Shin Chon Bridge, Korea

Uwe Starossek, Prof., P.E., Hamburg University of Technology, Hamburg, Germany. Contact: starossek@tuhh.de

Summary

Shin Chon Bridge consists of two viaducts of 1060 m length each and a total height of about 100 m. The superstructures are post-tensioned concrete box girders that were built by the balanced cantilever method and have a main span length of 170 m. The viaducts were designed as semi-integral structures with columns of high slenderness. Nonlinear structural analysis was performed to confirm the suitability of the structural system. The beauty of the bridge is produced by slenderness, simplicity of form, and good proportions.

Keywords: Post-tensioned concrete; box girder bridge; balanced cantilever method; slender piers; flexible bridge system; semi-integral structure; nonlinear structural analysis; crack width calculation.

Introduction

The new Iksan–Jangsu link through the mountainous terrain of South Korea's Jeollabuk-do province adds 61 km to the fast-growing Korean expressway network of currently well over 3000 km. Shin Chon Bridge is a part of that new link. It carries the expressway at about 100-m height over a valley near Shin Chon village, being immediately preceded and followed by tunnels (*Fig. 1*). The bridge consists of two independent viaducts for westbound and eastbound traffic along different alignments, which run roughly parallel at a center-to-center distance of 50 m (*Fig. 2*). The superstructures are post-tensioned concrete box girders built by the balanced cantilever method with cast-in-place segments. All piers are monolithically connected to the girders, and expansion joints are placed only at the abutments, which leads to semi-integral structures. These features necessitated a particular prestressing measure (i.e. longitudinal jacking of superstructures before casting the last key segments) and nonlinear structural analysis to verify the global stability of the flexible bridge systems and to determine the sectional forces and crack widths in the piers.

Fig. 1: Shin Chon Bridge in front of Man Deok San peak

Fig. 2: Shin Chon Bridge shortly before completion

Superstructure

Longitudinal Design

The total length between abutments of either viaduct is 1060 m. A main span length of 170 m was chosen to satisfy both economic and aesthetical demands. This represents the Korean span length record for girder bridges. The side span lengths were determined such that the piers of both viaducts are transversely aligned to minimize the visual obstruction of the valley. The span arrangements thus chosen are $93 + 5 \times 170 + 117$ m for the westbound viaduct and $113 + 5 \times 170 + 97$ m for the eastbound viaduct (*Fig. 3*). The two superstructures were designed as fully prestressed post-tensioned concrete box girders.

