Stonecutters Bridge Towers

Stonecutters Bridge, Hong Kong, is a cable stayed structure with a total length of 1596 m and a main span of 1018 m. Opened at the end of 2009, the bridge crosses the Rambler Channel and is the main entrance to the busy Kwai Chung Container Port. It is visible from many parts of Hong Kong Island and Kowloon. The most striking features of the bridge are the twin tapered mono towers at each end supporting the 50 m wide deck. These tapered towers rise to 295 m above sea level; the lower sections are reinforced concrete while the upper 115 m are composite sections with an outer stainless steel skin and a reinforced concrete core.

Material Selection

Figure 1: General view of Stonecutters Bridge

The design life of the bridge is 120 years. A highly durable material was required for the upper sections of the bridge towers because of the harsh marine and polluted environment. Additionally, post-construction maintenance on the towers will be extremely difficult, due to the live traffic beneath. Stainless steel was chosen for the skin of the composite section of the upper tower because of its durability and also its attractive appearance. Carbon steel would have required protective coatings that would have needed replacing after an estimated 25-30 years.

Standard molybdenum-alloyed austenitic steel grades were initially considered but discounted because of their relatively low design strength (220 N/mm^2) and uncertainty regarding corrosion performance, given the roughness of the desired surface finish. Higher alloyed austenitics with better corrosion resistance, e.g. 1.4539 (N08904) and 1.4439 (S31726), were not considered in detail as they would not have met the requirements for cost, availability and strength. Duplex steel 1.4462 (S32205) was chosen as it has high strength (460N/mm^2) with good corrosion resistance and tolerance on surface finish.

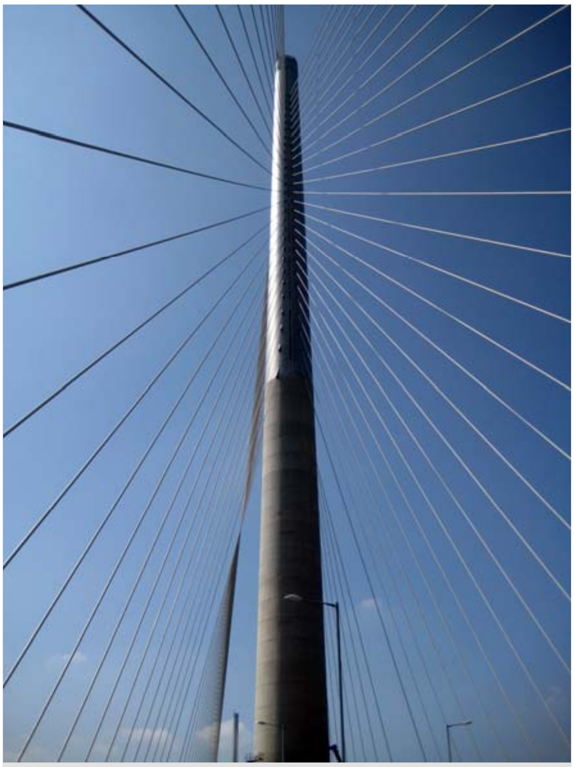

Figure 2: Mono tower and stay cables

A polished 1K finish (as defined in EN 10088 Part 2 [1]) was specified for all exposed surfaces, with an average surface roughness R_a of 0.5 µm. A slightly textured, non-directional, low reflective appearance was then created by shot peening the surface with a mixture of aluminium oxide and glass beads.

Design

The initial design for the upper 115 m of the towers was an all structural steel section. However, circular towers are known to be susceptible to vortex shedding vibration. An investigation showed that the frequencies of the lateral oscillations at the top of this tower coincided with the natural frequencies of the stay cables; this meant that excitation of the cables due to linear resonance was possible.

To avoid this, the design of the upper part of the tower was changed to a composite structure with a stainless steel skin around a reinforced concrete annular core enclosing the steel cable anchor boxes. This structural form showed an improved response to vortex shedding, due to its greater mass, stiffness and damping.

The tower skin is made up of 32 stainless steel segments (Figure 3), each of 20 mm thick hot rolled plate and varying in height from 5.6 m to 3.2 m. The diameter of the circular tower section tapers from 10.9 m to 7.2 m with the concrete wall thickness decreasing from 1.4 m to 0.8 m. The lower three sets of stay cables are anchored into the reinforced concrete lower tower section. Above this there are 25 steel anchor boxes into which the remaining 25 sets of stay cables are installed. At the top of each tower, space is left for future installation of pendulum mass dampers, should these be required to mitigate tower vibrations.

The structural capacity of the upper tower section relies on composite action between the stainless steel skin, the concrete core and the anchor boxes. Duplex stainless steel shear connectors (Figures 5 and 6), 16 mm in diameter and up to 300 mm long, transfer the load between the concrete and the skin or anchor box. The number of connectors is determined by the requirement to ensure the transfer of all short-term and long-term loads between the skin and the concrete wall. The connectors are evenly distributed at approximately 300 mm centres to prevent local buckling in the skin and anchor box plates before the yield strength is reached. Carbon steel studs could have been used as they can be welded to duplex stainless steels; bimetallic corrosion was not considered a risk in this case as both steels would have been cast in concrete [2]. However the duplex studs were stronger.

A prototype segment of the stainless steel skin was fabricated before construction began in order to develop appropriate welding procedures. A range of fabrication techniques for achieving a suitable fit and bearing between contact surfaces were also studied on the prototype, alongside an investigation into how the required high quality finish could be obtained and how easily it might be reinstated.

