West Seattle Swing Bridge

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Construction is under way on a double leaf concrete swing bridge across the Duwammish River in Seattle, Washington. This bridge represents a revival of a type of movable bridge long out of favor and incorporates some new concepts in machinery and movable-bridge technology. The structure and its site are described and some of the factors that led to this novel design are detailed. Construction progress to date is reviewed.

The West Seattle Swing Bridge connects two portions of the industrial port area of Seattle across one of its major shipping channels. Prospects for widening the current 150-ft-wide ship channel to 250 ft required replacement of the existing bascule, built in 1927. In 1988 navigation traffic required approximately 10 openings per day of the existing bridge, which provides 45 ft of vertical clearance over high water. Increasing the vertical clearance from 45 ft for the existing bascule bridge to 55 ft for the new bridge is predicted to be sufficient to reduce the number of openings to an average of seven per day, primarily for large ocean-going barges and ships.

Vehicular traffic crossing the waterway bound for residential areas in the western part of Seattle had previously been routed over a new high-level bridge (140-ft vertical clearance). Construction of the high-level bridge had been long planned but was finally precipitated when an inbound freighter struck the bascule bridge that originally parallelled the bascule bridge being replaced by the swing bridge. The high-level bridge did not, however, provide for the local traffic between the two industrial areas on either side of the waterway because of the difficulty of providing adequate ramps to the elevated structure. This local traffic was 3,500 vehicle a day, 15 percent trucks, before closure of the existing bascule and is predicted to increase to 11,600 vehicles a day by the year 2000. Pedestrians and bicycles are also provided for on the new swing span but are excluded from the high-level bridge. The total structure width of 49 ft 9 in. provides for two traffic lanes and one combined bicycle-pedestrian way of 12 ft 0 in.

SITE DESCRIPTION

The swing bridge alignment was placed on the existing bridge alignment to minimize required right-of-way and revisions to the existing street network. This alignment results in skewing of the bridge axis approximately 45 degrees to the channel. A total of 19 different alignments in the vicinity and three basic structure types was evaluated before selection of the swing bridge for final design development. Other structure types investigated in the preliminary design stages were a vertical lift bridge and a bascule bridge. The vertical lift bridge was believed to be aesthetically incompatible with the immediately adjacent high-level bridge. A vertical clearance of 140 ft would have been required and the towers would have extended above the level of the deck of the adjacent bridge. The 45-degree skew of the channel alignment to the street alignment would have required a lift span length of approximately 460 ft. A double leaf bascule bridge was also investigated. The required span length was approximately 380 ft trunion to trunion, with a skew of 30 degrees between the channel and the roadway. The alignment shift to reduce the skew from 45 to 30 degrees would have added two reverse curves to the street and required additional right-of-way.

The 480 ft center to center of pivot piers for the swing bridge was established by the width of the open west leaf and the location of a column of the high-level bridge (see Figure 1). The east leaf is then symmetrical about the center line of the channel. The tail span length of 173 ft was also set by the column of the high-level bridge. Once the alignment was chosen, the span lengths were fixed. The west approach length is determined by the need to cross over a railroad track and intersecting street, the east approach by grades. Stair towers for pedestrian access are provided on each approach. A control tower for the bridge operator is adjacent to, but separate from, the west pivot pier. The control tower is a 120-ft-high structure so that the bridge operator has an unrestricted view of the channel and all of the approach roadway.

The crossing site is near the mouth of the Duwammish River. Soils encountered at the site range from hydraulic fill and recent alluvial sands and silts to heavily preconsolidated glacial till. Depth to the till varies from 50 ft on the west end of the project to 200 ft on the east end. Lenses of loose silt exist erratically throughout the alluvium layer. The loose surficial silts and hydraulic fill are deemed susceptible to liquefaction in major seismic events. Densification by Vibrofloatation was specified to prevent such liquefaction.

Seismicity of the area is moderately active (UBC Zone III seismic acceleration coefficient $A = 0.25$). The seismicity studies and the seismic design of the high-level bridge were discussed in a previous paper (*1*) along with details of the geology and foundation conditions at this site.