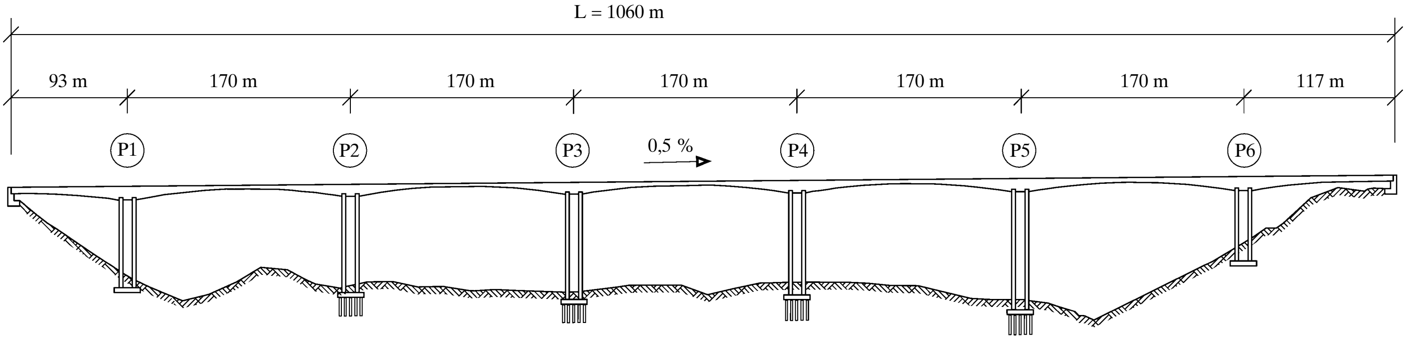

Fig. 3: General elevation

The girder depth is variable with a maximum of 10,0 m at the pier faces and a minimum of 3,4 m at the center of the main spans and at the abutments. The resulting span-to-depth ratios are 17 and 50, respectively. The girder depth varies according to a cubic parabola. The parameters of this curve were chosen such that, according to a preliminary and simplified analysis,[1] the shear flow in the webs is approximately constant, and the tensile force in the top slab varies linearly, over a major part of the span length. By fulfilling these criteria, economy in terms of concrete and reinforcement quantities is furthered. The subsequent detailed analysis of longitudinal moments and shear forces confirmed the usefulness of the simplified analysis method employed for choosing an optimum girder depth curve. Compared to a quadratic parabola, the chosen cubic parabola provides about 200 mm additional construction depth around the main span quarter points.

The longitudinal prestressing consists of 2×22 cantilever tendons 19-0,6" for the westbound viaduct (*Fig. 4*) and 2×26 cantilever tendons 19-0,6" for the eastbound viaduct. These tendons are placed and anchored in the top slab and in the top part of the webs. As cantilever construction proceeded toward midspan, at least two tendons were anchored at the end of each cast-in-place segment. After the cantilever tips were joined through closure pours (key segment), midspan tendons were installed and stressed. They consist of 2×8 (westbound) or 2×9 (eastbound) tendons 22-0,6". These tendons are placed in the bottom slab near the webs and are anchored in blisters. Additional bottom slab tendons are placed in the end parts of the side spans. Provision was made for the future placement of additional cantilever and bottom slab tendons, as well as external continuity tendons, should the need arise.

For economy in construction and maintenance and to improve durability, the two viaducts were designed as semi-integral structures. The superstructures are continuous over their entire lengths and expansion joints are placed only at the abutments. Furthermore, the superstructures are cast monolithically to all piers thus limiting the need for bearings to the abutments. The bearings at both abutments are movable in longitudinal direction. The seating lengths were chosen large enough to safely accommodate the maximum expected longitudinal displacements during an earthquake.

