Engineering Sciences for Large Bridge Construction

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1. INTRODUCTION

The construction of large bridges demands very substantial engineering input throughout the construction planning and execution stages. In many forms of large bridges, the partial structures during the erection stages are vulnerable to the actions of construction loads and forces of nature. The safe construction of the partial structures through different structural systems to completion of the bridge, challenges the very best in the engineering sciences. The engineering sciences that make possible the safe and efficient construction of large bridges draw on a unique combination of complex theoretical principles and robust practical experience. Prime examples of the application of engineering sciences to tackling the technical challenges in large bridge construction are found in two projects involving the world's two longest cable-stayed bridges.

Stonecutters Bridge in Hong Kong is a high-level cable-stayed bridge, with two towers located in the back-up areas of Container Terminals 8 and 9, a main span of 1,018m across the Rambler Channel at the entrance to the busy Kwai Chung Container Port frequented by the world's largest container vessels, and of a total length, including the backspans, of some 1.6km. The main span closure of Stonecutters Bridge was accomplished in March 2009 and the bridge was opened to traffic in December 2009, The bridge is one of the longest cable-stayed bridges in the world.

Sutong Bridge in China crosses the Yangtze River, connecting the cities of Suzhou and Nantong. The 7-span cable-stayed bridge has a record long main span of 1,088m and a total length of 2.088km. The main span closure for the world's longest cable-stayed bridge closure was completed in June 2007 and the bridge was opened to traffic in May 2008. At the time of completion, Sutong Bridge was the longest cable-stayed bridge in the world.

The very significant construction engineering activities included contractor's design, development of construction methodology, stage-by-stage erection analyses and bridge geometry monitoring, control and adjustment; bridge aerodynamics and wind tunnel testing; vibration mitigation measures/devices; verification of permanent work safety and performance during construction; substantial falsework and cofferdam design; navigation studies and marine traffic management; marine jetty design; temporary traffic management; geotechnical engineering; together with research and development, amongst many other coordinated strands of engineering science.

Throughout our mission in constructing these two large bridges, innovation was an integral part of our activities. There were innovations through evolution, in that improvements on existing techniques are made progressively. There were also innovations through revolution, in that new, or even radically new, techniques were developed in the face of a difficult challenge.

The experience gained in the course of harnessing the challenges posed by these two large bridge projects was phenomenal. Working closely with construction contractors AECOM Asia Co. Ltd (AECOM) was part of the momentum in pushing the frontiers of large bridge construction, and is part of an even greater momentum in striving for the best innovations and further accomplishments in a new era. This paper provides a first-hand account of the engineering sciences applied to make possible large bridge construction.

2. STONECUTTERS BRIDGE

Stonecutters Bridge in Hong Kong is an ultra long-span cable-stayed bridge of 1018m main span. To enable construction of this magnificent structure, very substantial and complex engineering inputs were required.

Figure 1 - Stonecutters Bridge

The contract for the construction of Stonecutters Bridge was awarded to Maeda-Hitachi-Yokogawa-Hsin Chong Joint Venture (JV) in April 2004. AECOM assisted the JV in the tender preparation and, upon JV's success in securing the construction contract, AECOM was appointed by JV as consultant for the comprehensive construction engineering services in the construction phase.

Very substantial and complex engineering inputs were involved in enabling the construction of this ultra long-span structure. These construction engineering activities included erection analysis, bridge geometry monitoring, control and adjustment, bridge aerodynamics, wind tunnel testing, vibration mitigation measures, falsework for the construction of the concrete back spans of the cable-stayed bridge (60m-high falsework systems including precast segmental concrete tower and steel trusses in longitudinal and transverse planes), cofferdam design, development of deck lifting procedures, navigation simulation, marine traffic management, marine jetty design, temporary traffic management, geotechnical engineering, amongst many other strands of work. This paper focuses on selected aspects of our work.

2.1 ERECTION ANALYSIS AND BRIDGE GEOMETRY CONTROL

AECOM developed bridge geometry control strategies and methods for use in a prediction, survey, re-analysis and possible adjustment cycle in the bridge erection (Figure 2). The overall objectives were to ensure that the final, target geometry of the completed structure was achieved without unacceptable locked-in stresses and to ensure the structure has adequate strength and performance at all construction stages.

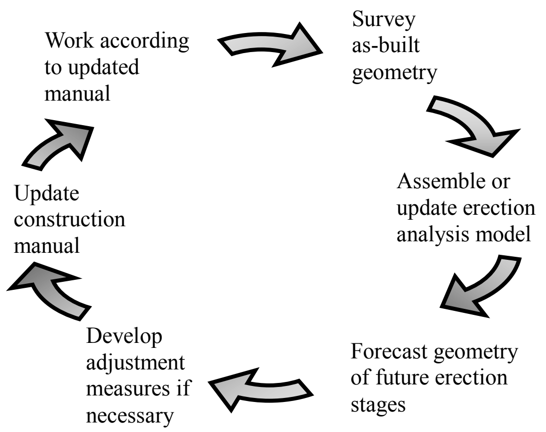

Figure 2 - Bridge Geometry Control Cycle at Each Erection Stage

The framework for bridge geometry control consisted of a co-ordinated set of activities in the construction planning phase, the fabrication phase and the erection phase. All activities in these three phases were robustly integrated to support the prediction, survey, re-analysis and possible adjustment cycle. Erection analyses underpinned many of the activities in the framework and therefore they are described in detail in the following sections.

The erection analyses of the entire bridge structure produced data for geometry control, and identified any special design requirements for temporary works including falsework, temporary stays or propping, and dynamic stabilization measures such as damping devices. The main outputs from the erection analyses were unstressed lengths of compression and tension members, pre-camber of flexural members, design loads on temporary works, natural frequencies and mode shapes at intermediate construction stages, and the structural effects of wind loading in the partially erected bridge structure at the erection stages.

