Design and Construction of North Halawa Valley Viaduct

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The North Halawa Valley Viaduct is a 2-km-long prestressed concrete box girder bridge on the island of Oahu, Hawaii. It is the first cast-in-place cantilever segmental bridge in the United States to be built from an overhead erection gantry. The design features of the project are described, and the reasons for choosing that construction method are given. The main features of the operation of an erection gantry are described, and some of the problems arising during the construction of the bridge are discussed. Finally, the instrumentation of the structure to monitor its long-term performance is detailed.

The North Halawa Valley Viaduct is a part of the so-called Interstate route H-3 project on the island of Oahu, Hawaii. As shown in Figure 1, the project runs from the Halawa interchange on H-1, north of Honolulu, across the Koolau Mountains (which form the backbone of Oahu), to Kaneohe, on the windward side of the island. The project is intended to relieve congestion on the existing Pali and Likelike highways, connect the Pearl Harbor naval station with the Marine Corps station at Kaneohe, and provide for future trans-Koolau travel demand.

Project Description

The H-3 project, besides including several miles of at-grade highway, includes three major structures: the trans-Koolau tunnel, which runs approximately a mile beneath the crest of the island; the recently completed Windward Viaduct immediately to the east of the tunnel; and the North Halawa Valley Viaduct immediately to the west of the tunnel. The North Halawa Valley Viaduct was designed by Nakamura & Tyau, of Honolulu, Hawaii, and T. Y. Lin International, of San Francisco, California.

Site Conditions

The bridge runs through the upper North Halawa Valley, which is typical of the deep erosional valleys that have formed on the flanks of the ancient Koolau volcano. The valley is generally V-shaped, with irregular side valleys and ridges extending to the valley bottom; slopes along the project alignment are frequently as steep as 2h:1v to 1h:1v.

The viaduct design reflects the fact that the North Halawa Valley is an environmentally sensitive site. Several native Hawaiian burial sites have been found in the valley; these are now the subject of archaeological investigation. The valley is also a watershed area and will be closed to the general public after the H-3 project is completed. The viaduct has been designed and constructed so as to minimize disturbance to the site.

The North Halawa Stream meanders along the viaduct alignment as shown in Figure 2. The stream is sub-

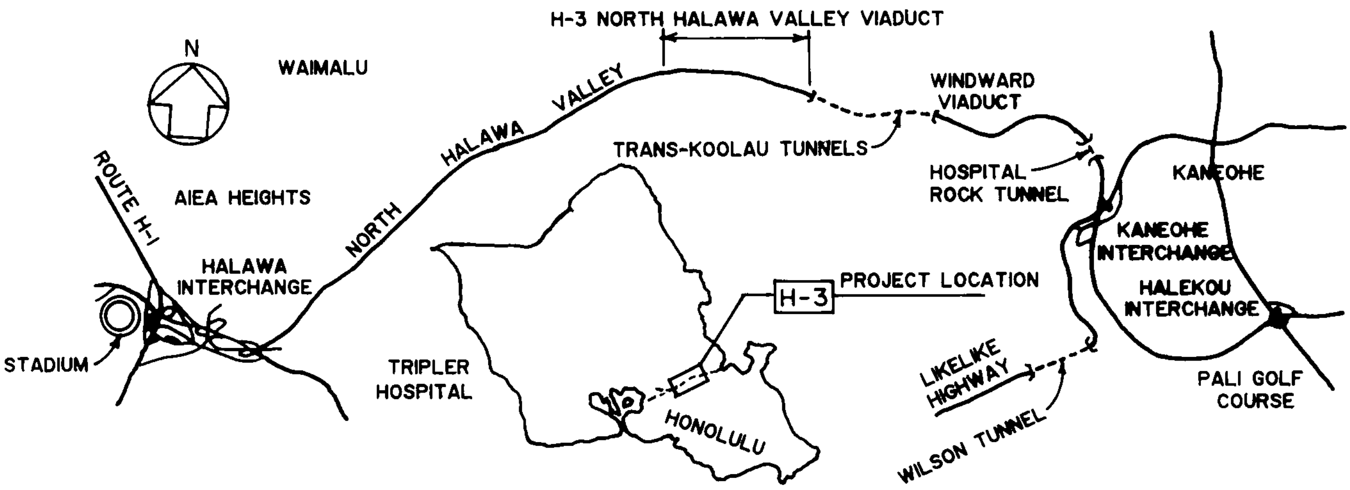

FIGURE 1 Project location.

ject to frequent floods because of an average of 150 in. of rain annually at the top of the valley. An important part of the project involves improving the stream to protect the viaduct foundations from scour.