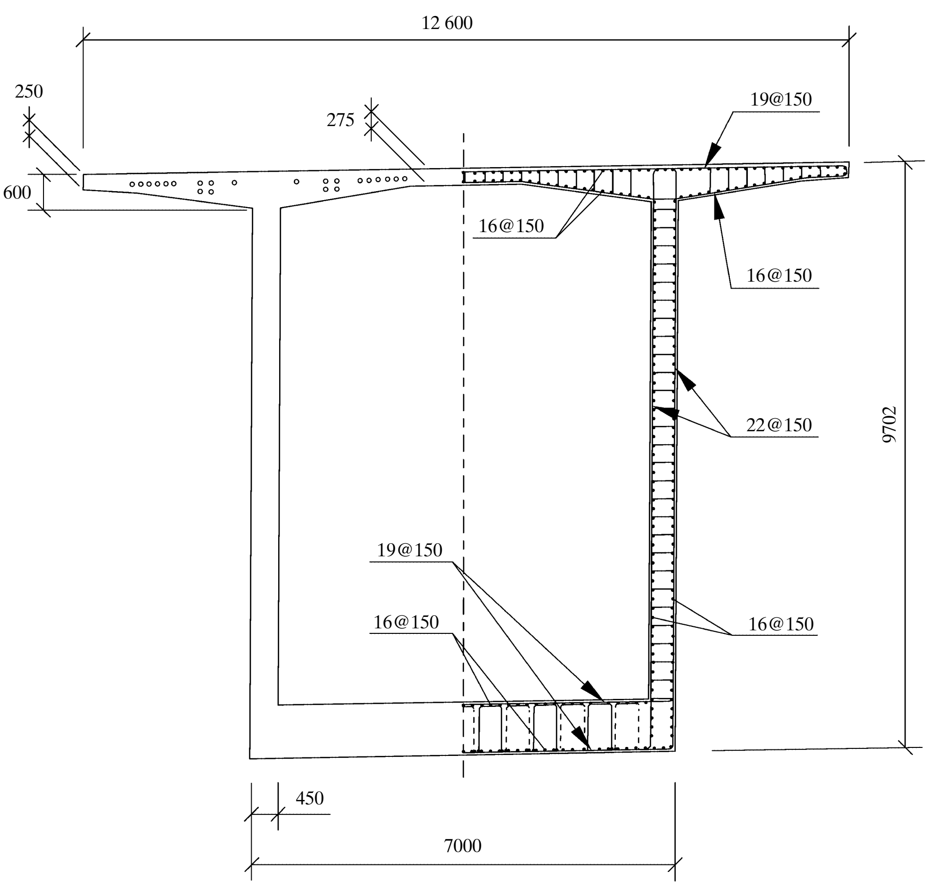

Fig. 4: Cross section of westbound superstructure at 2,0 m from pier face with post-tensioning layout (left) and reinforcing arrangement (right) Units: mm

Transverse Design

Both superstructures are made up of monocellular hollow boxes with cantilever deck slabs. The deck slab width is 12,6 m for the westbound viaduct, which carries two lanes of traffic (*Fig. 4*), and 14,7 m for the eastbound viaduct, which carries three lanes of traffic. The webs are vertical. The outer width of the box and the bottom slab is 7,0 m for the westbound viaduct and 8,0 m for the eastbound viaduct. The deck slab thus cantilevers out to either side by 2,8 m (westbound), or 3,35 m (eastbound), respectively. The top slabs of both viaducts follow the unidirectional transverse slope of 2,0% required by the road design. To facilitate formwork and reinforcement works, both webs were given the same depth. Consequently, the bottom slabs slope parallel to the top slabs and the outer contour of the boxes are parallelograms instead of rectangles.

The minimum deck slab thicknesses are 250 mm at the edge of the cantilever slabs and 275 mm between the webs, respectively. The deck slab haunches have a maximum depth of 600 mm at the web faces. This depth is not required structurally but it results from detailing considerations (longitudinal tendon alignment and anchoring). The deck slab is transversely post-tensioned for crack control. The thickness of the bottom slab varies linearly from 900 mm at the pier face to 250 mm at a section close to the quarter point. In the remaining midspan region of the main spans, a minimum thickness of 250 mm is maintained.

The web thickness adjacent to the piers and in the end parts of the side spans is 450 mm for the westbound viaduct and 500 mm for the eastbound viaduct. Apart from the end parts of the side spans, these widths were not dictated by strength requirements but by detailing considerations (alignment and anchorage of cantilever tendons). Beyond the sixth cantilever segment on each side of the piers (i.e. 23,6 m beyond the pier faces), the web thickness was reduced to the structurally required value of 350 mm. This reduction has a pronounced beneficial effect on the longitudinal bending moments due to dead load although it caused some extra effort during construction. For crack control, the webs are vertically post-tensioned with prestressing bars at selected locations around the third points of the main spans and in the end parts of the side spans.

Piers

Design Concept

Each viaduct is supported on its own set of piers with heights between 51,0 m and 88,4 m (the latter again being a national record). Each pier consists of a pair of slender parallel columns (twin legs). This results in pier flexibility sufficient to accommodate the longitudinal shortening of the superstructure due to creep, shrinkage, and temperature variation. Intermediary expansion joints in the superstructure could thus be avoided while at the same time strong bending resistance and stability during construction were provided. On the basis of nonlinear structural analysis, it was confirmed furthermore that the superstructure-to-pier connections can all be monolithic and the initially envisaged use of movable bearings between superstructure and outer piers (i.e. those next to the abutments, which are mostly affected by the shortening of the superstructure) can be dispensed with. The center-to-center spacing of the two columns forming one pier is 10,0 m. This value is somewhat smaller than the optimum spacing, in terms of structural performance and total costs, estimated at 12 m to 15 m. The smaller spacing was chosen to improve the appearance of the bridge.