The primary objectives of the global stage-by-stage erection analysis are described below, working through the prediction, survey, re-analysis and possible adjustment cycle:

- to establish the stay cable unstressed lengths and structural element prestrains (precambers) required to achieve the target geometry at the end of construction
- to establish the stay cable jacking forces
- to predict the displacements of the structure at each erection stage for inclusion in the Construction Manual so as to enable on-site geometry control
- to determine the state of stress in the structure at each erection stage and verify structural adequacy
- to use as-built survey data to track the changes in geometry of the bridge and to forecast the geometries in the future erection stages
- to identify any corrective actions required to ensure the target bridge geometry is achieved.

The principal aspects of the bridge construction investigated in erection analysis and bridge geometry control included:

- Tower and deck cantilever erection cycles
- Alignment control of tower
- Stress resultants in concrete back spans in the final state
- Effects of displacements of falsework and temporary foundations on the shape and stress state of the back spans; and on the final profile of the main span and towers
- Effects of creep and shrinkage
- Effects of temperature and wind on the bridge geometry in order to correctly interpret as-built survey data
- Devising of typhoon procedures to ensure structures integrity
- Sensitivity analyses on tolerances in input parameters
- Design of back span temporary works
- Design of temporary post-tensioning to concrete cross girders

- Main span closure.

2.2 SEGMENT ASSEMBLY AND DECK LIFTING

The steel deck plate panels were fabricated in North Eastern China and then they were transported to an assembly yard in Southern China, where the plate panels were assembled into bridge deck segments.

Extensive studies were conducted to investigate

- the effects on the segments during transport
- the effects due to potential settlement in the storage yard
- different conditions at the trial assembly

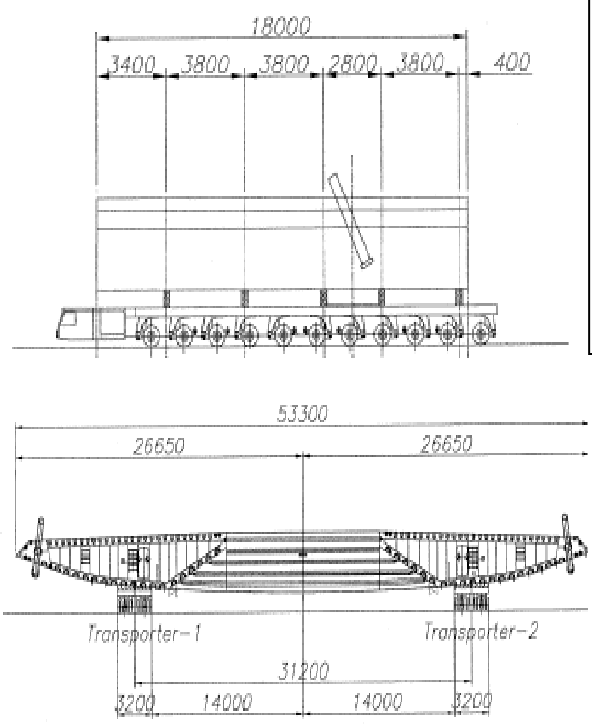

Figure 3 – Segment Support Configuration on Transporters

Detailed finite element analyses were conducted to investigate the feasibility of transporting the segments on a possible form of transporter as depicted in Figure 3.

The effect on the cross girder, arising from differential longitudinal movements of transporter-1 and transporter-2 was examined. The results demonstrate that the cross girder was capable of resisting the twist induced by the movements in the transporters.

Detailed finite element modeling has been used to examine the conditions of segment storage, through 16 number trestles supported on rigid foundations (Figure 4 and Figure 5).

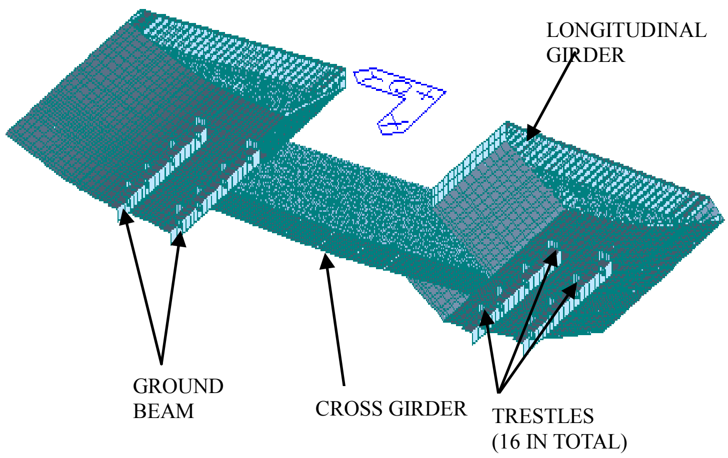

Figure 4 – Model of Segment with Support Conditions in Storage Yard

Two sets of loading conditions were imposed in the model separately to determine the critical stress state of the segment during storage.

The first condition allows loading for the potential imperfection in supporting levels of trestles, which may be caused by vertical misalignment of supporting wedges and/or un-uniformity of timber material properties. A total of 16 different load cases were analysed, allowing both for the over-length and the shortening of each individual trestle (Figure 5).

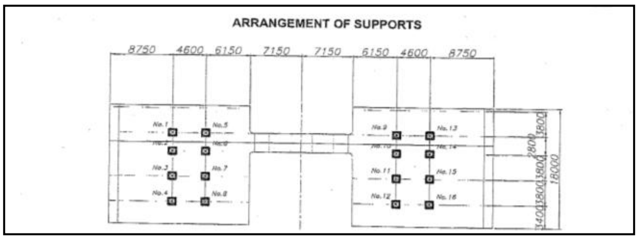

Figure 5. Plan Configuration of Segment Supported on 16 Trestles

The second loading condition allows for the potential differential settlement of the ground beams, which may be caused by any non-uniformity in soil properties.

A total of 5 load cases were analysed. Each case represented the differential settlement of 10mm at each ground beam supporting a line of 4 trestles. The details are shown in Figure 7.

