Segmental Box Girders for the High Level West Seattle Bridge

Ching K. Yu, P.E.
Manager
Contech Consultants, Inc.
Seattle, Washington

For many years two bascule bridges over the west waterway of the Duwamish River were the only direct link between West Seattle, Washington, and the downtown area. Busy maritime traffic and an ever-increasing vehicular volume on the bridges caused serious traffic congestion, making the need for a high level structure apparent.

On June 11, 1978, a fully-loaded 12,000 ton (10,900 t) freighter slammed into the north bascule bridge, severely damaging the superstructure and powerhouse wall and rendering the bridge inoperable. This accident left only the south bascule bridge, with its four-lane capacity, to handle all traffic, creating an even more acute need for a high level replacement bridge.

The initial planning for the ensuing replacement project has been described previously.[1]

As shown in the vicinity map (Fig. 1), the entire project was divided into four units: West Interchange, Main Span, Harbor Island Approach and East Interchange. A paper published in the November-December 1983 PCI JOURNAL describes the design and construction of the three approach structure units.[2] This article presents a detailed discussion of the main span.

Structures selected for final design included three types: (1) steel box girders with an orthotropic steel plate deck, (2) cast-in-place segmental prestressed concrete box girders, and (3) precast segmental prestressed concrete box

Rising 150 ft (46 m) over the Duwamish River in the State of Washington, the recently completed main span of the West Seattle Bridge is an excellent example of efficient use of prestressed concrete for a long-span structure. This article presents highlights of the structure's design and construction.

girders. Only the superstructure design of the two concrete alternates will be covered in this article.

Fig. 2 shows the general plan and elevation of the concrete bridge, which has a span arrangement of 375-590-375 ft (114-180-114 m). The superstructure is rigidly connected to the two center piers. The bearings at the end piers are free to slide in a longitudinal direction but restrained laterally. The center piers are designed to withhold the temperature, creep, and shrinkage deformations of the center span.

The overall outlines of the structure are identical for both the cast-in-place and precast schemes. However, the minimum concrete strengths at 28 days are 5000 and 6000 psi (35 and 41 MPa) for the cast-in-place and precast alternates, respectively. This allows a thinner bottom slab for precast segments at the center piers. Otherwise, all the concrete dimensions were kept uniform for both construction methods. Unless otherwise stated, the description hereinafter applies to both superstructure alternates.

The bridge section is composed of twin single cell boxes of trapezoidal shape (Fig. 3). The total deck width of 104 ft 5 in. (32 m) provides a roadway of six traffic lanes and shoulders. Following a parabolic curve at its soffit line, the

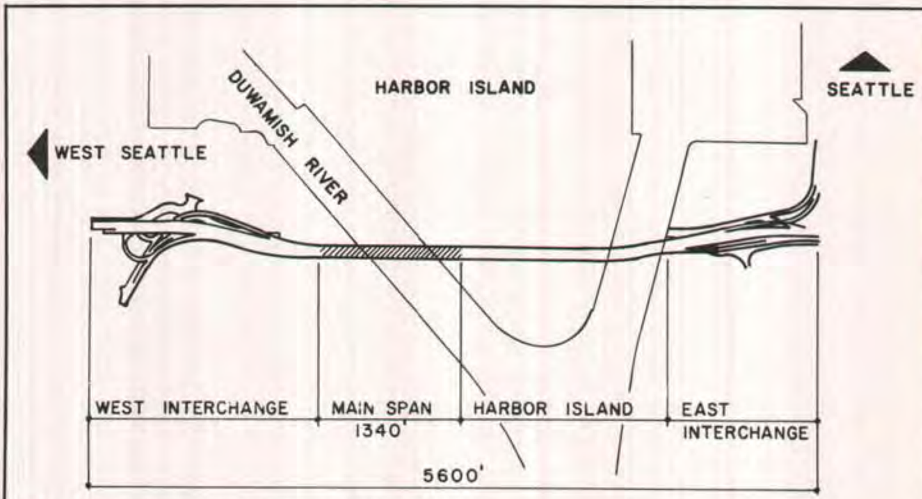

Fig. 1. Vicinity map of West Seattle Bridge.