Cross Section

On the basis of preliminary checks of global stability and bending moments, the pier columns were conceived as rectangular hollow reinforced concrete boxes with a width in longitudinal direction of 2,8 m (*Fig. 5*). This width aims at achieving an optimum in terms of constructability and structural performance during and after construction. It was confirmed by detailed analyses of global stability and bending moments that the resulting slenderness ratio $k\,l_u/r$ of the columns of up to about 90 is uncritical for both the construction stages and in service. Even a width smaller than 2,8 m would have been possible (at the cost of some increase in reinforcement for providing sufficient moment resistance and stiffness for critical construction stages). To assure constructability, however, it would have entailed a solid cross section and thus a larger quantity of concrete. For a width of 2,2 m or smaller, horizontal tie beams between the two columns of one pier would have been needed in some piers.

The pier and column width in transverse direction is 8,5 m for the westbound viaduct and 9,5 m for the eastbound viaduct, respectively. The box wall thicknesses of all columns are 500 mm for the long transverse walls and 900 mm for the short longitudinal walls. The column widths and wall thicknesses are kept constant over the height of the pier, which is deemed advantageous in terms of constructability. They exceed the corresponding widths of the superstructure boxes by 1,5 m. At the top, the columns visually extend beyond the soffit line of the superstructure by 2,3 m to offer a sense of the superstructure being safely cradled on the columns thus giving a novel architectural appearance to this structure (*Fig. 6*).

Accessibility

For sake of inspection and maintenance, access to the inner space of the column boxes should be provided. Such requirement had not normally been recognized and implemented in Korea until recently. To avoid a weakening of the highly stressed cross section at the bottom of the columns, access is provided through a vertical circular opening of 1,0 m diameter through the pier

Fig. 6: Superstructure-to-pier transition

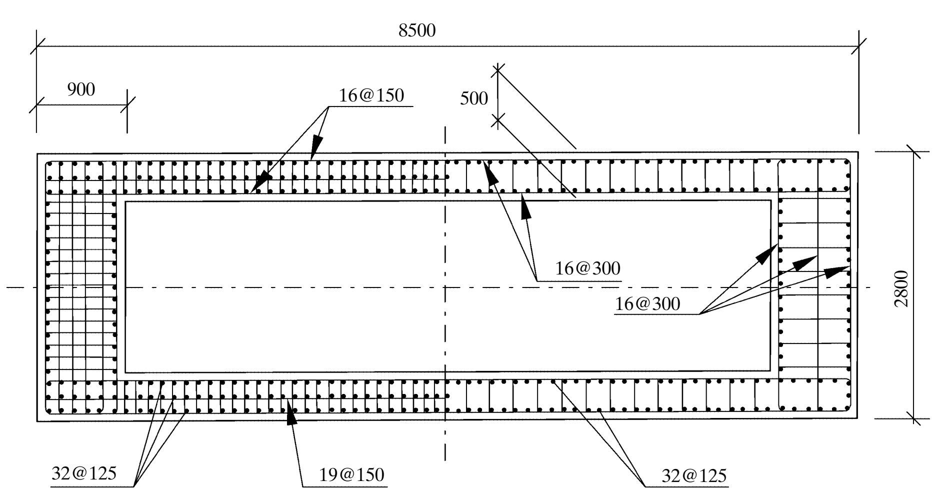

Fig. 5: Cross section of pier columns and reinforcing layout in highly stressed sections (left) and elsewhere (right) Units: mm

head diaphragm that is located on top of each column. The inner spaces of the columns can thus be accessed from the superstructure. It was decided during construction to close these openings not with manhole covers but with concrete plugs, which can be removed when necessary.

Reinforcement

The reinforcement consists of deformed reinforcing bars with a yield strength of 400 MPa. In longitudinal (i.e. vertical) direction, most sections are reinforced with 32-mm bars spaced at 125 mm along all inner and outer faces (32@125). This results in a reinforcement ratio of 2,3%. At top and bottom of the outer piers and at the bottom of piers higher than 70 m, one center layer 32@125 is added in each of the long transverse walls, which corresponds to a maximum reinforcement ratio of 3,2% (*Fig. 5*). In the center part of some of the inner piers, the minimum reinforcement ratio of 1% would have been structurally sufficient. For the sake of uniformity, however, this possible reduction of quantities was dispensed with. The concrete cover of the longitudinal reinforcement is 90 mm.