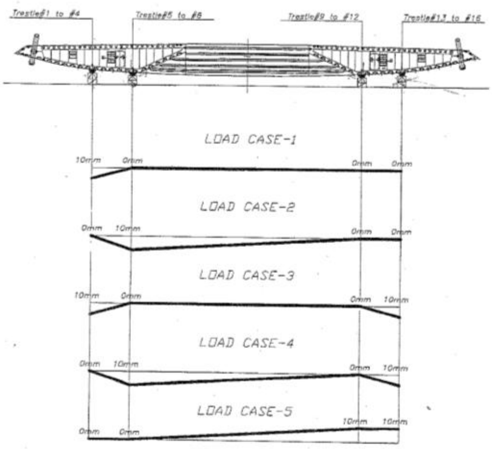

Figure 6. Differential Settlement of Ground Beams

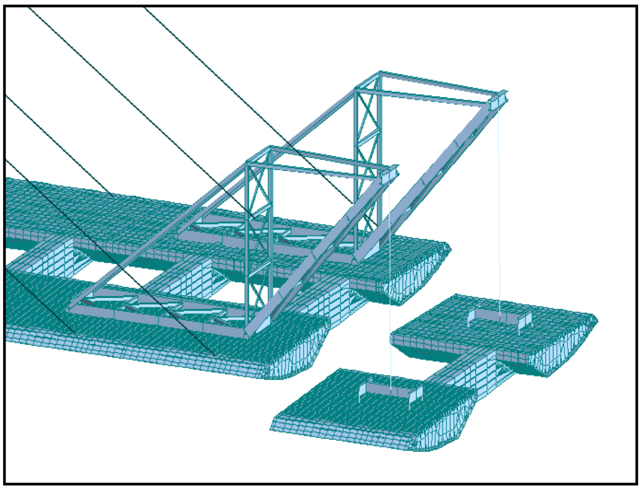

Figure 7. Finite Element Modelling of Segment Compatibility

Some local stiffening of the welded connections was necessary, to be implemented in conjunction with rigorous site survey to ensure the trestles were at the correct levels prior to the placing of steel segments.
Trial assembly of the deck segments at the assembly was required and the theoretical support points would be at the locations of stay cable anchor tubes. For practical trial conditions, investigations were made on the feasibility of providing supports at the intersection points between the line of the anchor tube webs and the segment intermediate diaphragms. Finite element analyses demonstrated that the proposed practical support conditions were feasible and no stiffening of the trial segments was necessary.

To assist in the planning of the trial assembly, further investigations were also conducted to determine the deformations of the trial segments under unit-loads for support conditions along the deck edges and for discrete point-support conditions during sea transportation.

Extensive studies of the deck lifting procedures were conducted, including marine traffic management, deck lifting operations in typhoon conditions and geometric compatibility in a typical erection cycle.

In a typical erection cycle, the construction front was supported by stay cables near the edges of the bridge and it deformed primarily under the action of deck girder self-weight and the weight of the deck lifting gantry. The lift-in segment was suspended from the deck lifting gantries at support points positioned near the centres of the longitudinal box sections.

The differences in support conditions of the erected deck and those of the lift-in segment were such that their vertical deflection profiles would have a certain degree of mismatch. This phenomenon was studied in detail by finite element modeling (Figure 7). The mismatch was rectified by applying temporary external prestress. The prestress took the form of a bowstring above the lift-in segment, which induced deflections in the cross girder and thus caused the new segment to deform in the same manner as the segment on the construction front, and thereby achieving geometric compatibility.

2.3 BACKSPAN FALSEWORK AND DECK CONSTRUCTION

Already at the time of tender preparation the erection of the backspan concrete decks was identified as one of the most significant challenges in building Stonecutters Bridge construction. The difficult configuration and structural detailing of the deck led to a falsework scheme emerging as the only viable option. It was one of the most substantial ground-supported falsework systems ever erected, keeping approximately 30,000 tons of superstructure concrete (per backspan) supported at about 70m height.

The falsework system was developed by AECOM in close interaction with the JV to suit its construction methods and operational means. Safety, efficiency, and constructability were the priorities of the falsework scheme development. The final scheme is modular, has direct load paths and was highly efficient in the use of material.

The backspan concrete girders were constructed on a 60m-high falsework system which was one of the most substantial ground-supported systems erected to this height (Figure 8).

Within each span between permanent piers there was a 2-bay falsework structure, consisting of three pairs of temporary towers, braced with steel members and founded on bored piles. In the longitudinal direction, the falsework towers were positioned under the centerlines of the intermediate cross-girders in the deck.

The central portion of each intermediate cross-girder was cast on a birdcage falsework structure that was supported on steel cross-girder trusses that span transversely between the temporary towers. Under each intermediate cross-girder in the concrete deck there were 4 number steel trusses, each some 5.5m deep. The trusses were simply-supported at their ends on an arrangement of fabricated steel “crown beams”. The crown beams were positioned on the top of the temporary towers.

The end portions of each intermediate cross-girder were cast on a birdcage falsework structures that were supported on steel “wing trusses”. The wing trusses were 20m-deep triangular trusses that cantileverd from the temporary towers. The top of each wing truss was supported on the crown beams and the base of each wing truss was clamped to the temporary tower.

The temporary towers consisted of precast segmental concrete blocks with external dimensions of 2m x 2m. The majority of the segments were 2m high with a hollow core, and with 250mm-thick walls. At connection points with the steel bracing members, the segments were 1m high and were solid. The segments were match-cast with shear keys at joints. Vertical ducts in the walls of the segments accommodated unbonded high tensile bars that connect the segments together and continued for the full height of the towers. The temporary towers were braced in three orthogonal planes by diagonal steel bracing members.

The temporary towers were supported on single 1.8m diameter piles that are founded at depth, on bedrock. Each pile had a pilecap that supported the plinth at the base of each tower. Ground beams linked the pilecaps to distribute horizontal forces between the piles and provide rotational resistance to the tops of the piles and bases of the towers.

