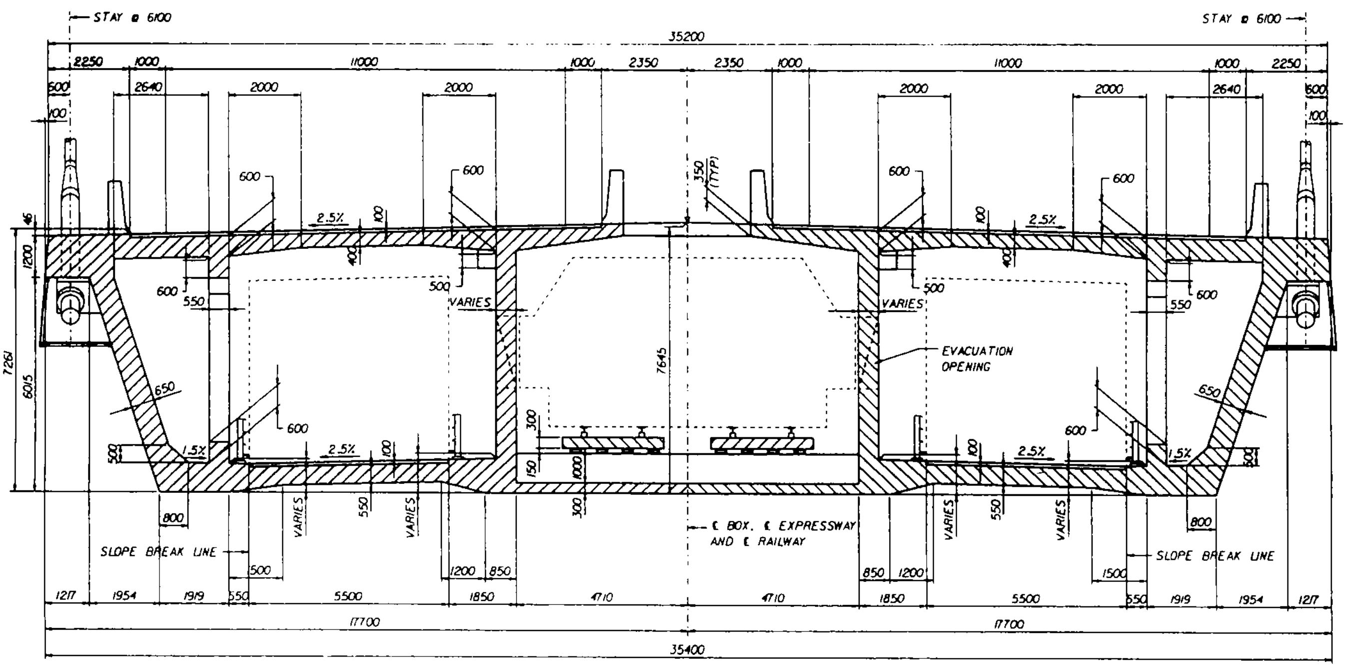

FIGURE 5 Typical section of side span superstructure.

post-tensioning is provided to satisfy the erection and final longitudinal forces.

Main Span Superstructure

The center 387 m of the 430-m main span is a steel framework with composite concrete carriageway decks (Figure 6). The section is fully composite for both the upper and lower carriageways for both dead loads and live loads. The transverse steel frames are spaced at 4.35-m centers and support a nominal reinforced concrete slab 250 mm thick. The slabs are locally thickened to 500 mm at the edges of the section for local force introduction (stay cable introduction). The depth of the superstructure varies from 7.3 m at the center of the section to 6.8 m at the edges. The transverse frames are designed considering vierendeel truss action. Top lateral bracing is provided in the form of X-bracing in alternate bays. This bracing is provided to increase the overall torsional stiffness of the deck, which was necessary to ensure satisfactory aerodynamic behavior. The top slab chamfer feature was provided as a consequence of the wind-tunnel testing to ensure acceptable aerodynamic behavior.

The main girder longitudinal load-carrying system is composed of the longitudinally stiffened web, a trussed inner "web," and the composite concrete slabs. The outer web is connected to the sloping edge of the cross frames. Only nominal steel flanges are required, as the upper and lower carriageway slabs are fully composite for both dead loads and live loads. The main girder field sections are 8.7 m long and contain two cross frames. These field sections are fully shop welded and then are joined to adjacent sections by bolted connections.

The railway envelope is located within the center portal and supported by precast prestressed trackslabs seated on the lower chords of the cross frames. An intermediate floor beam is located between cross frames to satisfy a 3-m maximum trackslab span requirement of the project specifications. The stays are anchored in steel anchor boxes, shop welded to the frames at cross-frame locations.

Launching Nose

The steel launching nose is a unique feature of this structure. This transition element between the normal steel composite main span and the concrete side spans serves a dual purpose as a lunching nose for incremental launching of the side span superstructure and then is incorporated into the permanent structure as the transition element. The length of the launching nose taken in conjunction with the portion of the side span superstructure that is launched through the towers was carefully selected to ensure that adequate water depth for erection of the first composite main span sections from a barge was available.