Longitudinal splices are staggered. Instead of lap splices, mechanical connections (couplers) were used. This reduces reinforcement congestion as well as the length and weight of individual reinforcing bars. It is considered an improvement mainly in terms of ease of construction (handling of reinforcing bars) and structural quality. The longitudinal pier reinforcement extends to the top of the pier heads to provide moment continuity between piers and superstructure.

The transverse reinforcement in most sections consists of 16-mm hoop and tie bars at a maximum transverse spacing of 375 mm and a vertical spacing between layers of 300 mm. The corresponding reinforcement ratio, in each direction, is about 0,3%. Over a length of about 12 m at the top and bottom of each pier, much heavier transverse reinforcement is provided for plastic hinge confinement according to seismic design requirements. In these regions, the transverse spacing of hoops and ties is only 125 mm and the vertical spacing between layers is 150 mm; 19-mm tie bars are used here. The corresponding reinforcement ratio, in each direction, is equal to or larger than the required value of 1,2%.

Pier Heads

Superstructure and pier columns intersect within the pier heads. Each pier head consists of a closed box formed by massive reinforced concrete panels, which are the two solid transverse diaphragms, one on top of each column, the two webs between these diaphragms, and the top and bottom slabs of the superstructure. The thickness of each transverse diaphragm is 2,8 m and thus corresponds to the outer width, in longitudinal direction, of the pier columns. Both mild and prestressed reinforcement is used to accomplish the transfer of forces between superstructure and piers. Access to the pier head chamber is provided by a longitudinal channel of 1,6 m × 1,2 m through each diaphragm. The vertical openings for providing access to the inner space of the columns start at the center of the longitudinal channels.

Structural Analysis

The sectional forces and deflections of the superstructures during and after construction and the camber were computed with a linear finite-element program (taking into account time-dependent deformations). For these purposes, linear analysis is deemed sufficient in view of the uncracked state of the fully prestressed superstructures. The partly cracked state of the pier cross sections will not greatly influence the superstructures. Stresses and deflections thus determined were checked against the corresponding limit values stipulated by the Korean design codes. This analysis, although extensive and complex, followed the procedure as used for similar structures. The further presentation, therefore, focuses on the analysis of the piers, which required new approaches and was particularly demanding, and the analysis of the pier heads.

Preliminary Analysis

Because of the absence of fixed bearings at the abutments, the structural system can be characterized as flexible.[1] Any longitudinal forces acting on the superstructure, during or after construction, are transferred to the ground through the piers. During preliminary design, the pier dimensions were checked against the criterion of global longitudinal stability of the flexible structural systems that evolve during and after construction. A simplified method was used in which the P-Δ effect is considered in a linear manner and external horizontal forces and initial deformations (imperfections) are neglected.[1]

Linear Code-Based Analysis

For the purpose of final design, the calculation of sectional forces was based on analysis with a linear finite-element program. The first-order bending moments in the pier columns thus obtained were multiplied with moment magnification factors to take into account the effect of slenderness. The procedure is described in the applicable Korean design code, which, in this regard, corresponds to the AASHTO Specifications.[2] Care was taken to use appropriate values of column bending stiffness to avoid overly conservative results for the longitudinal (weak) direction. At first, the determination of stiffness was based on an approximation formula as it is given in the design code. This implies the assumption that all columns are cracked. The first-order longitudinal column moments were multiplied with the moment magnification factors obtained in this way. When comparing these magnified moments with the decompression moments, it was found that some columns remain uncracked. The calculation of the moment magnification factor for sway, δ_s, was therefore repeated using the stiffness of the uncracked sections, adjusted for creep, for the uncracked columns. As expected, smaller magnified bending moments were obtained. Additional columns were found to remain uncracked in some cases and the process was repeated accordingly.

Nonlinear Analysis

While this procedure is deemed transparent and safe, it lacked formal recognition or mentioning in applicable codes. A computer program was therefore developed to independently and more accurately verify the global stability of the flexible bridge systems and analyze the longitudinal bending moments in the columns for the various structural systems (during and after construction) and selected critical load combinations.