Description of Structure

A general plan and elevation of the bridge are shown in Figure 2. The project actually consists of two parallel viaducts, one carrying two lanes of traffic inbound to Honolulu and one carrying two lanes of outbound traffic. The inbound viaduct is 1897 m (6,225 ft) long and the outbound viaduct is 1667 m (5,470 ft) long. Both viaducts are aligned horizontally on curves with radii of approximately 1800 m (5,906 ft) at the lower end of the valley and 2900 m (9,514 ft) at the upper end of the valley. The viaducts are on a nearly constant 6 percent grade sloping up toward the mountains (the vertical scale is exaggerated by a factor of two in Figure 2). The typical (and maximum) span length is 109.728 m (360 ft), with some span lengths as small as 91.44 m (300 ft) to accommodate the vagaries of the terrain and the stream in the valley bottom.

Figure 2 shows the viaducts to be divided into three structural units each, averaging 565 m (1,854 ft) in length between expansion joints. Each unit has two fixed piers toward its center and two or three flanking expansion piers on either side. Each unit was constructed by the segmental cantilever method using an overhead erection gantry.

The expansion joints between the units are located at the top of so-called end piers. This was done to avoid the excessive deflections that sometimes accompany midspan hinges and the construction problems that often accompany cantilever construction past a quarter-point hinge. The fact that the end piers, located at what would otherwise appear to be the middle of a span, are perhaps unattractive was discounted because the valley will eventually be closed to the public. The rather large movement ratings of the expansion joints, up to 21 in., combined with the steep 6 percent grade of the bridge led to an unusual design detail. At each end pier the bearings were aligned along the grade of the structure in order to constrain the movement of the bridge to be parallel to the expansion joint. If the bearings had been set horizontally, as is done normally, the movement of the bridge would have had a component perpendicular to the expansion joint, causing a jog in the roadway surface of 1¼ in. Of course, inclining the bearings induces a slope load into the pier and footing, equal to the grade of the bridge times its dead load. But the dead load reactions on the end piers are relatively small, and, fortuitously, most of the end piers are fairly short.

The viaduct cross section is shown in Figure 3. The out-to-out width of the bridge is 12.497 m (41 ft), which accommodates two lanes of traffic plus shoulders. The cross section varies in depth from 2.438 m (8 ft) at midspan to 5.468 m (18 ft) near the piers. These dimensions are typical of cast-in-place structures of about 100-m (328-ft) span length.

Piers

The bridge piers are designed conventionally. They are of hollow construction with a wall thickness of 457 mm (1 ft 6 in.) and exterior dimensions of 3.048 × 7.010 m (10 × 23 ft). They vary in height from 8.336 to 31.766 m (27 ft 4 in. to 104 ft 3 in.). The piers were built in tall lifts of concrete, up to 12.192 m (40 ft), using flying forms. The construction took place without incident.