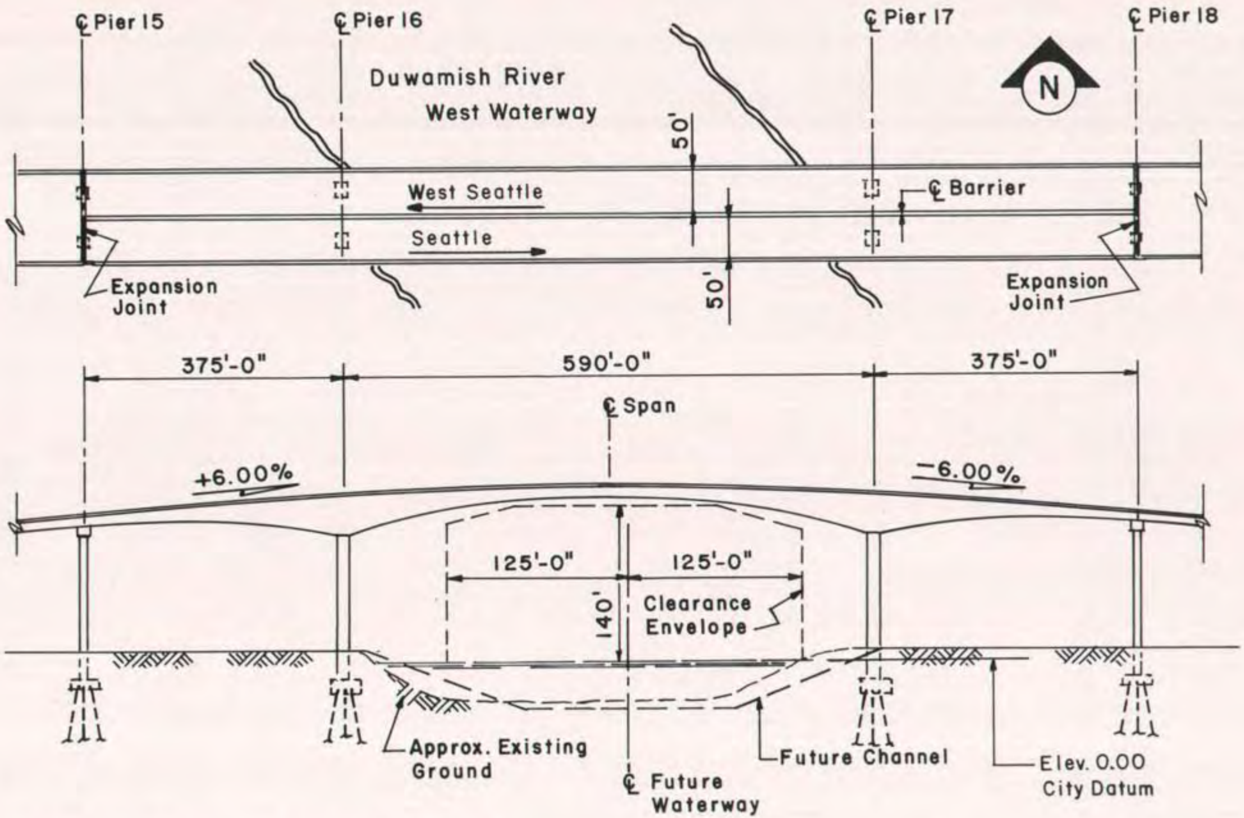

Fig. 2. General plan and elevation of main spans.

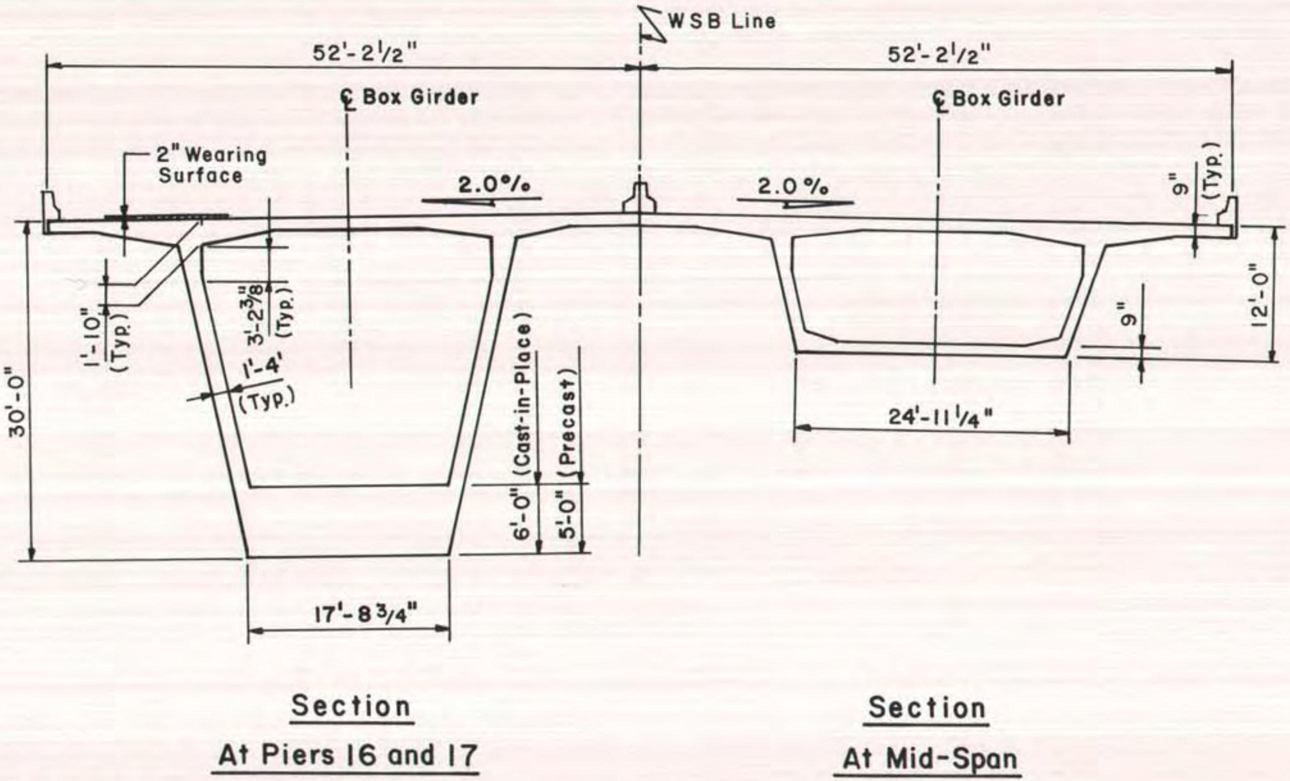

Fig. 3. Typical sections of main spans.