The refined and nonlinear structural analysis took into account nonlinear material behavior, concrete cracking, time-dependent deformations, and the P-Δ effect. It is based on the method for calculating the ultimate load of a flexible bridge system presented in

Ref. [1]. The axial column forces and the longitudinal displacements at the top of the columns obtained from linear analysis were used as input data. The underlying assumption that all columns can be considered fully restrained against rotation at top and bottom was maintained. This simplification was deemed admissible in view of the large superstructure-to-column construction depth and stiffness ratios. Other simplifications introduced in Ref. [1] were not adopted though and the theory was generalized accordingly.[3] For instance, differential longitudinal displacements between neighboring piers due to axial deformations of the superstructure were considered. Furthermore, short-term actions were treated separately from long-term actions to properly assign the column stiffness which applies to each case. Each column was subdivided into a number of segments. The column stiffness at each section was determined by nonlinear sectional analysis. Because this stiffness depends on the sectional forces, the output values of the bending moments were fed back as input values of subsequent iteration cycles until input and output data were in agreement.

The computation of sectional capacities and the design checks were based on the strength design method (load factor design) as described in the applicable Korean design code, which, also in this respect, is similar to Ref. [2].

Crack Width Calculation

Crack width calculations were performed for the critical locations, i.e. at top and bottom of the outer piers. The load combination used for this purpose comprised 100% of permanent loads plus 50% of nonpermanent loads (all unfactored). The effect of the ensuing sectional forces, obtained from the nonlinear structural analysis just described, on the reinforced concrete section was determined by nonlinear sectional analyses. The calculated tensile strains were introduced into the Gergely–Lutz formula[4] to estimate the crack widths. The maximum computed crack width was 0,20 mm, which is within acceptable limits. The suitability of the chosen semi-integral structural systems was thus confirmed.

Pier Heads

The transfer of forces and moments from the superstructure to the piers is accomplished by the pier heads. The flow of forces is complex. The situation is further complicated by the longitudinal and vertical access openings through the transverse diaphragms and the bottom slab of the pier heads. For analysis and detailing, extensive use of the strut-and-tie method was made. Analysis by means of the finite-element method was also performed to independently corroborate some of the results. Based thereupon, it was found necessary to increase the originally envisaged pier box construction depth of 10,0 m, which corresponds to the girder depth at the pier faces, to 10,2 m. By virtue of this change, the centroid axes of the bottom slabs of girder (inclined) and pier box (horizontal) meet centrically with the centroid axis of the corresponding pier column. Local eccentricity moments and the ensuing local stress peaks are thus avoided.

Construction

Five of the twelve pier foundations are spread footings with dimensions 20 × 20 × 3,5 m, and seven consist of pile caps of the same dimensions, each supported on 25 bored cast-in-place reinforced concrete piles ϕ1,8 m. Temporary drilling shaft support was provided by steel tube casings. The pier columns were erected with climbing formwork; the length of each segment was 4,0 m.

After completing the pier heads by using conventional formwork, the cantilever portions of the superstructures were built by the balanced cantilever method (*Fig. 7*). The cantilever segments were built cast-in-place with form travelers which successively proceeded from each pier into both adjacent spans until midspan. The first 2,0 m of each cantilever was cast as monolithic parts of the pier heads and served as starter segments. The length of the subsequent segments, cast on form travelers, was 3,6 m for the first ten segments and 4,4 m for the remaining nine segments (*Fig. 8*). This arrangement was chosen to make steady use of the capacity of the form travelers. Regarding the eastbound viaduct, for instance, the self-weight of the first and heaviest 3,6-m segment was approximately 180 t. The self-weight of the first 4,4-m segment was only 135 t. This weight acted with a larger lever arm on the supporting formwork traveler, however, and thus conveyed about the same moment and overturning effect.

The end parts of the side spans were outside the balanced cantilever range and were cast on ground-supported scaffolding. The cantilever tips in the main spans as well as the tips of the side span cantilevers and the end parts of the side spans were joined by closure pours (key segments) of 2,0 m length each, which were reinforced and prestressed to provide full continuity. Before casting the key segments, misalignments of the cantilever tips were corrected with the help of one remaining form traveler and by means of temporary horizontal prestressing bars (*Fig. 9*).