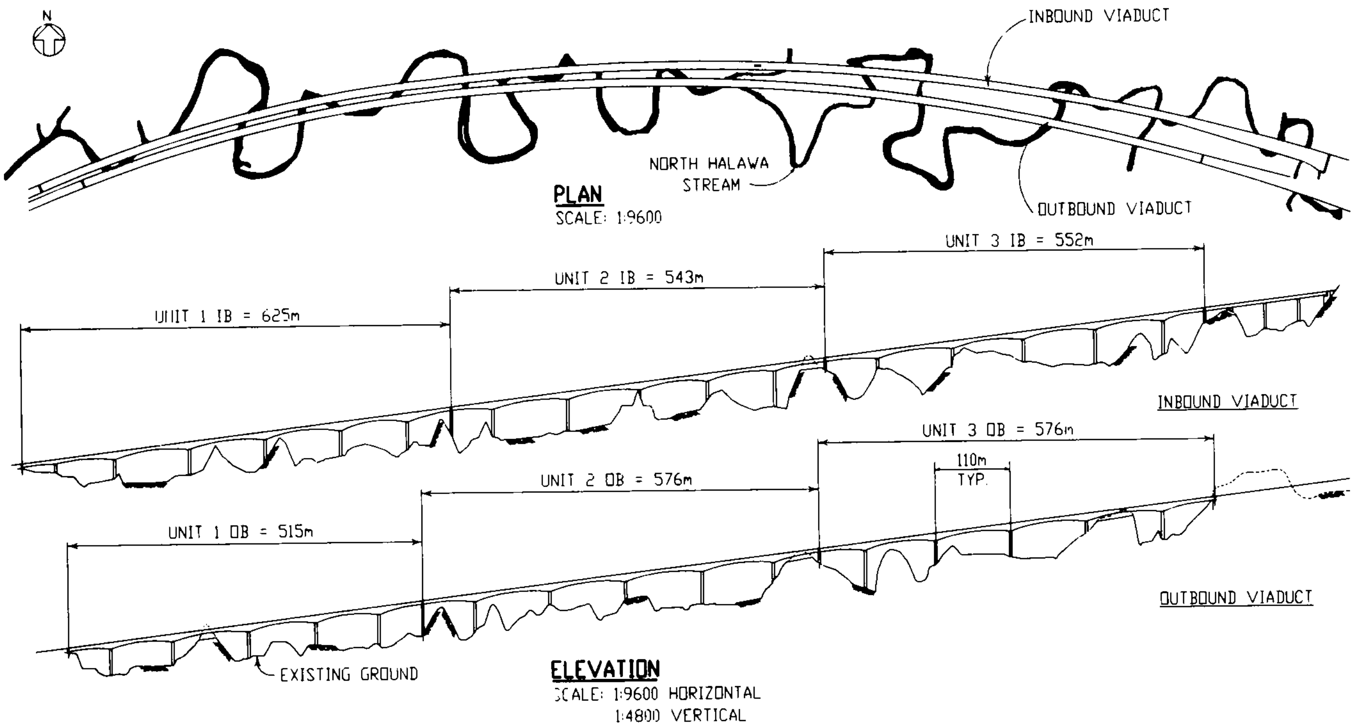

FIGURE 2 General plan and elevation of viaduct.

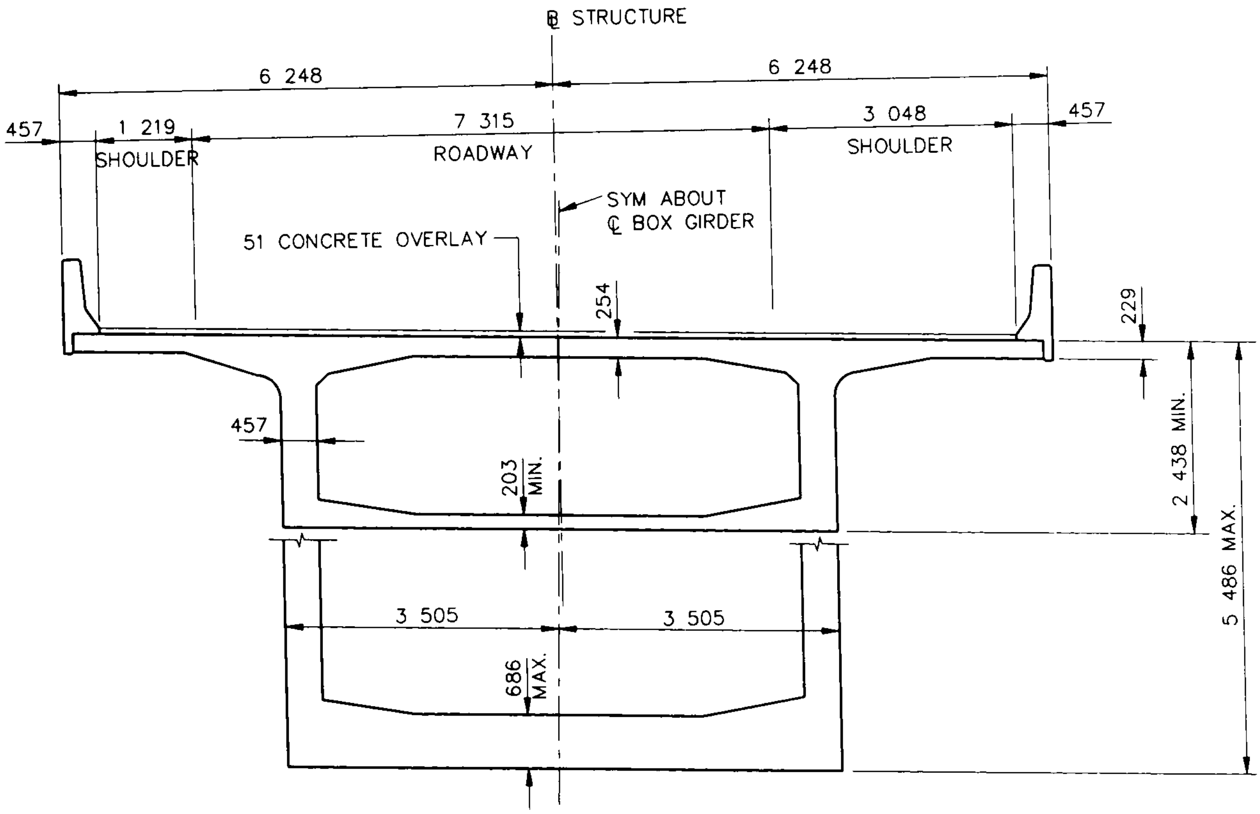

FIGURE 3 Bridge cross section.