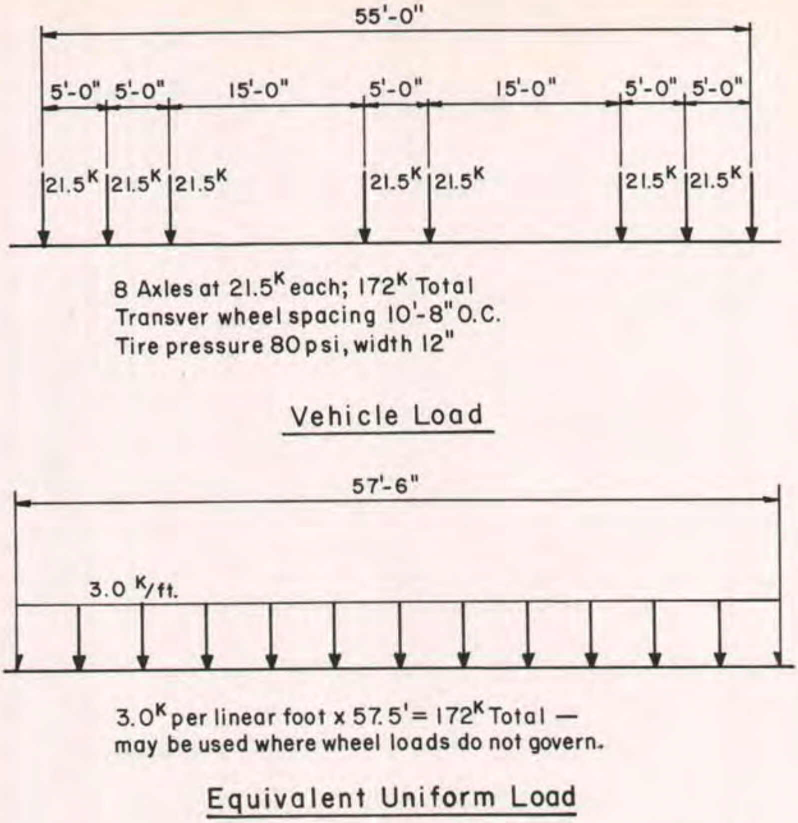

Fig. 4. Assumed overload due to vehicle load and equivalent uniform load.

depth of the girder varies from 30 ft (9.1 m) at center pier to 12 ft (3.7 m) at midspan. The slightly arched form of the superstructure provides not only efficient use of concrete materials and prestressing, but is also aesthetically appealing.

Design Criteria

In general, the structural design follows AASHTO Standard Specifications.[3] For subjects not covered by AASHTO, other commonly used codes for segmental construction were adopted.[4,5,6]

In application of AASHTO loading combinations, the transverse design included a temperature differential of 10 deg F (6 deg C) warmer or 5 deg F (3 deg C) cooler at the outside than the inside of the box. A temperature for the top slab 18 deg F (10 deg C) warmer or 9 deg F (5 deg C) cooler than the rest of the section was incorporated into the longitudinal design.

In addition to the interstate highway bridge loadings specified in AASHTO, Article 1.2.5(G), the bridge was also designed for an overload vehicle or its equivalent (Fig. 4).

Allowable concrete stresses in tension specified in AASHTO, Article 1.6.6, were modified to account for frequent segmental joints in the superstructure

Table 1. Allowable tensile stresses in psi (f_t) for cast-in-place construction.

Loading combinations	No additional reinforcement	All tension force resisted by additional reinforcement
Dead + Live or Dead + Construction	0	$0 < f_t < 3\sqrt{f'_c}$
All other combinations	$3\sqrt{f'_c}$	$3\sqrt{f'_c} < f_t < 6\sqrt{f'_c}$

Note: f'_c = compressive strength of concrete in psi at 28 days or, for dead and construction load combination, at time of stressing.

Table 2. Minimum compressive stresses in psi for precast construction (longitudinal design only).

Loading combinations	Deck slab local stresses not included	Deck slab local stresses included	Other area
Dead + Live or Dead + Construction	200	100	30
All other combinations	30	30	30

Note: 1 psi = 0.006895 MPa.

and to provide better control over deck slab concrete cracking. Table 1 shows the allowable prestressed concrete tensile stresses for cast-in-place construction, applicable to the design of both transverse deck and longitudinal cross sections.

The precast design's allowable stress for the transverse deck section remains the same as in Table 1. However, in recognition of reinforcement discontinuity at segmental joints, allowable longitudinal tensile stress requirements are more stringent. In fact, minimum compressive stresses are required for this direction (Table 2). Allowable compressive stresses for each construction are in conformance with AASHTO requirements.

A dense concrete or latex-modified concrete wearing surface of 2 in. (51 mm) nominal and 1½ in. (38 mm) minimum thickness over the bridge deck was specified. An additional superimposed dead load of 25 lb per sq ft (1.2 kN/m²) of deck surface was designed for as a future surfacing allowance.

Construction Sequence

As shown in Fig. 5, the suggested construction sequence may be divided into four stages:

Stage 1: Casting the pier table over both the north and south columns at the east side of the river. Post-tensioning all the tendons in the pier table and placing two sets of form travellers in position, one pair for each box girder. For the purpose of reducing the unbalanced pier moment, the pier table is offset a half segment length from the centerline of the pier.

Stage 2: Performing the balanced free cantilever construction by casting 16 ft 6 in. (5 m) segments on alternate sides of the pier table. Stressing half of the transverse tendons in each girder from the outboard edge of the deck slab, as shown in Fig. 6. The other half will be stressed only after the longitudinal closure between the two girders is cast. Progress of the two parallel girders should be approximately the same, with a difference of no more than two segments. This is to minimize possible misalignment of the continuous transverse tendons due to concrete creep effect.