Figure 8 – Backspan Falsework

A detailed stage-by-stage finite element analysis model of the concrete backspans was created, which was used to track the changes in structural configuration and loads through the construction process and duly accounted for stiffness contributions from falsework and partially cast concrete members, changes in weight and stiffness during staged casting and the removal of falsework trusses as well as the subsequent installation of stay cables and the removal of the falsework tower supports. Furthermore, creep and shrinkage effects as well as the post-tensioning sequence were accurately represented. The complex concrete grillage deck was constructed on the ground-supported falsework. The cross girders were cast first and subsequently utilized to support the falsework for construction of the longitudinal girders. Transverse post-tensioning was applied at a number of intermediate stages after cross girder completion and during longitudinal girder construction. The permanent transverse tendons in the bottom chord needed to be augmented by temporary tendons at the top chord in the outer regions of the girders.

The deck was first completed in the centre part of the bay before it was then connected to the pier crossheads by "stitch" bays. Before casting these stitch pours the deck geometry was carefully checked. At this stage level corrections by jacking on the falsework towers would have been possible but this was found unnecessary because the settlement predictions were accurate; thereby achieving further economy and speed of construction.

2.4 SEGMENT ASSEMBLY AND DECK LIFTING

2.4.1 Steel Plate Fabrication

The steel deck segments for Stonecutters Bridge were formed from thermo-mechanically controlled process steel, Grade S420M/ML in accordance with BS EN 10113. The plates required a very accurate control process during heating, rolling and water cooling. This grade of steel was becoming increasingly common in Europe, but was still relatively unusual in the Far East. Consequently sourcing of the material was difficult and was eventually procured from a number of sources in Europe and Japan.

The total weight of deck steel was 33,200 tonnes with a typical segment weighing 500 tonnes.

Deck plates were fabricated at the workshops of China Railway Shanhaiguan Bridge Group (CRSBG) in Shanhaiguan Northern China, an advanced facility which had been used on many of the major bridges constructed in China in recent years. At this facility the steel plates were blasted, primed and cut. The edges of the plates and associated stiffeners were then bevelled and the stiffeners were welded to the deck plates. A typical deck plate weighed 15 tonnes.

Figure 9 – Diaphragm Cutting at CRSBG

For all welding at the fabrication yard CO2 (gas shield) arc welding was selected. This form of welding was adopted for all positions (flat, vertical, horizontal and overhead) and allowed for a continuous welding process. Figure 10 shows the welding of stiffened panels.

From a geometry control viewpoint the most critical components were the stay cable anchor tubes which had to be fixed to the segment to an accuracy of 0.1°.

After completion, the deck plates were then transported to the nearby port for transportation to the next stage in the fabrication process, assembly at Dongguan in the Pearl River Delta.

Figure 10 – Welding of Stiffened Panel

2.4.2 Deck Segment Assembly

There were 65 deck segments, each comprising of about 200 components.

Assembly of the deck segments took place on two production lines, which are each capable of working on seven or eight segments at one time. The production lines included a moveable shelter which enabled assembly to remain in the shade and out of the rain (see figure 11). The assembly of deck segments operated on a 60-day cycle for each production run.

The deck plates were unloaded at a purpose-built jetty and then prepared in a pre-assembly area.

Figure 11 – Deck Segment Assembly

2.4.2.1 Match Fabrication and Geometry Control

The segments were assembled bottom upwards on special trestles which were capable of limited adjustment by means of hydraulic jacks. The segments were assembled in runs of seven or eight segments, with each segment matched to the adjacent segments to ensure a close fit when finally erected on site in Hong Kong. The deck segments were carefully assembled with continual checks on alignment and elevation.

When the first three segments of a production run were almost complete a trial assembly was carried out. In the trial assembly the segments were checked for alignment, elevation, segment dimensions and plate flatness. If acceptable, the excess top and bottom plate material (green) was cut from the segment, leaving the pre-defined weld gap between segments. Having been matched, adjacent segments were temporarily connected together by bolted splice plates (keeper plates) which were then used later during erection to ensure the relative geometry on-site was the same as the trial assembly geometry at the assembly yard.

Each typical segment comprised the two longitudinal girders connected by a cross girder. In the case of the back span segments these components were not welded together at the assembly yard, but main span segments left the yard complete.

2.4.2.2 Blasting and Painting

The existing facilities for blasting and painting were significantly enhanced at the assembly yard. All internal and external surfaces were blasted to Sa 2.5 in accordance with ISO8501-1. The external surfaces were then treated by a traditional coating process of epoxy rich primer, two coats of epoxy MIO and acrylic polyurethane topcoat. The internal surfaces, which were later subject to dehumidification when the bridge is in service, were provided with just a coating of 50 microns of epoxy zinc phosphate primer. On completion the segment was silver grey with a semi-gloss finish.

2.4.2.3 Transportation

The painted segments are then transported to the storage area by means two multi-wheel transporters, one for each longitudinal girder. The transporters have synchronised control to ensure the segment is not subject to excessive differential distortion. AECOM conducted detailed finite element analyses to investigate the feasibility of transporting the segments on transporters. The effect on the cross girder, arising from differential longitudinal movements of the two-halves of the transporter system was examined. The results demonstrated that the cross girder was capable of resisting the twist induced by the movements in the transporters. Detailed finite element modeling was also carried out by AECOM to examine the conditions at segment storage. The results led to local stiffening of the welded connections being necessary to be implemented in conjunction with rigorous site survey to ensure the support trestles in the storage yard were at the correct levels prior to the placing of steel segments.

From storage the segments were moved by the transporters to the dynamically positioned (DP) barge which was grounded at the jetty for easy loading. The segment was then shipped to Hong Kong for erection.