Foundations

The soils along the bridge alignment consist of surficial layers of alluvium and colluvium underlain by thick layers of residual soils and saprolite decomposed from basalt. At depths approaching 30.480 m (100 ft), the saprolite gradually gives way to slightly weathered and unweathered basalt rock.

Both prestressed concrete pile and drilled shaft foundations were designed and fully detailed and bid as alternatives. A typical pile foundation consisted of octagonal piles 70 508 mm (20 in.) in diameter. Because at depth the ground is fairly hard, it was anticipated that these piles would be driven through predrilled holes 457 mm (18 in.) in diameter to depths ranging from 15.240 to 30.480 m (50 to 100 ft). Not surprisingly, no contractor bid the pile alternative—it probably does not make sense to predrill piles if one can just make a drilled shaft.

The successful drilled shaft alternative was only the second application of this construction method in Hawaii. A 3 × 3 pattern of drilled shafts 1.524 m (5 ft 0 in.) in diameter is typically used, with one or two shafts occasionally omitted from the pattern. The shafts vary in length from 24.384 to 36.576 m (80 to 120 ft). Further details are given later in the paper.

Three site factors complicated the foundation work. The construction of drilled shafts near the stream had to contend with large fields of boulders. These boulders were deposited over the years as the stream meandered back and forth over the valley bottom. (The boulders were another reason that drilled shafts were favored over piles.) Footings near the stream had to be constructed in cofferdams to protect them from scour, because flooding was a possibility at any time of the year. And several of the viaduct foundations fall on the ridges that intersect the valley bottom; each of these required a large excavation.

Transition Area Box Girder

In addition to the three segmentally constructed units, the inbound viaduct also has a portion 178.308 m (585 ft) long that is built on falsework. This is a conventional multicell box girder at which the viaduct widens so rapidly (to accommodate some transition lanes before the trans-Koolau tunnels) that segmental construction was considered impractical.

Bridge Types and Construction Methods Considered

Only segmental construction methods were considered when the bridge type was selected. Both precast and cast-in-place superstructures were considered, with span lengths ranging from 48.768 to 109.728 m (160 to 360 ft). Balanced cantilever construction, with either form travelers or erection gantries, was considered for both the precast and cast-in-place alternatives. Span-by-span construction was also considered for the cast-in-place alternative.

The precast alternatives were thought to be relatively expensive because there was no suitable location for a segment precasting yard near the bridge site. The only suitable locations were several miles away, outside of the North Halawa Valley. Unfortunately, the access road to the site was long and narrow and had to be shared with several other construction projects in the valley. The Hawaii Department of Transportation and the bridge designers favored cast-in-place construction over precast for subjective reasons: the greater durability of cast-in-place bridges and their possibly better behavior during earthquakes.

In considering only cast-in-place construction, erection from conventional form travelers was thought to be expensive because of the time-consuming assembly and disassembly of the form travelers on each pier (31 in number). Erection from an overhead gantry (as shown in Figure 4) was thought to offer several advantages with respect to erection from form travelers.

The fundamental advantage of this construction method is that, except for the pier segment, the superstructure construction is independent of the ground; all of the necessary materials, equipment, and personnel can be delivered to the point of work from overhead, along the completed structure and over the gantry. And the difficult topography of the valley and the project's environmental restrictions made it desirable to work from overhead as much as possible.

To be sure, 3564 m (11,695 ft) of viaduct is a lot of bridge. Once mobilization costs are overcome, construction with an erection gantry is inherently fast, because large segments are possible and because the launch of the gantry from one pier to another can be quick. Savings are also possible in piers and foundations because the gantry can absorb some of the unbalanced moment of the cantilever. And although the erection gantries themselves are expensive, there are offsetting savings in auxiliary equipment, which is not required to service cantilever construction at each pier, and in spur roads and lay-down areas at each pier, which are not required to service the same volume of material as for conventional construction with form travelers.

The typical span length of 109.728 m (360 ft) is thought to be about the maximum practical span length for this method of construction. The maximum span length was chosen in order to minimize the number of piers and foundations to be built in the difficult terrain. The North Halawa Valley Viaduct is the first cantilever