Stage 3: After the balanced cantilevers

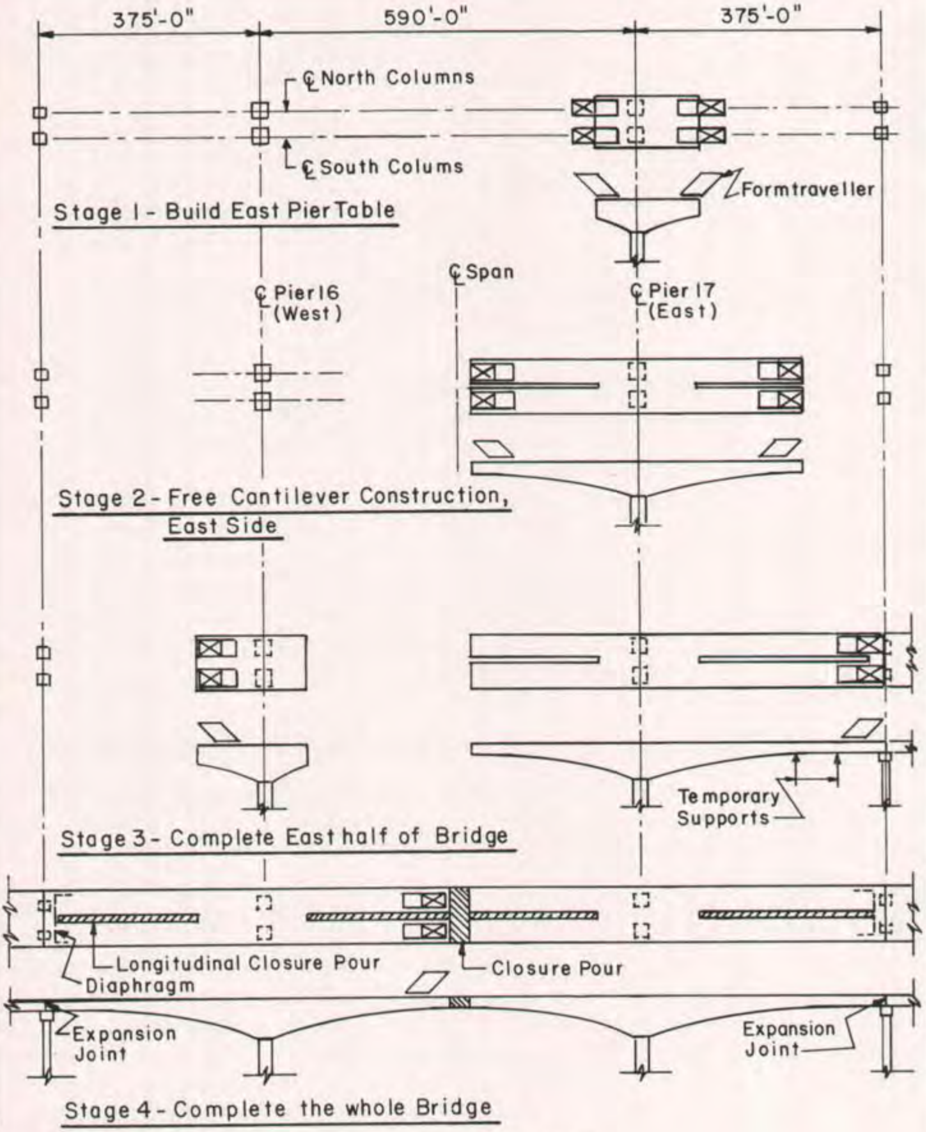

Fig. 5. Suggested construction sequence of main spans.

reach their limits, moving the center span form travellers to the west pier table, installing temporary supports near the edges of the side span cantilevers and continuing segmental construction until the girders land on the end pier.

Stage 4: Repeating the construction as described above for the west half of the bridge. Pouring the center and longitudinal closures, stressing all the continuous transverse tendons and completing other work.

The construction sequence for the precast scheme is basically the same as described above. However, to facilitate transportation, handling and erection of the precast units, the segment length is reduced to between 6 ft (2 m) at pier and 12 ft (4 m) at midspan.

Prestressing System

Longitudinal tendons consisting of 12 x 0.6 in. (15 mm) diameter strands were specified in the design. As shown in Fig. 7, the cantilever tendons are anchored, three at each web, on the bulkhead face of each segment, the continuity tendons at buttresses built up from the slab. The cantilever tendons follow the profile grade and the continuity tendons follow the soffit line.

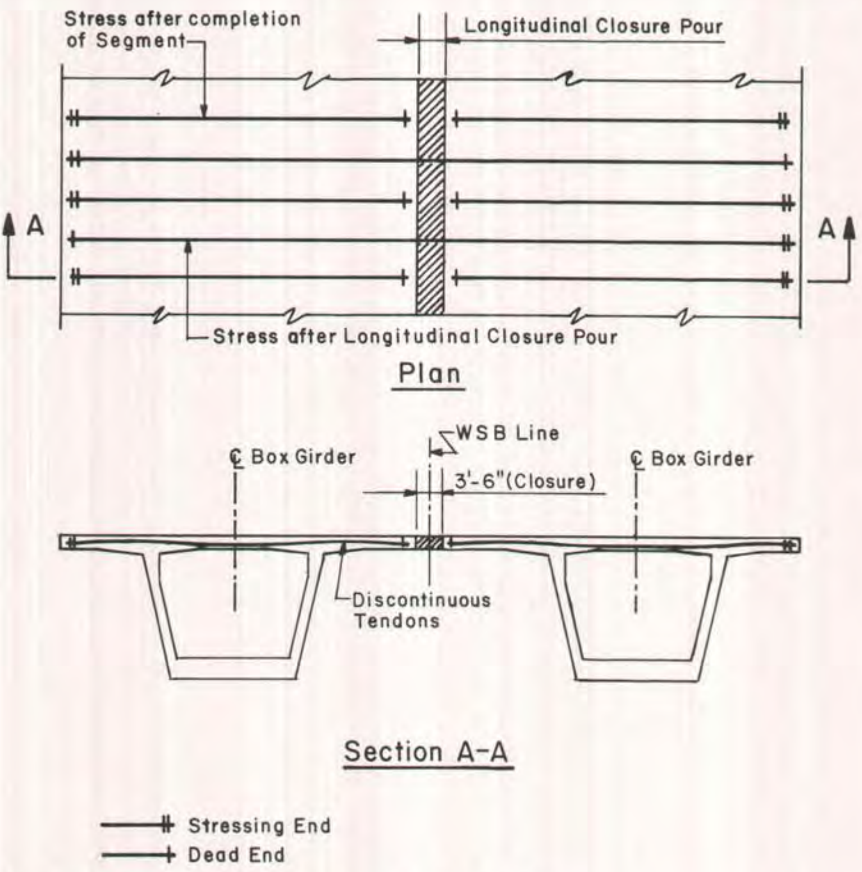

Fig. 6. Plan and section of transverse tendon details.