2.5 HEAVY LIFT FOR STEEL DECK ERECTION

The steel segments to be erected in the vicinity of the bridge towers, required a different method of erection – they had to be erected over land as it was not possible to lift the segments directly from the barge. A very substantial ground-supported falsework system was originally envisaged, whereby the segments would be lifted from the barge to final deck level. The segments would then be slid along rails at high level to final position and then welded together. Such a scheme would have significant impacts on cost and programme. JV therefore reviewed alternative schemes and subsequently formed an Alliance with VSL to develop a heavy lift scheme in order to achieve cost and programme advantages. AECOM worked closely with JV-VSL Alliance in the development of the heavy lift scheme and was responsible for the permanent work verification, assessment of adequacy of the backspan falsework and geometry control analysis for the entire heavy lift planning and implementation.

2.5.1 Assembly of the Longitudinal Girders

The first stage was the construction of a gravity wall jetty, designed by AECOM adjacent to each tower. This required the removal of about 100m of seawall on each side of Rambler Channel and the placement of large precast concrete blocks to form the jetty. At the same time an unloading frame was erected, cantilevering from the tower, with temporary stay cables acting as support.

The steel deck segments were lifted from the barge by the unloading frame and lowered onto carts. The carts were slid along a series of rails by hydraulic jacks until the segments were positioned at ground level below their final position. Once the alignment and elevation relative to each other had been confirmed the segments were welded together to form the two 88m-long longitudinal girder units. The cross girders, at this stage, were placed in storage.

2.5.2 Lifting and Sliding the Deck

Once the welding of the segments was complete, the 88m-long longitudinal girder units were lifted using strand jacks mounted on a bracket attached to the tower and a deck lifting frame cantilevering from the concrete deck. The two longitudinal girders were lifted simultaneously to ensure balance of load between the brackets on the tower. The overall weight of the lift was 4,000 tonnes, with a load distribution between tower brackets and deck lifting frame of about 80:20.

Figure 12 shows the basic arrangement for the heavy lift. Guides were attached to the tower and backspan concrete deck falsework system to restrict lateral movement during the lift. Initially the deck was raised to about 50m above ground level in a series of 0.5m strokes of the jacks. Due to the tapering tower form, the longitudinal girder units had, at that stage, to be jacked laterally by 4m towards one another.

The lift then continued till the longitudinal girder units were at their final elevation of about 75m above ground level. At this level the longitudinal units were jacked again laterally by 2m inwards and then finally 2m longitudinally towards the concrete back span deck, leaving a 2m gap. The decks were secured by ties and props to the concrete deck and by temporary bearings to the tower.

Figure 12 – Heavy Lift Operation

2.5.3 Completion of the Heavy Lift Operation

Following the lift of the longitudinal girder units extensive surveys were undertaken and the position of the decks was fine tuned. The cross-girders were lifted, again by strand jacks. Once all five cross girders had been lifted and the geometry was confirmed the longitudinal and transverse girder units were welded together, while the concrete stitch was cast and then stressed between the steel deck and the concrete deck. With the cross-girders welded and the stitch complete, installation of the permanent stay cables then commenced.

The heavy lift operation was carried out two times, for the East and the West of Rambler Channel. On both occasions the lifting and sliding operation was completed successfully within two days. With the heavy lift complete, the focus changed to main span deck erection.

Throughout the planning and the execution of the heavy lift scheme, AECOM conducted rigorous analyses, carried out structural verification of the permanent works, determined necessary strengthening measures, developed detailed geometry control procedures and implemented on-the-day back-analysis and control.

2.6 MAIN SPAN DECK SEGMENT ERECTION

2.6.1 Marine Considerations

One of the main constraints to the construction of Stonecutters Bridge was Rambler Channel and the need to maintain the flow of shipping through the channel unhindered by the construction of the bridge. Consequently AECOM assisted JV in developing a number of measures to maintain and control marine traffic during deck-lift. Firstly a dynamically positioned (DP) barge was used to transport the segment to the lifting location. By reference to GPS satellites the barge was able to automatically position itself and then hold position by means of thrusters, located at the four corners of the barge. Under the terms of the Contract no anchors could be used to secure the barge at the lift location. The second measure was to carry out the lift in as short a period as possible. This was achieved by using winches as opposed to strand jacks giving a lift time of about 40 minutes.

While practical measures were taken in terms of equipment, a number of studies were also undertaken to investigate how to minimise disruption to the port. Current measurements were taken in Rambler Channel, from which a current atlas was prepared. From this atlas the current in Rambler Channel was predicted for every deck lift operation. In association with the Hong Kong Pilots Association, simulations were carried out of ship movements through the channel during various critical deck lifting operations. This exercise helped prove that shipping movements did not need to be halted during a deck lift and also acted as a familiarisation exercise to the pilots.

2.6.2 Lifting Operations

Figure 13 shows a typical deck lift operation. The DP barge, accompanied by four guard boats, moved out to the lift location. The lifting gear was lowered to the barge below where it was attached to the segment lifting lugs. Once secured the lifting frames started to take the load until the segment lifts off. At this point wooden wedges beneath the segment were immediately removed in order to avoid rebound and potential resonance between the barge motion and the deck cantilever. The segment was lifted smoothly till it was level with the end of the deck cantilever, where it was secured and the barge and guard boats could be released.

Figure 13 – Deck Lift Operation

The site connection of a segment in a wide, flexible deck to the tip of a wide deck cantilever under the actions of deck lifting gantries was investigated by AECOM. The difference in support conditions of the lift-in segment and those of the erected deck was such that their vertical deflection profiles would have a certain degree of mismatch. AECOM's work concluded that connecting the lifted segment to the end of the cantilever required special measures as the cantilever end was deformed by the load from the main span lifting gantries. It was therefore necessary to deform the lifted segment in a similar manner and this was achieved by applying a bowstring prestress system. Figure 14 shows the arrangement of the external prestressing system designed by AECOM, respectively in the trial jacking and in the on-site operation. The system included posts on each side of the cross girder, two steel sections connecting the posts and diagonal prestressing bars fixed to the deck plate. Loads were applied by means of hydraulic jacks at the base of the posts, inducing a transverse deformation on the segment. The load applied to the jack was adjusted until the deck plates of the lifted segment matched with the cantilever end. Measurements were also taken across the joint to ensure the weld gap was consistent with the measurements taken at the assembly yard. If the geometry of the lifted segment was satisfactory the keeper plates were fitted, for welding to proceed.