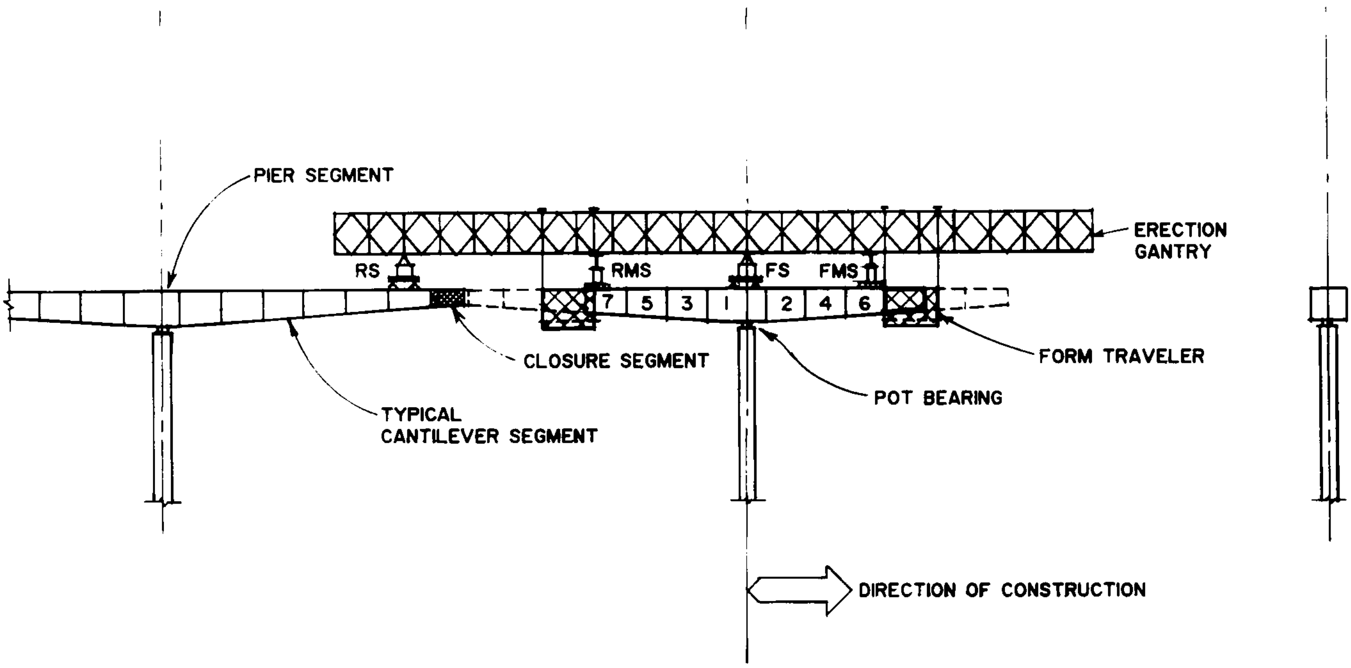

FIGURE 4 Operation of erection gantry.

segmental bridge in the United States to be cast in place from an overhead gantry. The method has been used several times in Europe, however.

Construction

After a very competitive bidding process, a contract for construction of the viaduct was awarded to the Kiewit Pacific Company on December 19, 1991, with a notice to proceed on February 21, 1992. The bid price was approximately $141 million, for the viaducts and the ancillary site work; the viaduct itself cost about $105 million. Nine hundred ninety calendar days were allowed for the construction of the viaducts and for the site work. Major subcontractors to Kiewit Pacific included the VSL Corporation, for erection gantry design and superstructure construction engineering, and the Malcolm Drilling Company, for drilled shaft construction.

Foundations

Drilling the shafts was the first construction activity on the site. They were constructed both in the dry and by tremie inside a casing near the stream. The boulder fields were found to be a significant obstacle to the drilling. In fact, the drilling subcontractor filed a successful claim against the state, charging that the boulders were larger and more densely packed than indicated by the test borings. Boulders smaller than one shaft diameter were usually removed with a choker; the subcontractor was generally able to drill through larger boulders, but some shafts were cut off when very large boulders were encountered. Aside from this problem, however, the construction of the drilled shafts was straightforward. The drilling subcontractor was able to drill about one shaft a day, on average. A total of 6781 m (22,429 ft) of shaft 1.524 m (5 ft) in diameter and 2856 m (9,370 ft) of shaft 914 mm (3 ft) in diameter were drilled and cast in just over a year.

Superstructure

Cantilever Construction

The superstructure was built by cantilever construction about each pier. Each cantilever (or pair of cantilevers) was cast in place in segments using an erection gantry, as shown in Figure 4. At each pier, the construction was started by casting a pier segment directly on top of the pier; this was actually cast in place in formwork supported on the pier shaft. In each case, the pier segment was eccentric with respect to the centerline of the pier, thus applying an unbalanced moment to the pier shaft. The eccentric length of the pier segment was equal to half of the typical segment length.

After casting the pier segment, the typical cantilever segments were cast in place in formwork supported from the erection gantry. The segments were cast alternately to either side of the pier, starting from the side opposite the pier segment cantilever. When released from the formwork, each segment potentially applied a net unbalanced moment to the pier shaft corresponding to half of the segment length times the remainder of the cantilever length minus a quarter of the segment length.

The cantilever construction at each pier was concluded by casting a closure segment between the rear cantilever segment (relative to the direction of construction) and the tip of the previously completed cantilever. The closure segments were cast using the same formwork as for the cantilever segments (with some revisions to the support of the interior formwork).