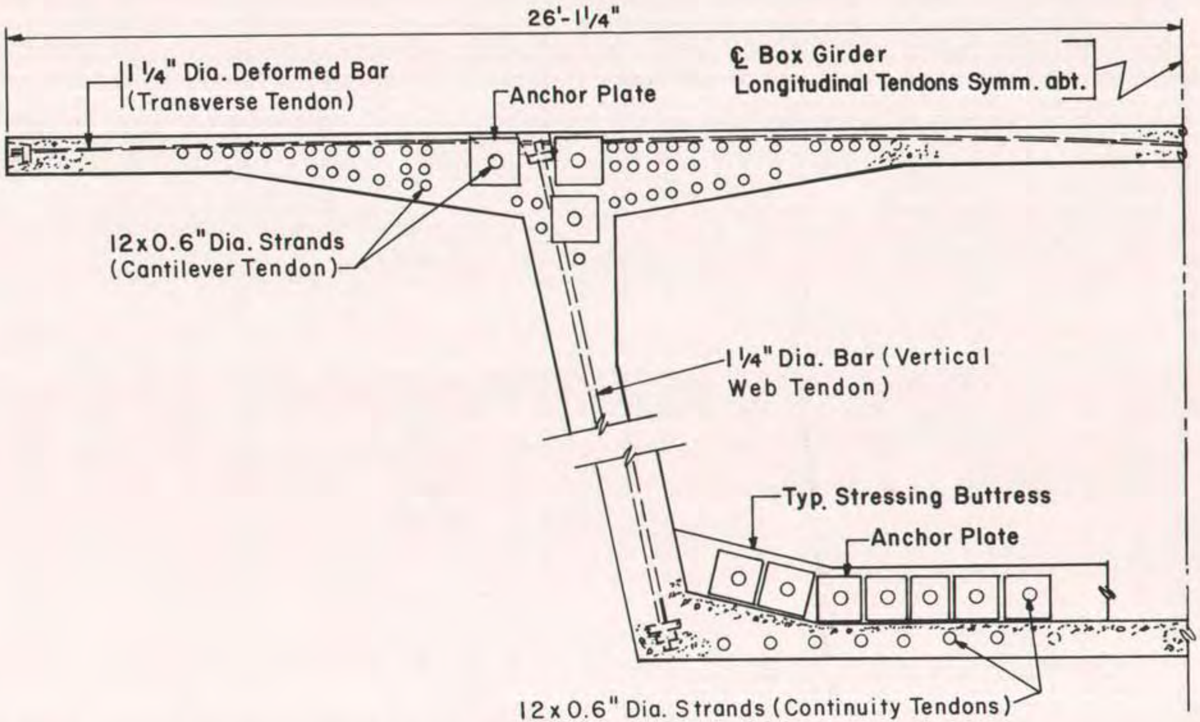

Fig. 7. Tendon arrangement inside box segment.

Straight tendons are preferred for their ease of installation and ability to minimize friction losses. They also prevent the necessity of having a wide web to accommodate the anchor plates, as they are all anchored away from the webs.

Additional longitudinal bar tendons are installed across the deck slab of the closure segments. They are required to resist the negative moments induced by live loads before creep-induced moments settle in.

For transverse and vertical tendons, 1¼ in. (32 mm) diameter deformed high strength bars are preferred. The screw type bar tendon anchorages reduce the seating loss to about $\frac{1}{10}$ in. (2.5 mm), rendering them most effective for short-length prestressing. However, strand and wire systems providing equivalent effective forces are also permitted.

Corrosion Protection

For corrosion protection of reinforcing bars and prestressing tendons, a minimum concrete cover of 1½ in. (38 mm) was specified for the deck slab in addition to the wearing surface. Epoxy coating was required on all top mat deck bars as well as web bars having less than 3½ in. (89 mm) concrete cover, measured from the top of the monolithically-cast deck slab. To eliminate the need for epoxy coating long web bars, a special detail was developed (Fig. 8).