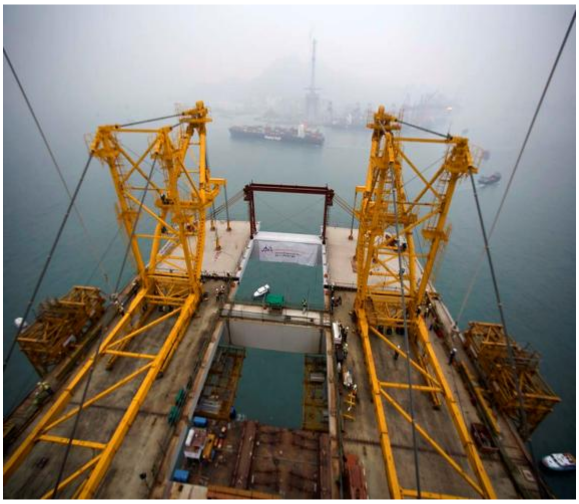

Figure 14 – Main Span Segment Mismatch – On-Site Jacking

While the welding of the lifted segment was being carried out the back span stay cable was installed and stressed. JV and AECOM investigated different welding processes in search for an optimum solution. The welding process took place in a fixed sequence with the perimeter welded first. The stiffeners across the erection joint were then welded, the bowstring prestress system removed and installation of the main span stay cable carried out.

2.7 WIND TUNNEL TESTING

AECOM advised the JV in bridge aerodynamics and completed the planning, management and supervision of the wind tunnel investigations for Stonecutters Bridge construction.

Comprehensive wind tunnel investigations were commissioned to cover conditions arising during erection, including the representation of temporary works and construction plant/equipment wherever relevant. Tests were also commissioned on the stay cables, including textured sheathing as a countermeasure to rain-wind induced excitation.

The section model testing verified the stability of the bridge against divergent amplitude response during construction, and the efficacy of the guide vanes in mitigating vortex shedding response at the erection stages. The guide vanes therefore are installed prior to deck lifting to assist in suppression of vortex shedding during deck cantilever construction. The dynamic tests on the section model also demonstrated the significance of the vertical and torsional aerodynamic damping compared with intrinsic structural damping, and hence the dominance of aerodynamic damping in vertical and torsional buffeting response. The static wind loading measurements on the section model identified the significance of the temporary handrail system on the overall drag of the bridge deck. A number of additional tests were therefore conducted to develop a form and configuration of temporary safety hand rail system to limit the lateral wind forces on the structure during construction to acceptable levels.

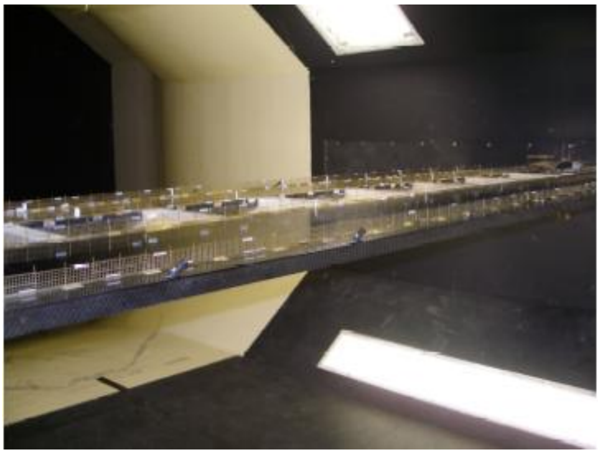

Figure 15 – Stonecutters Bridge Section Model in the Wind Tunnel

The aeroelastic tower model wind tunnel investigations verified the aerodynamic performance and structural integrity of the tower during the erection stages including the full-height freestanding conditions. Damping was effective in mitigating vortex shedding response and such response was in general reduced by the presence of construction plant and equipment.

In the aeroelastic bridge model testing, the vertical responses recorded were generally less than the "a-priori" analysis based on the buffeting response observed in the section model testing. The buffeting responses recorded in the aeroelastic bridge model testing also corroborated the observations made in the section model tests, in that the vertical and torsional responses were dominated by aerodynamic damping. The very large aerodynamic damping of vertical motion, up to 0.6 log dec at design wind speeds, meant that relatively little benefit would be obtained through additional (mechanical) damping. Only a 10% reduction of the resonant contribution to response in the first vertical model was obtained by adding damping up to 0.15 log dec. These findings meant that the application of damping devices to mitigate buffeting response in these modes would present significant challenges. The aerodynamic damping of lateral response was relatively modest, additional damping could significantly reduce the dynamic displacements. Reduction by up to 40% was obtained by increasing damping to 0.2 log dec. The effect of the free end of the cantilever appeared only modest, and introduction of the end of the adjacent deck cantilever in close proximately had little consistent effect on critical responses. The aeroelastic bridge model study was used to validate a comprehensive numerical model which was then used for further investigations of the buffeting response of the bridge structure in different erection scenarios. It was found that buffeting effects posed a significant demand on the structure in the cantilever conditions.

The rain-wind induced oscillation tests on stay cables investigated the effects of dimpled pattern on the cable sheathing and of increased damping on the dynamic behaviour of the stay cables. The dimpled pattern suppressed or alleviated rain-wind induced vibrations. Increased damping was also effective in suppressing such oscillations. The drag coefficients measured on cables with dimpled surface texture were within the permissible design values.