STRUCTURE

Considerations in the choice for the superstructure design were construction economy, maintenance costs, traffic safety, and aesthetic compatibility with the adjacent high-level bridge (a concrete box girder). Two designs were prepared and advertised for bids, a posttensioned segmental concrete box girder and a steel box girder with a precast prestressed concrete deck made composite after erection. Typical cross sections of the two alternatives are shown in Figure 2. Five bids were received for the concrete box girder alternative; they ranged from $33,537,636 to $37,583,660 including all approach

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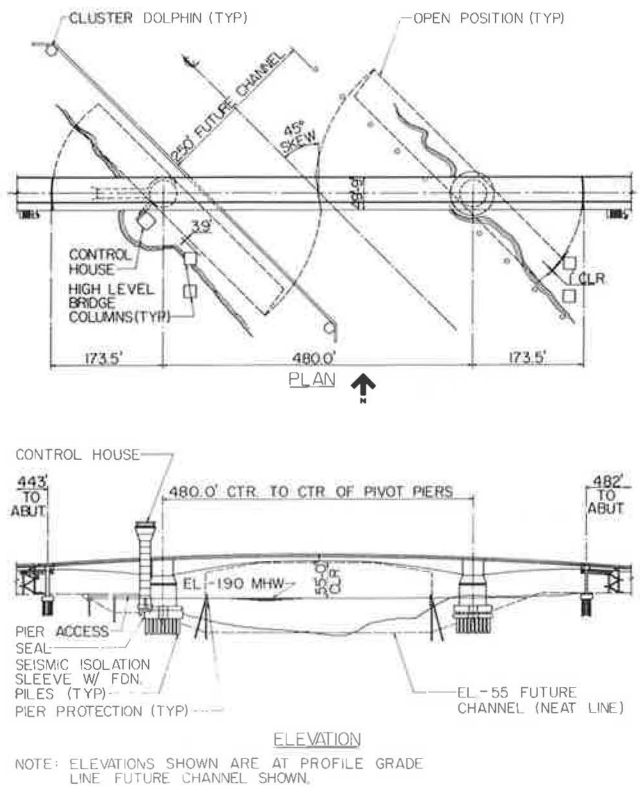

FIGURE 1 West Seattle Swing Bridge: plan and elevation.

work and site work. The concrete box girder was awarded. Principal construction quantities are given in Table 1, and the cost breakdown for major work categories (based on the low bid) is presented in Table 2. Bids received and the engineer's estimate are given in Table 3.

A major concern regarding the design of the concrete box girder bridge was control of long-term deformations. Provisions for control of both long-term and short-term deformations included additional posttensioning beyond that required for stress control, some unbonded tendons, additional future tendons, adjustment of approach span elevation at the tail span joint, vertical adjustment of each leaf as a whole, and specifications requiring nearly simultaneous construction of the two movable leaves.

The first line of defense against unwanted long-term deformation was the adoption of the principle of load balancing for the design of the longitudinal posttensioning. The amount of prestress provided is the amount required to provide 100 percent load balancing for the final dead load condition. This required approximately 30 percent more posttensioning than the amount required to satisfy service load stress conditions. The deck of the box girder is posttensioned transversely and vertical posttensioning is included in the webs. The additional longitudinal posttensioning reduced the need for vertical posttensioning in some areas. The box girder cross section is similar to that adopted for the adjacent high-level bridge.

Static balance of each leaf about the pivot pier is achieved by thickening component section elements in the tail span and by adding ballast concrete so that the last 40 ft of the tail span is solid except for a 4-ft-square access shaft. Some precast ballast blocks are provided for adjustment of the static balance. As-built measurements after casting of each section are required so that the final static balance can be closely predicted. The 42-ft-diameter pier house (see Figure 3) carries the superstructure loads to the foundations and houses the drive machinery, emergency generators, and part of the control system. Pier house walls are 32 in. thick and heavily reinforced.

The pier table element of each leaf is supported by a transition element that provides two load paths to the foundation. The closed position path (serving vehicular traffic) is from the superstructure pier table through a conical shell to the walls of the pier house. Service bearings made of steel plates with reinforced elastomers separate the transition element from the roof of the pier house. The operating position load path is from the pier table through the center portion of the transition element to the 12-ft-diameter pivot shaft. The pivot shaft is a concrete-filled steel shell that rests on the hydraulically operated lift-turn cylinder. It is maintained in the vertical position by guide bearings at the roof and operating floor level of the pier house. In the operating position, the whole movable leaf, including transition element and pivot shaft, are raised approximately 1 in. to transfer the load from the service bearings to the pivot shaft. A reinforced-concrete footing founded on 36-in.-diameter concrete-filled steel pipe piles completes both load paths.