One of the following measures must be taken to protect the transverse ten-

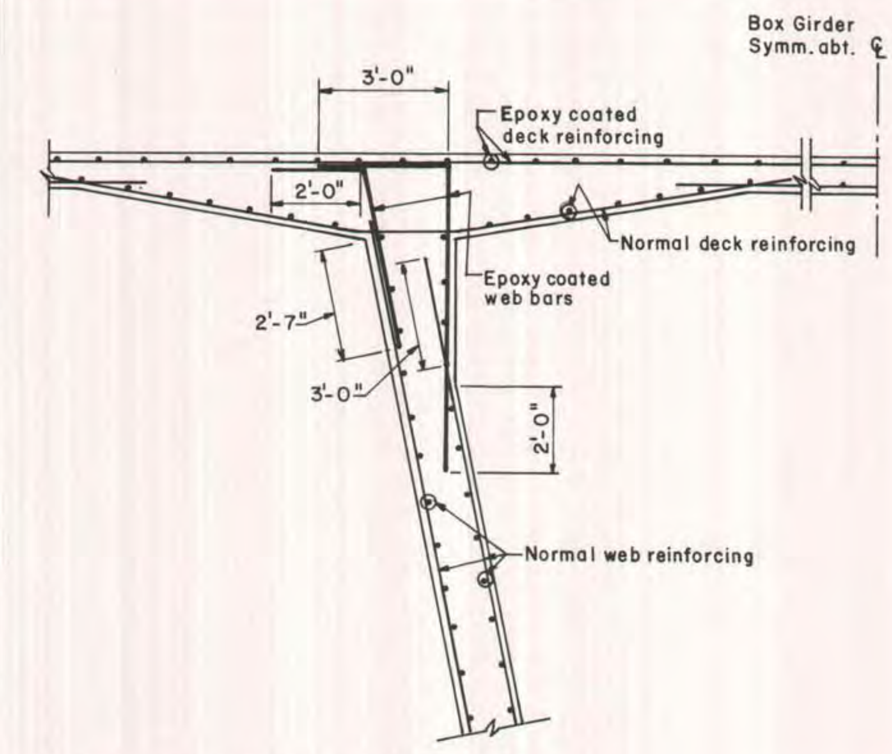

Fig. 8. Epoxy coated web bars in box segment.

dons, depending on the type of tendons used: (1) epoxy coating the deformed high strength bar tendons, (2) using plastic ducts with spiral corrugations, either polyethylene or polyvinylchloride, (3) using epoxy coated metallic ducts.

Because the longitudinal tendons are located well below the riding surface, galvanization of the metallic ducts is considered adequate for their corrosion protection.

Special Requirements

The following prestressing-system related requirements were specified:

1. Pull-out tests: If epoxy-coated flat or plastic ducts are used for transverse tendon sheathing, pull-out tests must be performed to prove the bonding-strength adequacy of the ducts. The tests should show that a force equal to 40 percent of the ultimate tensile strength of the tendon, $0.4 f_{pu} A_{ps}$, can be transferred from the tendon through the duct to the surrounding concrete at a length of 2 ft 6 in. (0.76 m). A total of 12 pull-out tests should be conducted of which 10 must pass the bonding requirement.

2. Lift-off tests: As short tendons are apt to lose too much prestressing force due to slip at anchorages, tendons shorter than 20 ft (6 m) must be equipped with an anchorage device which allows lift-off checks and restressing. This requirement applies to most vertical tendons.

3. Water freezing prevention: Covers and plugs should be provided to prevent rain water from entering vertical ducts. As a complete seal of ducts at deck level is always difficult, some water will inevitably be collected, and its subsequent freezing could cause cracks in the webs or delamination of the bottom slab. Unless a complete seal can be achieved, the ducts should be filled with a mixture of glycol antifreeze and water when the temperature falls below 45 F (7 C).

Design Considerations

Although sections subjected to prestressing forces were designed according to allowable stress requirements, their ultimate strengths must be checked by load factor design, as specified by AASHTO. For the longitudinal direction, allowable stress checks are performed at all construction stages, i.e., when a new segment is added, a form traveller advanced or removed, additional tendons stressed and a closure poured. In addition, stress conditions at completion of the bridge, 18 months after completion, and at infinity must be evaluated to insure the soundness of the structure with all prestress losses and creep effects considered.

In the transverse direction, the cross section was analyzed as a frame structure. In order to take longitudinal distribution of wheel loads into consideration, influence surfaces prepared by Homberg[7] were used to determine the equivalent beam fixed end moments prior to the frame analysis. The method is described in detail in the PTI Box Girder Bridge Manual.[8]

The load factor method was used for designing reinforcement in the webs and bottom slabs, whereas allowable stress design was used to determine the transverse tendon spacing and profiles. Since only half of the transverse tendons run continuously across the longitudinal closure, dead load stresses and secondary moments introduced by the discontinuous tendons need to be estimated. This is done by using a separate structural model consisting of only one box girder section.

Vertical bar tendons were selected as the main reinforcement for shear force. However, in applying AASHTO design equations specified in Article 1.6.13, half of the nonprestressed reinforcement required for transverse flexure design is considered effective in resisting shear forces. This is justified, since po-

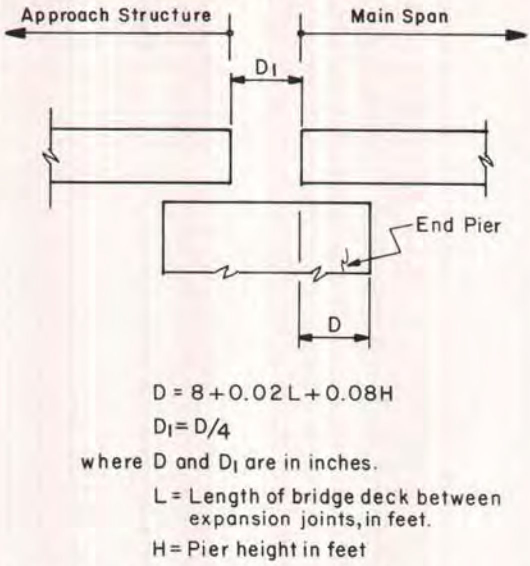

Fig. 9. Minimum gap and seating length requirement.

tential shear failure normally originates at the half-depth location of the web where transverse bending stresses are not critical.[9]

For seismic design, recommendations developed by the Applied Technology Council (ATC-6)[10] were followed. The design objective is to ensure that in an earthquake, with a 50-year probability of occurrence, the bridge will remain usable for emergency passage, although it may suffer repairable damage. This corresponds approximately to designing the ultimate strength of the bridge for a load equivalent to that induced by an earthquake of a 500-year return period. The structure is designed to allow plastic hinge formation at the pier top and bottom only when its failure mechanism is reached.