Figure 16 – Stonecutters Bridge Aeroelastic Bridge Model in the Wind Tunnel

3. SUTONG BRIDGE

The 1,088m main span Sutong Bridge in China was the world longest cable-stayed bridge upon its completion. The Sutong Bridge project was masterminded and directed by China Jiangsu Province Construction Commanding Department, which has a record of success in projects such as Jiangyin Yantze River Highway Bridge, Runyang Yangtze River Highway Bridge and now Sutong Yantze River Highway Bridge. Contract C3 for the construction of Sutong Bridge was awarded in early 2005 to Second Navigational Engineering Bureau, CCCC to whom AECOM was consultant. A fast-track construction programme was such that main span closure was completed in June 2007, and the bridge was opened to traffic in May 2008.

One of the most significant undertakings in the construction of the super long span Sutong cable-stayed bridge was construction control. The unique complexity of Sutong Bridge required specially developed methods and procedures to control bridge geometry and to ensure safety of the bridge during construction. The aerodynamic stability and performance of the bridge during construction were investigated by extensive wind tunnel investigations; and mitigation measures were developed and implemented as necessary. This paper describes selected aspects of the integrated techniques adopted for Sutong Bridge construction control, with illustrations of the robust principles and practices in analysis-survey-prediction-correction cycle.

Figure 17 – Night View of Sutong Bridge

Figure 18 – Sutong Bridge Main Span Erection

The new challenges posed by the construction of a record long span and a complex structure required specially developed methods and procedures to control bridge geometry and to ensure safety of the bridge during construction.

The framework for bridge geometry control consisted of a co-ordinated set of activities in the construction planning phase, the fabrication phase and the erection phase. All activities in these three phases were robustly integrated to support the prediction, survey, re-analysis and possible adjustment cycle. This paper focuses on the prediction, survey, re-analysis and possible adjustment cycle in the erection phase.

Figure 19 – Tower Shaft Construction

Figure 20 –Upper Tower Stay Anchor Box Installation

3.1 TOWER ERECTION CONTROL

The construction sequence was identical for both the North and the South Towers; each consisting of the following principal activities

- Construction of tower concrete elements
- Construction of tower steel elements
- Application of tower temporary supports
- Application of tower temporary loads
- Application of stay cable loads acting on tower

For the purpose of tower geometry control a total of 176 erection stages (key events in the tower construction activities) were judged to be of interest; the last stage being the application of superimposed dead load at the target or reference state. Each of these stages was modeled in the erection analysis.

In order to achieve the target geometry, all structural displacements that occurred during the construction stages were taken into account in determining geometrical adjustments for each erection step. The adjustments would consist of over-lengths, pre-camber and pre-set of the formwork, and they are described in detail below.

Over-lengths - as a result of axial shortening, creep and shrinkage effects, the concrete tower shortened during construction. In order to achieve the target geometry at the reference state, an axial over-length was specified for every concrete lift of the tower. The over-length values were determined from the stage-by-staged erection analysis.

Over-length values used for set-out calculation - for the set-out procedure, the elevation of the top of the concrete lift being set-out was specified as a design elevation plus the value of the over-length to compensate for the further displacement of the top joint due to additional loads.

Pre-camber – owing to the inclination of the lower tower and middle tower legs, the self-weight of the concrete induced deflections of the tower legs in the transverse direction of the bridge. This effect was compensated by pre-cambering the tower legs. Pre-camber values were determined from the stage-by-stage erection analysis. The values of pre-camber and over-length were used to calculate the intermediate expected geometry during the error assessment and correction procedures.

Formwork Pre-set - the deformations of each concrete lift was not only the deflections of the tower legs but also the deflections of the jump form scaffolding system. The transverse component of the self-weight of the wet concrete induced deflections in the form and thus the concrete lift geometry follows the deflections of the jump form system. It was therefore necessary to compensate for this effect by pre-setting the jump form system.

Temporary props - the bending of the lower legs due to self-weight induced bending stresses, and these effects were mitigated by installation of transverse props (thrusts) between the tower legs. The props were activated by jacking.

Survey data processing – as-built displacements were measured at survey points and tower monitoring points. For the sections in the concrete tower shafts the as-built displacements at the top of concrete lift were surveyed upon completion of each concrete lift or after installation of the transverse prop. The expected intermediate geometry of the concrete section was specified at the centre of the section. For the stay anchor boxes, the survey points of the anchor boxes were at the outer corners of the box section. The centre of the box section was derived from corner survey data. The expected intermediate geometry of the anchor box section was specified at the centre of the section.

Corrective actions -

After calculating the difference between as-built survey and the expected intermediate values, the corrective measures were made in the set-out for the next cycle to aim at achieving the target geometry of the tower within the allowable tolerances.

3.2 DECK AND CABLE ERECTION CONTROL

The backspan steel segments were prefabricated in nine units and positioned by floating crane onto the permanent piers and temporary supports (Figure 21). The erection of the cantilevered section of the deck commenced after the erection of the three deck units at the tower location (Figure 22), their corresponding cable stressing, and the installation of the temporary fixity onto the tower crossbeam.

Figure 21 – Backspan Segment Erection

Figure 22 – Cantilevered Segment Erection

Prior to backspan closure (Figure 23) the installation cycle was to erect the segments at the ends of the double cantilever with the cables in their final positions. Subsequent to backspan closure the backspan segments were in position and single cantilever construction towards midspan progresses.

Figure 23 – Backspan Closure

Figure 25 – Segment Erection Cycle

Figure 24 – Main Span Closure

The installation sequence for a segment in the cantilevered construction involved lifting the segment from the barge, welding to the existing cantilevered deck, installing the cable stays and applying first-stage stressing. The lifting gantries were then moved to their forward lifting position, followed by the second-stage and final stressing of the stay cables.

Step-1 segment lifted from barge

Step-2 after segment adjusted, matched up and welded up, remove connection between segment and lifting gantries; first stage stressing of cables.

Step-3 move lifting gantries forward; second stage stressing of cables.