Because the end spans in each structural unit are longer than a typical cantilever length, often 76.200 m (250 ft) versus about half the span length, or 54.864 m (180 ft), a portion of each end span was cast in place in formwork supported on the ground. These supported segments were typically 12.192 to 18.288 m (40 to 60 ft) long.

Contractor Modifications

Although segmental construction using an erection gantry was anticipated in the design of the viaducts and shown on the contract plans, the actual construction involved two important modifications to the original design. Each of these modifications was made to increase the rate of construction, relative to the conservative assumptions made by the designers of the bridge. One modification made by the contractor was to increase the segment length from the 6.401-m (21-ft) length assumed in the original design to 7.315 m (24 ft). (The original design would have allowed construction with conventional form travelers.) Another modification was to increase the rate of casting of segments from an assumed 5-day cycle (in effect a weekly cycle without working on the weekends) to a 4-day cycle.

Taken together these modifications allowed the contractor to build the bridge with three erection gantries rather than the four shown on the contract plans. One gantry was used on each of the inbound and outbound viaducts, moving uphill from the abutments to construct Units 1 and 2. The third gantry (actually the first to be launched since it was on the critical path) was used to construct Unit 3 of both viaducts; it started on the outbound viaduct, moving downhill from the abutment toward one of the other gantries. When Unit 3 of the outbound viaduct had been completed, this gantry was pushed back up the completed structure and then sideways onto the transition area box girder. It was then launched downhill again, to build Unit 3 of the inbound viaduct.

The posttensioning layout was also revised by the contractor to accommodate the revised segment length; the overall posttensioning scheme was maintained, however. The changes to the segment and posttensioning layouts, and particularly the increased rate of construction of the bridge, made it necessary for the contractor and the designer to reanalyze the bridge, because the creep and shrinkage of concrete renders the behavior of the bridge time-dependent.

Casting Cycle

As mentioned previously, a 4-day cycle was used for casting cantilever segments (indeed, occasionally the contractor was able to build a segment in 3 days). The daily operations in the typical cycle were as follows:

- Day 1: Install the reinforcing and posttensioning in the bottom slab of the cross section.
- Day 2: Install the reinforcing and posttensioning in the webs and the top slab, adjust the forms, and perform a button-up survey.
- Day 3: Perform a prepour survey and cast the segment.
- Day 4: Stress the posttensioning tendons anchored in the segment, break down and move the forms forward, and perform an as-built survey.

The superstructure concrete mix designed by the contractor was an important factor in the success of the 4-day cycle. A strength of 24 MPa (3,500 psi) (70 percent of the design concrete strength) was required for posttensioning of the structure. The mix used had a high cement content, low ratio of water to cement, and superplasticizers. In the insulated forms used, the required strength routinely was reached at about 18 hr after casting. [The 28-day strength of the mix is about 48 MPa (7,000 psi).] Frequently, when the required strength for full posttensioning was not achieved on schedule, partial posttensioning at 17 MPa (2,500 psi) strength was used to allow the forms to be moved forward.

Creep and Shrinkage Testing

As required by the specifications, the contractor performed creep and shrinkage testing of his proposed superstructure concrete mix, both to prove the suitability of the mix for construction and to confirm the characteristics used in the design. Although the creep and shrinkage properties of Hawaiian concrete were researched by the designers of the bridge, such tests are always necessary because of the high variability of concrete as a material.

The original research suggested that the shrinkage of Hawaiian concrete (in general) was much larger than predicted by the CEB-FIP model of the creep and shrinkage of concrete (commonly used for bridge design). The designers assumed the shrinkage of concrete to be 2½ times that predicted by the model, and this factor was confirmed by the shrinkage testing. (Hawaiian aggregates are of volcanic origin, which is not common elsewhere.) The original research did not reveal any systematic difference between the creep of Hawaiian concrete (considering materials from several sources) and that predicted by the CEB-FIP model, but this particular assumption was not confirmed by the testing, which showed the mix proposed by the contractor to have about 25 percent higher creep than predicted. Consequently, the reanalysis of the bridge, made necessary by the contractor's increased rate of construction, used this higher creep coefficient. From the testing, the ultimate shrinkage strain of the superstructure concrete mix was taken to be 0.000449 and the ultimate creep coefficient was taken to be 2.47, for 28-day loading.