Special consideration was given to the earthquake requirements for expansion joints and restrainers. Fig. 9 schematically shows the gap and seating length as specified in the design. The minimum seating length, D, was established in accordance with Article 4.9.1 of ATC-6, and the gap distance, D_1, was predicted by response spectrum analysis.[11] This provision stipulates that temperature induced movements must be added to D_1 in determining the design expansion joint gaps.

To provide a positive horizontal linkage between the main span structure and the approaches, earthquake restrainers were installed at each expansion joint. A sufficient number of earthquake restrainers were selected at each expansion joint to provide a total ultimate strength equal to 15 percent of the weight of the main span structure. The length of the restraining cables is such that they will accommodate without rupture to the total joint movement during an earthquake. In order to transmit this restraining force to the remainder of the superstructure, additional tendons were provided at the weak parts of the side spans.

Contract Awarding

A total of six bids was received on October 1, 1980: five for cast-in-place box girders and one for steel box girders with an orthotropic steel plate deck. No bids were received for the precast concrete superstructure alternate. The bids for the concrete structure varied from approximately $24 to $28 million, and

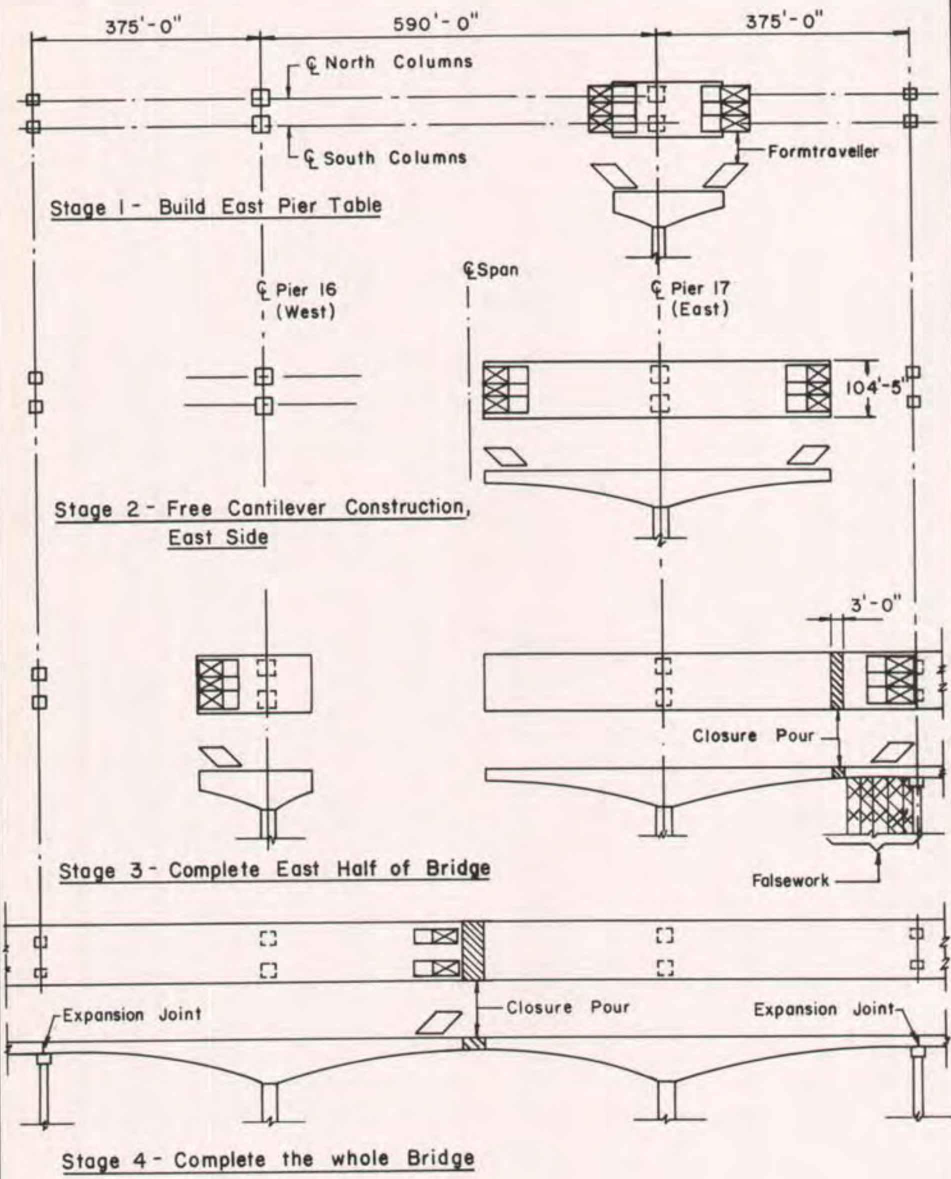

Fig. 10. Actual construction sequence.

Table 3. Comparison of tendon system from as designed and as built condition.

Tendon location	As designed	As built
Longitudinal	12 x 0.6 in. (15 mm) ϕ strands	19 x 0.5 in. (13 mm) ϕ strands
Transverse	Epoxy coated 1¼ in. (32 mm) ϕ deformed bar	4 x 0.6 in. (15 mm) ϕ strands with epoxy coated flat duct
Vertical	1¼ in. (32 mm) ϕ deformed bar	6 x 0.5 in. (13 mm) ϕ strand

the sole steel bid was approximately $40 million. The winning bid put the construction cost of the structure at about $170 per sq ft ($1935 per m²). A breakdown of the bid is as follows:

Common Items (mobilization, utilities, etc.) ....$3.3 million
Substructure (steel pipe piles, foundation and reinforced concrete pier columns) ....$9.1 million
Superstructure (post-tensioned structure) ....$11.4 million

Construction Modifications

Basically, the bridge was built in accordance with the conforming cast-in-place design. However, the following modifications were furnished by the contractor:

1. Instead of casting the two box girders separately, the contractor elected to pour the complete bridge section all at once. This was accomplished by tying two parallel form travellers together during concrete casting. This modification eliminated the need for leaving a longitudinal closure between the two box girders. Since all transverse tendons were made continuous, any possible misalignment of the transverse tendon ducts was avoided. This simplified the construction but required a more intensive labor force, reducing flexibility of manpower allocation.