Step-4 prepare for lifting next segment.

The typical erection activities consisted of only first and second stage stressing. The segment was installed to the unstressed installation geometry which was defined by its position to the existing structure. The unstressed installation geometry (the geometry excluding the deformation from self weight) was to be the same geometry as had been finally accepted in the trial assembly. The repetition of the trial assembly geometry was achieved by connection of the fixing plates that have been welded after final acceptance. However the alignment of the newly installed segment was verified on site by survey.

Cable Installation Procedure - the stressing of the cable was controlled by the displacement of the socket nut to the prescribed position. This was the primary control parameter. The secondary parameters were cable force and the change of the geometry of the anchor points – displacement due to stressing. The cable force and deck displacement were recorded as a cross check in the geometry control.
Cable Adjustments - the cable length could be changed by adjustment of the bearing nuts. This would be necessary for corrections

- after the detailed erection analysis for fabrication and installation error and
- after analysis of as-built and as-fabricated positions of the cable anchor points

Bridge erection was constantly monitored and the future predicted using a 3D finite element model of the whole bridge. The erection analysis model correlated to the actual structure through one-to-one correspondence of model and bridge control points. The model provided the target position for each segment installation, together with unstressed lengths for each cable. The model was also used to perform predictive analysis from any particular erection stage, using the as-built geometry, to predict the final bridge geometry upon completion. This was the prediction, survey, re-analysis and possible adjustment cycle.

In terms of geometry control of the deck girder installation, the most significant control parameters were the deck unstressed geometry and the cable unstressed lengths. Prior to commencement of the deck segment installation, the cable lengths were updated through analyses based on crucial parameters including the as-built geometry of the tower, the as-fabricated unstressed geometry of the deck girder, and updated loads weight densities and stiffness parameters.

3.3 MAIN SPAN CLOSURE

It was 13:30 hours 9 June 2007 Beijing time, the main span central segment (closure segment JH) of Sutong Bridge was lifted from a barge (Jin Hong No. 1) in the Yangtze River. All the preparatory work had been meticulously planned and executed, in anticipation of the main span closure operation for the world’s longest cable-stayed bridge.

Just over 48 hours later, at around 15:00 hours, 11 June 2007, the welding of closure segment to the adjacent record-long deck cantilevers was completed; thereby forming the 8 km link across River Yangtze between the cities of Suzhou and Nantong.

The Sutong Bridge main span closure method was based on integrating the merits of traditional Chinese approach using “natural temperature closure” with a typical jacking-back of the two deck cantilevers. The motivation for the combined method was to draw on the strengths of each approach to ensure efficient closure control for Sutong Bridge. The jacking-apart of deck cantilevers to provide sufficient air gap for fitting the closure segment was achieved by means of stressing/relaxing the temporary longitudinal diagonal “tie-ropes” at the deck-tower junctions.

The pre-closure preparatory work consisted of a number of carefully co-ordinated activities. Final adjustment and survey were carried out in the night of 7 June 2007. Survey data was rigorously analyzed to ensure smoothness of local geometry including rotational and translational fits, as well as to determine the final length of the closure segment.

The closure segment, which was precisely cut according to the instruction from the construction control team, was transported to site in the early morning of 9 June 2007.

The preparation work for the lifting operation was carried out from 10:00 hrs to 13:00 hrs of 9 June 2007. At 13:30 hrs, the closure segment was lifted by four deck gantries with four strand jacks. It reached a position immediately below the two cantilever decks at around 15:00 hrs. The precise insertion of Segment JH into the closure gap commenced at 18:00 hrs when the temperature started to drop below 30 degrees C. Following insertion of Segment JH into the closure gap, fine adjustments to fit-up the closure segment were carried out; involving also the progressive launching back of the deck cantilevers towards the closure piece. Welding of perimeter plates commenced at midnight and was completed by 06:00 hrs of the next day, together with the release of temporary fixities. The remainder of the time, until completion in the afternoon of 11 June 2007, was taken up by welding of the longitudinal plate stiffeners. The historic closure of the main span of the world's longest cable-stayed bridge was thereby successfully completed.

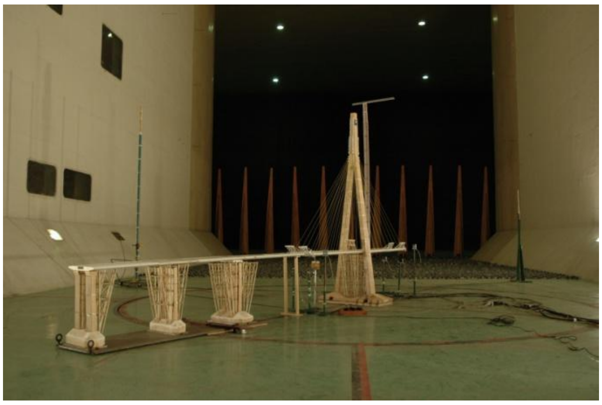

Figure 26 – Sutong Bridge Aeroelastic Bridge Model Wind Tunnel Testing

Figure 27 – Sutong Bridge Section Model Wind Tunnel Testing

4. CONCLUSION

The quest for building bridges of increasingly long spans stems both from necessity and from a sense of achievement in harnessing the forces of nature for the benefit of mankind. In many forms of long span bridges, the partial structures during the erection stages are vulnerable to the actions of construction loads and forces of nature. The safe construction of the partial structures through different structural systems to completion of the bridge, challenge the very best in the engineering sciences. In pushing the frontiers of large bridge construction, we have also gained extensive experience in resolving the complex construction engineering involved. Construction engineering for an ultra long span cable-stayed bridge is one of the keys to the success of the Stonecutters Bridge and Sutong Bridge projects. The record long span and the flexible nature of the bridges demand robust, versatile and simple to implement methods for geometry control and for ensuring safety of the structure throughout the erection stages. This paper provides a first-hand account of our efforts in pushing the frontiers of large bridge construction.