Stay Cables

The cables are composed of 51 to 102 parallel mono strands, 0.6 in. in diameter, with a tensile strength of

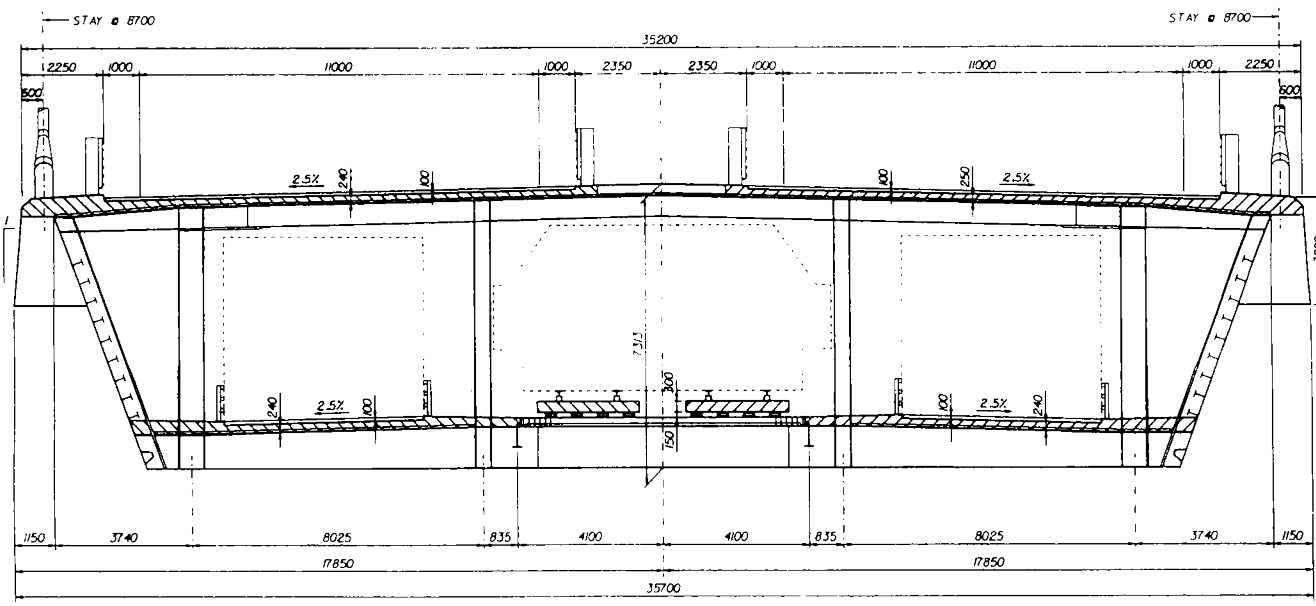

FIGURE 6 Typical section of main span superstructure.

1720 N/mm^2 after galvanizing. The corrosion protection system consists of hot dip galvanizing with 280 g/m^2 corrosion inhibitor grease and a 1-mm polyethylene sheathing around each strand. The entire cable group is further encased in a polyethylene pipe, which remains in its black condition. The strands are anchored in sockets, with permanent loads carried by high fatigue-resistant wedges. Live loads are carried by a combination of these wedges and epoxy bond.

The strands will be individually stressed with mono jacks to the same sag. Subsequently, the entire stay will be stressed with calibrated hydraulic jacks with a maximum capacity of 1500 tonnes. The structure has been designed to allow replacement of individual stays and for the accidental loss of a stay under full live load, ensuring structural stability.

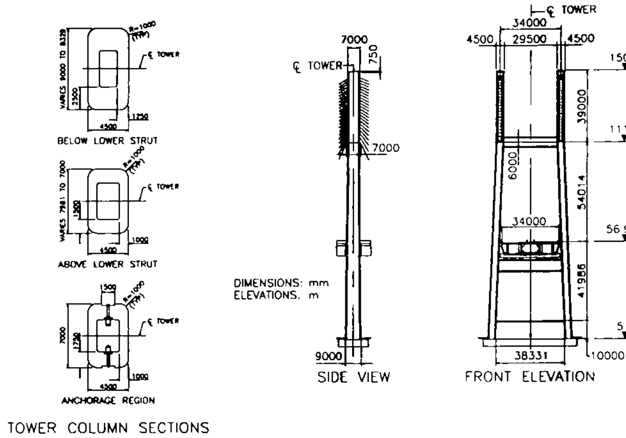

FIGURE 7 Details of Lantau Tower.

Bearings

The superstructure is integral to the Ma Wan Tower, so all longitudinal forces are resisted at this location. At the Lantau Tower, vertical forces are resisted by pot bearings on top of the cross girder and laterally by neoprene bearings between the tower legs and the upper and lower deck.

At intermediate piers prestress forces are resisted by pot bearings, and hold-down forces are provided by hold-down cables at the outer webs. These hold downs consist of 3 cables of 68 (or 98, depending on the location) 0.6-in. strands, anchored in the lower portion of the piers. The hold-down cables are sized such that two cables are sufficient for stability to permit future replacement of the cables, one at a time.