Fig. 10 indicates the construction sequence selected by the contractor. The girder cross section was kept the same as that shown in Fig. 3. The segmental length of 16 ft 6 in. (5 m) specified by the designer was used in construction.

2. As permitted in the specifications, the contractor had the option of selecting any post-tensioning systems providing they met the requirements specified in the special provisions. Table 3 compares the designed systems with the contractor's choices.

Although the contractor replaced the 12 x 0.6 in. (15 mm) diameter strands with 19 x 0.5 in. (13 mm) diameter strands for the longitudinal tendons, the pattern, distribution, and anchorage location of the tendons remain the same as shown in the design drawings. The spacings of vertical and transverse tendons were adjusted to provide the minimum effective forces required by the designer.

3. As described previously in the construction sequence (Fig. 5), the designer recommended that the unbalanced parts of the side spans be built segmentally by the form travellers, with the help of temporary supports. However, the contractor elected to construct that portion of the side span totally on falsework (Fig. 10). To allow room for adjusting misalignment, a 3-ft (0.91 m) wide closure was provided between the end of the side span cantilever and the falsework-supported portion. Since the contractor dismantled the form travellers in the proximity of the east end closure prior to making the closure pour, clamping the ends together proved to be difficult, and concrete block counterweights were placed on the higher side of the two free cantilevers to achieve vertical alignment.

Concluding Remarks

The construction went smoothly with only a few delays at the beginning of the job. The first two segments took about 2 months to complete. Once the crews became familiar with the repetitive operations of advancing the form travellers, placing reinforcing bars and tendon ducts, concreting, stressing the tendons and grouting the ducts, their productivity dramatically improved. Near the end of construction of each balanced cantilever, it was not uncommon for two segments to be completed within a week.

Fig. 11. West Seattle Bridge, halfway through construction.

A special feature of this project was the close involvement of the consultants in nearly all aspects of the construction. Continuous review of construction progress and daily field inspection were provided by the design engineers. This approach proved to be effective in ensuring that the construction met the intent of the contract documents.

There was one incident which exemplifies the importance of careful field inspection. Inspectors noticed that the brownish color of a freshly cast segment appeared to be abnormal. It was later discovered that an undue amount of fly ash had been accidentally mixed into the concrete by the supplier through a mechanical failure in the mixing process, reducing the concrete strength considerably.

Fortunately, the mistake was caught before any forward segments were cast and corrected by jackhammering off the

Fig. 12. West Seattle Bridge, nearing completion.

Fig. 13. West Seattle Bridge, completed and in-service.

defective segment. Had additional segments been poured without the correction, devastating consequences could have resulted.

Figs. 11 through 13 show various views of the bridge during construction and after completion. The bridge was opened for traffic in November, 1983. During the last 8 months the structure has performed with total satisfaction.

Acknowledgments

The following are those chiefly responsible for the realization of the West Seattle Bridge, and their areas of responsibility:

Owner: The City of Seattle, Washington; Bruce Wasler, Project Manager.

Prime Consultant: West Seattle Bridge Team, composed of: Anderson-Bjornstad-Kane-Jacobs, Inc., Kramer, Chin & Mayo, Inc., Parson Brinckerhoff, and Tudor Engineering Co.; Thomas A. Kane, Project Manager.

Contractor: Kiewit-Grice.

Superstructure Subconsultant: Contech Consultants, Inc.; M. C. Tang and J. B. Louie, Principal Designers; Paul Brallier, Construction Inspector.

REFERENCES

1. Kane, T. A., Carpenter, J. E., and Clark, J. H., "Concrete Answer to Urban Transportation Problems," *Concrete International,* August, 1981, pp. 85-92.
2. Kane, T. A., Carpenter, J. E., and Clark, J. H., "Approach Spans to the West Seattle Bridge," PCI JOURNAL, V. 28, No. 6, November-December 1983, pp. 58-67.
3. *Standard Specifications for Highway Bridges,* 12th Edition, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 1977 (with 1978 and 1979 Interim Supplements).
4. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, Michigan, 1977.
5. CEB/FIP Joint Committee, *International Recommendations for the Design and Construction of Concrete Structures,* Comité Europeén du Béton/Fédération Internationale de la Précontrainte, Cement and Concrete Association, London, 1970.
6. *Post-Tensioning Manual,* Third Edition, Post-Tensioning Institute, Phoenix, Arizona, 1981.
7. Homberg, Hellmut, *Fahrbahnplatten mit Veraenderlicher Dicke,* Springer-Verlag, New York, N.Y., 1968.
8. *Post-Tensioned Box Girder Bridge Manual,* Post-Tensioning Institute, Phoenix, Arizona, 1978.
9. Jungwirth and Baumann, "Shear Design with Dywidag Prestressing System," In: *Finsterwalder Festschrift,* Verlag G. Braun, Karlsruhe, 1973.
10. *ATC-6 Seismic Design Guidelines for Highway Bridges,* Applied Technology Council, Berkeley, California, 1981.
11. Mahoney, T. F., and Clark, J. H., "Seismic Design — West Seattle Bridge," Paper Presented at the Northwest Bridge Engineers' Seminar, Olympia, Washington, October 4, 1983.

* * *