Figure 3: Segment of stainless steel skin

Figure 4: Bridge under construction

Figure 5: Shear connectors on the inside of a tower segment

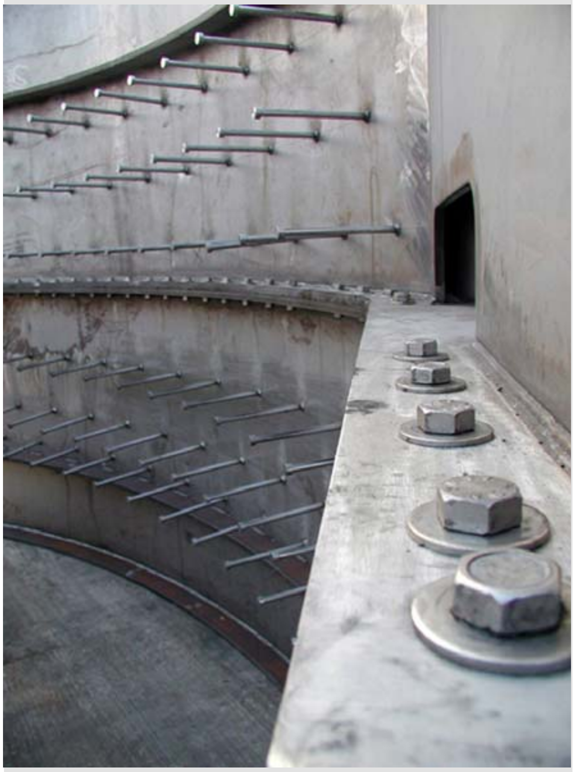

Figure 6: Edge flange connection

To facilitate lifting, each stainless steel skin segment was fabricated in two halves, with 25 mm thick stiffening flanges and 25 mm thick intermediate stiffening rings. High strength friction grip bolts of 22 mm diameter in duplex stainless steel were used for the vertical splices. The contact surface of the connecting plates required treatment to ensure that the coefficient of friction was greater than 0.2. The bolts were preloaded to achieve a shank tension of 165 kN. No slip was allowed at serviceability but slip was allowed at the ultimate limit state, with the remaining applied loads resisted in bearing.

The horizontal splices were assumed to be effective in resisting vertical compression forces only, with no shear stress being transferred. Since the height of the individual skin segments is small with respect to the overall section diameter, individual skin segments effectively act as separate horizontal hoops restraining the inner core. Therefore, the stainless steel skin is assumed to resist only direct stresses in the vertical and horizontal directions, not shear and torsion. The skin, however, does contribute to the shear resistance of the section by acting as a tension tie similar to reinforcing hoops (or links) in the section. The design of the skin plate was carried out such that the calculated stresses satisfy the von Mises yield criterion. The spacing of the shear connectors was governed by the shear transfer between the skin and concrete; the resulting spacing was such that the skin yields in compression rather than buckling.

The connection between skin and flange plates was a critical structural connection, requiring very close fit of the contact surfaces (maximum gap of 0.25 mm over 60% of the contact area). Machining the edge flanges to satisfy this requirement would have been extremely time-consuming and the large amount of welding required on site to achieve this would have been difficult to control and led to distortion. The solution was to bolt the edges together and use a high volume of welding laid by the manual metal arc process. Plates were tacked into position, flange plates bolted together and welding completed in sequence. It was found that whilst this method did not eliminate distortion entirely, it significantly reduced the size of gap and increased the contact area to within design code limits.

Fabrication and Erection

Fabrication: For typical half segments, four flat plates were connected using full strength butt welds and then the joined plate was rolled into the correct shape. However, for the first four skin segments, which are 5.6 m in height, the plates were rolled to the correct curvature before welding, because of the width limitations of the bending roller machine.

The fabrication works complied with BS 5400-6, clauses 4.1 to 4.16 except as stated in clauses 18.44 to 18.52 [3]. All tools and processing equipment used were dedicated to the processing of stainless steel and not used on other metallic items in order to prevent contamination.

Erection: The skin and anchor boxes were used as permanent formwork for the concreting. Erection of the skin and anchor box segments advanced two full cycles ahead of the concreting. Temporary attachments required for the handling and erection of stainless steel components were made from the same grade of stainless steel as that used for the structure. These attachments were generally located on surfaces which would not be visible on the finished structure, but where this was not possible, the surface finish on the stainless steel was reinstated to the specified finish after removal of the temporary attachment.

Welding: Guidance from IMOA on welding duplex stainless steels was adopted [4]. The consumables used for arc welding of stainless steel were chosen to ensure that the mechanical properties and corrosion resistance of the weld metal was not less than the parent metal requirements. Welders were qualified in accordance with EN 287 [5] and the welding procedures were in accordance with current European standards. Weld areas were bead blasted to maintain the uniform surface finish.

Figure 7: Installation of stainless steel skin segments

Information for this case study was kindly provided by Arup.

References and Bibliography

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[3] BS 5400-6:1999 Steel, concrete and composite bridges. Specification for materials and workmanship, steel

[4] Practical guidelines for the fabrication of duplex stainless steels. International Molybdenum Association, 2001 (*new Edition 2009*)

[5] EN 287-1:2004 Qualification test of welders - Fusion welding. Steels

Online Information Centre for Stainless Steel in Construction:
www.stainlessconstruction.com

Procurement Details

Client:	Hong Kong Highways Department
Designer:	Arup
Civil & Structural Engineer:	Arup
Main contractor:	Maeda-Hitachi-Yogogawa-Hsin Chong Joint Venture (MHYHJV)
Fabrication yard:	Zhongshan, Guangdong province, China

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