The intermediate side span locations are free to translate longitudinally and transversely, and at the end pier locations the deck is free to move longitudinally but restrained laterally. Lateral restraint is provided by steel shear lock assemblies.

Towers

The towers are H-shaped, as indicated in Figures 7 and 8. The upper region of the towers is vertical, for anchorage of the vertical plane of stay cables. Below the upper strut the tower legs are slightly inclined to clear the superstructure. The tower height is 93 m above the deck, corresponding to a ratio of 0.22 with the main span.

The tower legs are constructed of conventionally reinforced concrete. A horizontal strut is provided just below the lower stay location to resist deviation forces from the change in angle of the tower legs. A second strut is provided below the deck to provide support of the superstructures. Both struts are post-tensioned. The stays are anchored directly in the outer tower walls. Splitting is resisted by loop post-tensioning tendons and short post-tensioning bars.

The Lantau Tower is founded on a spread footing with an allowable bearing pressure of 5.0 MPa. The Ma Wan Tower was originally planned to be founded on hand-dug caissons 4 m in diameter. However, the excavation of the foundation revealed highly variable founding levels that precluded this type of foundation. The final configuration for the Ma Wan Tower was a combined foundation, composed of a spread footing over approximately 80 percent of the footing and two supplemental caissons 4 m in diameter at the southeast corner of the footing.

Piers

Individual pier shafts support each of the concrete box sections and serve to anchor the side span uplift forces.

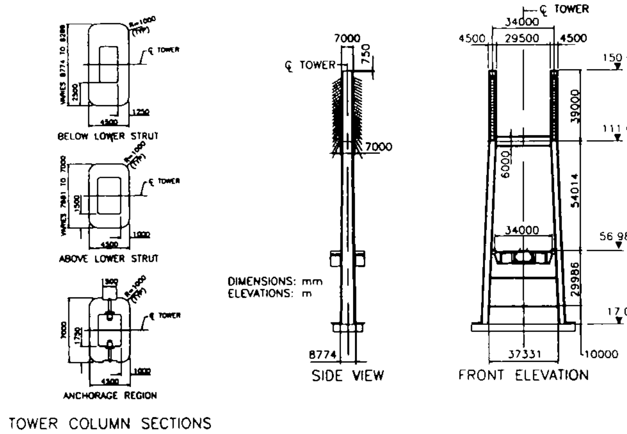

FIGURE 8 Details of Ma Wan Tower.

The piers are constructed of conventionally reinforced hollow concrete box sections. Their mass is engaged through vertical hold-down cables. These cables are anchored near the bottom of the piers to provide sufficient length to minimize cable bending as the superstructure undergoes longitudinal movements. The side span piers are founded on spread footings or hand-dug caissons, depending on location. Bearing pressures range from 4.5 to 7.5 MPa.

Erection

General

The total construction time is 54 months, commencing on 16 November 1992 and opening to traffic on 18 May 1997. Included in this design period is the preparation of the design, as well as all testing (aerodynamic, ventilation, trackslab, etc.).

Tower Construction

The towers will be constructed using a jump-forming system with typical lift heights of 3.9 m. The production schedule is based on approximately 5 days per typical lift. The design permits the side span superstructure to be launched across the lower tower strut before completion of the upper portion of the tower to expedite construction.

Side Span Erection—Incremental Launching

The side spans will be constructed by incremental launching, with segment lengths of approximately 18.30 m. The incremental launching method was selected in response to the inaccessibility of the terrain and for the advantage of the construction schedule so that construction of the towers and sidespan can proceed simultaneously. The bridge deck is launched in two parts, corresponding to the left and right halves of the superstructure. The cross girders that support the railway are cast in a second step after launching.

Temporary intermediate piers are provided midway between the side span piers to limit the maximum span during launching to 40 m. The Lantau sidespan is launched from a casting bed behind the Lantau abutment. The Ma Wan sidespan is launched from an elevated casting bed within the first 40 m of the side span.

Main Span Erection—Free Cantilever

The steel composite main span is erected by free cantilevering simultaneously from both towers. The individual elements are lifted together with their precast decks and joined by bolted connections and cast in place slab closures. The erection weight of the individual elements is 500 tonnes. A "purpose-made" erection traveler is used to hoist the superstructure elements. This assembly consists of two main girders and two cross girders and weighs approximately 390 tonnes.

Acknowledgments

The owner is the Hong Kong Department of Highways. The engineer for the project is Mott MacDonald, Hong Kong Ltd., with subconsultants Flint & Neill Partnership, London, and Harris & Southerland (Far East) assisting in checking the design of the steel and concrete structures, respectively. The design team was augmented by several specialty consultants: Robert H. Scanlan of the Johns Hopkins University, for wind stability studies; West Wind Laboratory, Carmel, California, for wind tunnel modeling and testing; Laurie Richards, New Zealand, for foundations (rock mechanics); Nick Jones of the Johns Hopkins University, for dynamic studies; and Ashdown Environmental, United Kingdom, for acoustic analyses.