The piling system incorporates 48-in.-diameter steel pipe sleeves around each foundation pile. The purpose of these sleeves is to control the elevation at which the foundation piling begins to receive lateral support from the surrounding soil. The annular space is excavated to a set elevation. The location of the piers is in the slope area of the channel excavation. Without the sleeves, the depth from the footing to the slope surface would vary from zero to 15 ft. Lateral stiffness of the piles would thus vary and significant torsional response to seismic excitation would result from the eccentricity between the center of mass and the center of stiffness. The sleeves eliminate this variation in lateral stiffness and the resulting eccentricity. The sleeves also support the tremie seal, which is separated from the footing.

Twin hydraulic slewing cylinders rotate each movable leaf from the closed position to the open position to allow for passage of ships. The operational cycle is based on a normal 2-min slewing time; the total cycle, which includes setting traffic lights and lowering gates, is approximately 4½ min. Friction is minimized because the structure is supported on the hydraulic fluid of the lift-turn cylinder. The principal source of friction is the pivot shaft in vertical alignment.

Power for raising and rotating the movable leaf is provided by three 100-hp, 125-gal/min hydraulic pumps in each pier house. Normal operation is with two pumps; the third pump alternates as a spare. Other redundancies built into the system include the ability to slew the bridge using only one slewing cylinder (with increased cycle time) and even to slew the bridge against the friction of the service bearings should the lift-turn cylinder fail to operate. This latter redundancy is an extraordinary condition to be undertaken only in extreme emergency. It requires manual overrides of pressure relief valves in the hydraulic system and would require replacement of part of the service bearings. The center lock and tail locks are driven and pulled by local hydraulic cylinders operated by separate pumps. Design for the locks included the ability to be driven against a 1-in. misalignment. Torsional stiffness of the box girder is sufficient so that locks are required only at the center line.