Increased Continuity Posttensioning

The increased rate of construction of the bridge and the higher creep coefficient both led to a small increase in the amount of continuity posttensioning needed in the bridge. The so-called continuity tendons are draped tendons in the webs in each span. They are an important factor in eliminating tension in the bottom fibers of the bridge at midspan; the design criteria did not allow any tensile stress in the structure under combinations of live load and temperature gradient. The aforementioned factors both cause tensile stress at midspan. The additional continuity posttensioning needed to compensate for this stress took the form of additional strands placed in the existing ducts; no additional tendons were required. The contractor was compensated for the additional posttensioning required to offset the higher creep coefficient but not for that required to offset the increased rate of construction, since that was his choice relative to the contract plans.

Gantry Design and Operations

The erection gantries, illustrated in Figure 4, were designed and built by the contractor. Each gantry had a crane rail and two cranes of 5.9-ton (13-kip) capacity running along its soffit. The cranes were used to deliver reinforcing bars and other materials to the point of work. Each gantry also had a 480V power line running along its soffit to power equipment at the point of work. Each gantry was 140 m (450 ft) long, weighed about 613 tons (1,350 kips), and cost about $2.5 million, including the form travelers and auxiliary equipment.

As can be seen in Figure 4, the form travelers were suspended from the gantry rather than from "horses" supported on the cantilever, as in conventional cantilever construction. The form travelers themselves were fairly conventional (except in one respect to be discussed later). During cantilever construction, each gantry was balanced on a front main support (FS) on the forward pier (in the direction of construction) and a rear main support (RS) on the tip of the previously completed cantilever. The sequence of casting concrete, stressing tendons, and moving the form travelers was the same as for conventional cantilever construction.

An important feature of the gantry operation was the use of struts between each gantry and the viaduct—the front mobile support (FMS) and rear mobile support (RMS) shown in Figure 4. These were jacks placed immediately behind the form travelers to support the gantry on the bridge deck. They took advantage of the inherent strength and stiffness of the structure to help the gantry carry the load of freshly cast segments. They were moved with the form travelers so as to be always just behind the segment being cast.

Also, at expansion piers, where the superstructure is supported on pot bearings, the struts were used to balance the cantilever on top of the pier. One strut, at least, was always maintained in compression between the gantry and the structure to react the weight of the unbalanced segment on the opposite side of the cantilever. Form travelers and struts being moved to the next segment were always on the heavy side of the cantilever (where a segment had just been cast), so that the strut on the opposite side was compressed naturally by the unbalanced moment. When the form travelers were stationary, both struts were maintained in compression. This scheme enabled the contractor to avoid a complicated moment restraint to stabilize the cantilever against the pier shaft.

Geometrical Control

Because struts were used to share loads between the gantry and the structure, they formed a system that had to be analyzed to determine its behavior under load. This use made the geometrical control of the structure more complicated than it is for conventional cantilever construction. But using the gantries to stabilize the cantilevers at expansion piers (by using the struts just described) allowed those cantilevers to be "rocked" in order to compensate for geometrical errors, in effect providing an additional variable for solving geometrical control problems.

In accordance with the specifications, the contractor wrote a geometry control plan that subsequently was reviewed by the designer. The goal of the plan was to compensate for the deflection and movement of the bridge during construction (due to the method of construction and construction operations) and for the creep and shrinkage of the bridge after construction (up to 20 years). In accordance with the geometry control plan, the bridge was cast to a cambered "control line" so that it would eventually deflect into the required vertical alignment. The plan also contained provisions to compensate for errors in construction (differences from the control line) of more than 25.4 mm (1 in.).

The successful geometrical control of the structure was due to a cooperative effort between the contractor and the designer. Both the contractor and the state of Hawaii maintained a team of bridge surveyors in the field, who independently monitored the position of the bridge at each stage of construction as well as the loads in the gantry front and rear mobile supports (struts). Both the contractor's engineer and the designers ran detailed computer simulations of the construction of the bridge, using the measured mobile support loads at each step. These simulations were compared with each other and with the measured geometry of the bridge before and after the casting of each segment. For each segment, the contractor and the designer agreed to a preset elevation of the segment to meet the control line.

A difficult part of the geometrical control of the bridge was compensating for the deflection of the tip of the previous cantilever, because of creep of the bridge under the gantry rear main support reaction. This deflection was as much as 50 mm (1.97 in.) because of the large lever arm of the reaction. Any deviations from the planned construction schedule led to relatively large deviations from the control line as well.

Closure Segments

Another interesting challenge was the geometrical control of the closure segments between the cantilever under construction and the previous cantilever. The closure segments were usually 7.315 m (24 ft) long, and occasionally 8.534 m (28 ft) long. It was found necessary to place struts on both sides of the closure segments in order to minimize the relative displacements between the tips of the cantilevers during casting. Even with struts on both sides of the closure segments, small relative displacements required compensation by an opposing camber.

Gantry Launching

Subsequent to the closure of each cantilever to the tip of the previous cantilever, the gantry was launched to the next pier, carrying the form travelers with it. The form traveler bottom platforms could split open so that they could pass around the piers.