The long-term longitudinal shortening of the superstructures due to creep and shrinkage results in large bending moments in the outer piers because they are monolithically connected to

Fig. 7: Balanced cantilever construction of superstructure

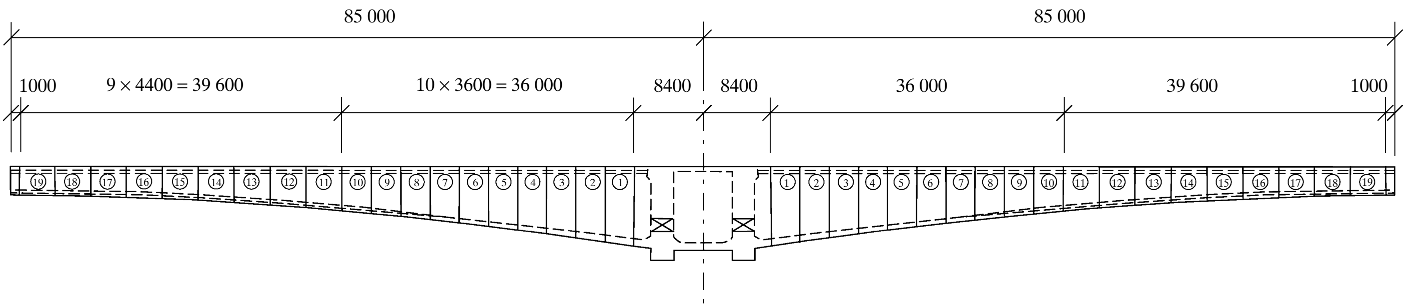

Fig. 8: Balanced cantilever construction sequence (Units: mm)

Fig. 9: Midspan closure (key segment)

the superstructures (instead of using movable bearings). The following prestressing measure is taken to counteract this effect. Before casting the last key segment in each superstructure (*Fig. 1*), the cantilever tips are jacked apart by approximately 100 mm. The corresponding jacking force of about 6 MN was chosen such that the total bending moments at the time of closure are of the same magnitude, but act in opposite direction, as the total bending moments at $t = \infty$. In this way, the moment envelope and the crack width over the lifetime of the structure are minimized.

The design and analysis of this structure were performed in 1998 when the Asian Financial Crisis was at its peak. The start of construction, therefore, was postponed to early 2002. Construction was completed in November 2007.

Conclusion

Shin Chon Bridge consists of two viaducts of 1060 m length each and a total height of about 100 m. The superstructures are post-tensioned concrete box girders that were built by the balanced cantilever method and have a main span length of 170 m. For the sake of economy in construction and maintenance, and to improve durability, the viaducts were designed as semi-integral structures. The superstructures are continuous over their entire lengths and expansion joints are placed only at the abutments. Furthermore, the superstructures are cast monolithically to all piers thus limiting the need for bearings to the abutments. Each pier consists of a pair of slender columns to accommodate the longitudinal shortening of the superstructures due to creep, shrinkage, and temperature variation. Nonlinear structural analyses were performed to verify the global stability of the flexible bridge systems, to determine the sectional forces and crack widths in the piers, and so to confirm the suitability of the chosen semi-integral structural systems. The application of modern engineering principles and the use of sophisticated analysis thus led to an economical, durable, and elegant structure.

References

[1] Menn C. *Prestressed Concrete Bridges*. Birkhäuser Verlag: Basel, 1990.

[2] American Association of State Highway and Transportation Officials. *Standard Specifications for Highway Bridges*, 16th edn. AASHTO: Washington, DC, 1996.

[3] Starossek U. Shin Chon Bridge – Second-order analysis of piers. VSL Korea, Seoul, 1999, available online at www.starossek.de.

[4] Collins MP, Mitchell D. *Prestressed Concrete Structures*. Prentice Hall: Englewood Cliffs, NJ, 1991.

SEI Data Block

Owner:
Korea Expressway Corporation

Designer:
Dong-Il Consulting, Seoul (foundations),
VSL Korea, Seoul (superstructures and piers)

Contractor:
GS Engineering and Construction, Seoul,
VSL Korea, Seoul

Concrete (m^3):	Approx. 80 000
Reinforcing steel (t):	17 149
Prestressing steel (t):	1947
Total cost (USD million):	67
Service Date:	November, 2007