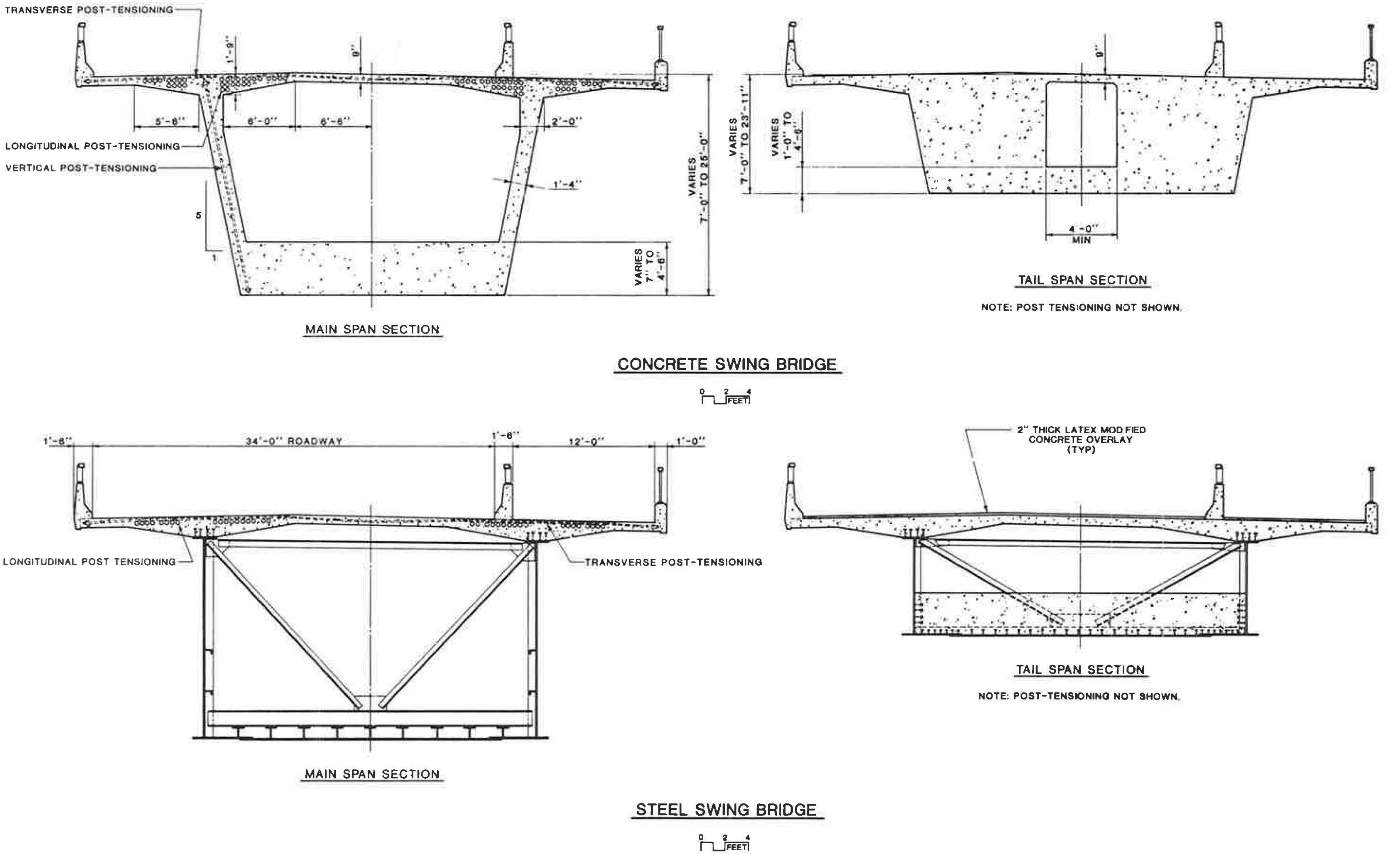

FIGURE 2 Cross sections of alternative designs: concrete and steel swing bridges.

TABLE 1 PRINCIPAL CONSTRUCTION QUANTITIES (STRUCTURAL)

		Approaches		Swing Spans	
Item	Unit	Sub Struc	Super Struc	Sub Struc	Super Struc
Structural Excavation	cy	3200		5220	
Shoring	sf				
Soil Densification	cy	38600		18900	
PSC Piling 16.5" Dia	lf	19335			
PSC Piles 16.5" Dia	ea	290			
Steel Piling 36" Dia	lf			10120	
Steel Piles 36" Dia	ea			64	
Concrete Cl D	cy			1372	
Concrete Cl C	cy			186	82
Concrete Cl B	cy				
Concrete Cl AX	cy	2880	1370	3284	65
Concrete Cl PC 5000	cy	275	606	1195	
Concrete Cl PC 6000	cy			237	6604
PSC Girder S4	lf		911		152
PSC Girder S120	lf		2750		
PSC Girder M120	lf		3404		
Reinf Steel	Ton	337	115	347	420
Rein Steel, Epoxy Ct'd	Ton		115		92
Prestressing Steel, Bar	Ton			1.3	14.6
Prestressing Steel, Strand	Ton				267
Concrete Barriers	lf		3496		2475
Metal Railing	lf		3035		
Latex Mod Concrete	sy				4450
Control System	LS				
Hydraulic System	LS				
Lift/Turn Cylinders	ea			3	
Pivot Shafts	ea		2		
Control Tower	LS				

TABLE 2 COST BREAKDOWN

		LOW BID
1.	Mobilization	$2,800,000
2.	Demolition	3,761,566
3.	Civil (Streets, Utilities, Traffic)	3,395,615
4.	Approach Spans	4,659,520
5.	Swing Piers	6,868,280
6.	Swing Spans	5,601,985
7.	Machinery	3,977,000
8.	Electrical	1,055,895
9.	Controls, Control Tower	941,000
10.	Pier Protection	476,775

		$33,537,636

TABLE 3 BIDS RECEIVED

BIDDERS	Concrete Alternate	Steel Alternate
Engineer's Estimate	$33,330,749.40	$36,621,554.40
Kiewit - Global	33,526,192.00	No Bid
General Construction Co./ 3A Industries, J.V.	33,750,799.50	No Bid
Guy F. Atkinson Const. Co.	34,854,453.00	No Bid
S.J. Groves & Sons Co.	36,803,487.00	No Bid
Paschen Contractors, Inc.	37,560,625.88	No Bid