Each gantry was launched by hydraulic rams mounted on the rear main support. The rams would either push the gantry uphill or lower it downhill, according to its overall direction of motion. Each gantry was held in position by toothed grippers acting between its bottom flange and the rear main support while the rams were recycled. These grippers were used to hold the gantry firmly in position during cantilever construction also.

The launching of a gantry was a complicated operation requiring many steps. Three basic operations were involved. The most basic of these was moving the gantry forward over the front and rear main supports by pushing against the rear main support. The gantries moved over Hilman rollers on the main supports. Another basic operation was the temporary support of each gantry on either the front or the rear mobile support (strut), which relieved the load of the gantry on the corresponding main support and allowed that support to be moved. The third basic operation was moving the form travelers to reposition the center of gravity of the gantry/form traveler system to change the reactions on the gantry supports.

At each pier, a sequence of these operations was planned to simultaneously maintain the equilibrium of the gantry and avoid any overstress of the structure. The launch sequence was summarized by the contractor in a launch manual that was carefully reviewed by the designer.

Perhaps the most critical step in the launch procedure was the relocation of the front main support to the next pier for cantilever construction. This relocation was accomplished by launching the gantry tip as far the next pier, in steps as described earlier. The front main support was then moved to the next pier while the gantry tip was supported temporarily on that pier by the front mobile support. Similarly, the rear main support was relocated to the tip of the just-completed cantilever; it was then posttensioned to the bridge to carry uplift forces.

The typical launch of a gantry involved about 100 steps. Despite this evident complexity, the typical launch took only $2^1/_2$ days once the contractor gained experience. This speed was possible because each of the steps was simple and the operations of the gantry were highly automated.

Instrumentation

An unusual aspect of the project is the instrumentation of one of the structural units, specifically, Unit 2 of the inbound viaduct (the last unit to be completed). The instrumentation program will gather data on the behavior of the structure for comparison with the design assumptions and the predicted behavior. The instrumentation program was a joint effort between T.Y. Lin International; the Construction Technologies Laboratory, Chicago, Illinois; and the University of Hawaii, Manoa. It will continue for 5 years after the completion of the bridge. The measurements include:

1. The forces in six posttensioning tendons will be measured by load cells placed behind the anchor heads.
2. At instrumented sections of the bridge, the strains in the concrete will be measured by cast-in-situ vibrating wire strain gauges and mechanical extensometers. There are four instrumented sections: two at midspan locations and two near piers.
3. At instrumented sections, the strains in some of the reinforcing bars will be measured by resistance strain gauges, adjacent to the concrete strain gauges.
4. At instrumented sections, the average temperature of the concrete and the temperature gradients over the depths of the section and the top slab will be measured by thermocouples placed around the perimeter of the box.
5. The vertical deflections of the bridge will be measured by reference to a high-strength piano wire stretched over the length of the unit and supported inside the cross section at the piers.
6. The rotations of the bridge over the piers will be measured by tiltmeters.
7. The horizontal displacements of the bridge will be measured at the joints by linear variable differential transformers spanning between the soffit of the box and the tops of the piers.
8. Any lateral displacements of the bridge will be surveyed from the ground.
9. The creep and shrinkage properties of the concrete used in the instrumented sections will be determined by testing under laboratory and site conditions.

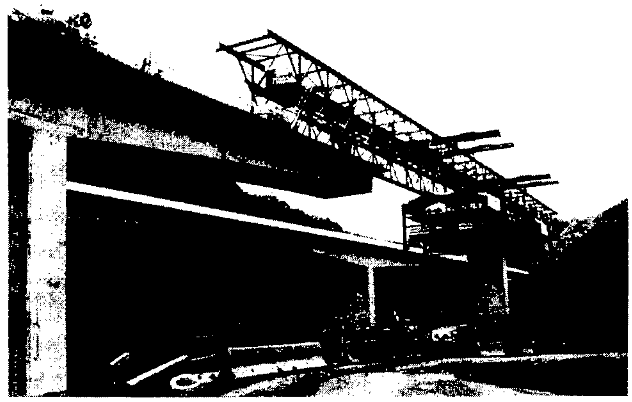

FIGURE 5 Partly completed structure.

Hopefully, comparison of the collected data with the design assumptions and the predicted behavior of the bridge will lead to an improved understanding of long-span posttensioned bridges in general, as well as to improved design methodologies.

Conclusions

Figure 5 shows a photograph of the two viaducts, one of them completed and one of them with an erection gantry in place. When this paper was written, the construction of the superstructure had been completed. Only the deck overlay and expansion joints and some site work remained to be completed. A total of 445 superstructure segments were cast in place in just over 2 years. The successful use of the overhead gantries shows the viability of this construction method where conditions make other methods unsuitable.