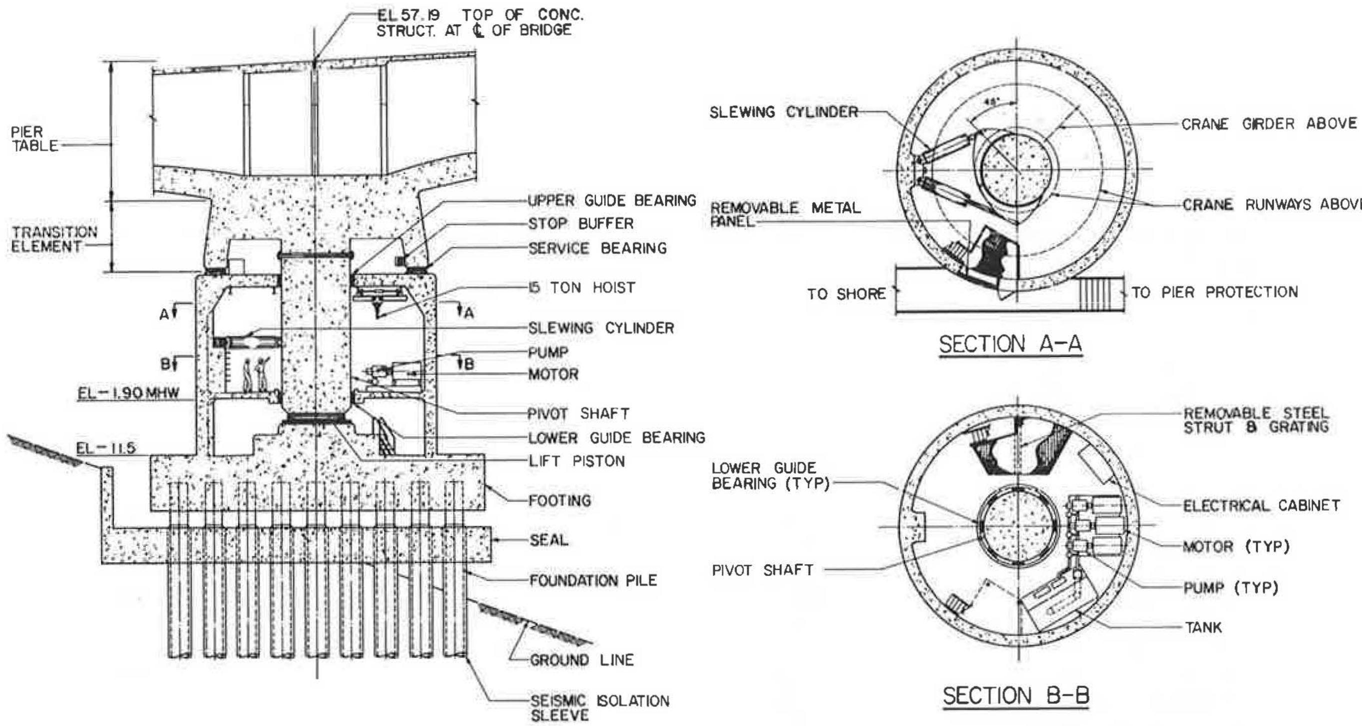

FIGURE 3 Pivot pier section.

Hydraulic system components are designed to operate at a normal pressure of 1,700 psi and an emergency (slewing against the service bearing friction) pressure of 6,000 psi.

Hydraulic buffers are designed to stop the leaf moving at full rotation speed of 0.57 degree per second in 0.44 degree of travel. Open position buffers are located on the roof of the pier house and contact stops on the inner surface of the transition element cone. Tail span buffers are located on the approach span piers and contact the tail span. Normal closing speeds at buffer contact are 35 percent of normal speed; thus normal operation buffer loads are small.

The control unit is a programmable controller that sequences operations and provides status information to the bridge operator. The primary position control is limit switches that initiate braking through the controller. The dynamics of the operating structure (large inertia, low damping) were not deemed suitable for dynamic feedback control. The control system is essentially the same as manual operation except that the controller "pushes the buttons," checks interlocks, and announces the status on a monitor. Manual override is possible for most steps of the operation.

Diesel-powered emergency generator sets (350 kW) are provided in each pier house on the lower floor. Fuel is stored in above-ground enclosed tanks on shore.

One of the design exercises of great value was a study of maintenance actions to ensure provision for access to all items. Adequate openings were provided for removal and replacement of each piece of machinery and all pieces of equipment and areas of the bridge had a means of safe access for inspection and maintenance.

CONSTRUCTION PROGRESS

Bids for the construction of the project were taken in September 1988. Five bids were received (Table 3). The contract was awarded to Kiewit-Global on September 28, 1988, and notice to proceed was given on March 1, 1989. The contract calls for a 22-month construction schedule. Construction progress as of May 1990 included machinery fabrication, demolition of the existing bascule superstructure, site utiity work, and substructure construction. Scheduled completion date is January 1991.

The project was designed by the joint venture West Seattle Bridge Design Team composed of Andersen Bjornstad Kane Jacobs, Inc.; Parsons Brinckerhoff, Quade, and Douglas; and Tudor Engineering. Contech Consultants were responsible for the segmental box girder design. Hamilton Engineering, Inc., designed the hydraulic machinery and Elcon provided the electrical design. The project sponsor is the Seattle Engineering Department, and Frank Yamagimachi is project engineer for the city.

REFERENCE

1. J. H. Clark. Foundation Design: West Seattle Bridge. In *Transportation Research Record 982*, TRB, National Research Council, Washington, D.C., 1984.

Publication of this paper sponsored by Committee on General Structures.